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Mechanical Testing of an Artificially Cemented Carbonate Soil

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SUMMARY This paper describes the manufacture and mechanical testing of an artificially cemented carbonate soil. The testing included unconfined compression, triaxial compression and direct shearing. The behaviour of the artificially cemented material was found to be similar in many respects to that observed in tests on naturally cemented carbonate soils, particularly calcarenite. Also presented is an elastoplastic model to describe the behaviour of this kind of material when subjected to direct shearing. Good agreement was found between the model predictions and the test results.

1. INTRODUCTION

Significant difficulties have been encountered with the foundations for some offshore structures in areas where the seabed consists of calcareous sediments, e.g. see Semple (1987) for a recent discussion of this problem. Many of these sediments are of biological origin and contain fragile carbonate particles such as shell fragments and skeletal remains that are often weakly cemented together by precipitated carbonate material. Sediments of this type are susceptible to crushing and can undergo significant volume reductions when sheared. Their mechanical behaviour can be markedly different from that of silicate soils and thus conventional procedures for foundation design, based on experience in silicate soils, generally will not be applicable.

The need to gain a better understanding of the behaviour of this type of material has stimulated much research and the results of some of this work have been published in a recent international conference devoted entirely to the calcareous soil problems (Jewell, 1988). For some time now, the mechanical behaviour of these materials has been studied at the University of Sydney in the School of Civil and Mining Engineering. A number of projects have investigated the behaviour of both uncemented and naturally cemented calcareous soils. The natural variability and the high cost of obtaining undisturbed naturally cemented soils has limited the scope of investigations into the behaviour of cemented materials until quite recently. Two current projects are now making use of artificially cemented materials in an endeavour to better understand this behaviour (Allman, 1988; Boey, 1989). In this paper some of the work carried out in one of these projects is presented. The method used to prepare one artificially cemented material is described, together with the results of mechanical testing of the manufactured material. The mechanical testing includes unconfined compression, drained triaxial compression and direct shearing.

A particular problem of interest concerns the behaviour of axially loaded piles grouted into calcareous formations. A study has been made of the behaviour of the manufactured material in direct shear, in an attempt to physically simulate the behaviour near the interface between a grouted pile and the surrounding carbonate deposit. Also presented is an elastoplastic model that may be used to predict numerically the behaviour of this kind of interface when subjected to direct shearing. Good agreement has been found between the model predictions and the test results.

2. SAMPLE MANUFACTURE

The major advantage in using an artificial material in this study is that some control can be exercised over sample variability, like the amount of cementation, the density and the moisure content. In particular, the significant variations in strength and consistency of the natural materials have been avoided.

The materials used to prepare the test samples were a natural calcareous soil and "hydrostone" cement (plaster). The calcareous soil was obtained from the North-West shelf region of Australia and the sand to silt sized particles were composed mostly of calcium carbonate (carbonate content = 90.7%). The soil is predominantly of bioclastic origin, including shell fragments, foraminifera, pteropods, coccoliths and sponge fragments (Allman and Poulos, 1988). The major mineral component of the cementing agent was gypsum.

The strength of the cemented soil samples greatly depended on the cement content, the homogeneity of the final mix and the uniformity and degree of compaction used in preparation. The strength also depended on the time after preparation at which the sample was tested. The moisture contents used in this series of tests ranged between 31 to 39% and the cement content was varied (to determine its effect), but most samples tested had a cement content of 20% by ovendried weight. The testing of most samples was carried out approximately 5 days after preparation.

The procedures used to prepare samples for the uniaxial compression, triaxial compression and direct shear tests are similar, the only major difference being in the size of the sample moulds. The dry soil and cement were mixed together thorougly with water and placed into cylindrical moulds. Once in the mould, each sample was subjected to a statically applied, average axial (vertical) total stress of 200 kPa for a period of 15 minutes. The samples while still in their moulds were then wrapped with plastic film and placed into a thick plastic bag to cure for 5 days. Some water was sprinkled into the bag to keep the contained air moist during the curing period.

Selected mechanical tests were repeated and the results indicate that the preparation procedure developed is capable of producing consistent samples, having reproducible strength and behaviour. Furthermore, this soil-cement material simulates well the mechanical response of natural calcarenites and the preparation procedure is relatively simple.

3. TEST PROCEDURES

Three different types of mechanical test were carried out on samples of the artificially cemented material. These were: uniaxial compression, drained triaxial compression, and direct shearing under conditions of constant normal stiffness. The first two of these were performed to assess the general behaviour and to allow a comparison with previously observed results for naturally cemented carbonate soils. A study of the effect of cement content was also performed using the uniaxial compression test.

The direct shear tests, carried out under conditions of constant normal stiffness, were performed on the prepared samples to simulate the behaviour at an interface between a grouted pile and the surrounding formation. The apparatus used for this type of test has been described previously by Ooi and Carter (1987). In drawing the analogy between shear failure at an interface between a grouted pile and cemented soil and the direct shear tests on these samples, an important assumption has been made. The assumption is that failure of the field interface will actually occur entirely within the cemented soil, close to but not directly at the interface with the grout. This removes the need to prepare samples containing an actual grout-cemented soil interface; samples composed entirely of cemented soil only need to be prepared. There is evidence that this assumption is valid for interfaces between grout and a material such as the carbonate soils that collapse on shearing (Johnston et al, 1988; Boey, 1989).

4. TEST RESULTS

4.1 Uniaxial Compression

A typical set of results from uniaxial (unconfined) compression tests is presented in Fig. 1. Curves of axial stress versus axial strain have been plotted for samples with various cement contents in the range 5-25%. Each sample was loaded at a constant rate of axial displacement equal to 0.2 mm/min, which meant that each failed in about 10 minutes. In all cases, except the sample with 5% cement, a pronounced peak strength was observed followed by a softening response. The variation of peak strength with cement content is plotted in Fig. 2, where it may be seen that the strength increases almost linearly with cement content over the range 10-25%.

4.2 Drained Triaxial Compression

Two series of drained triaxial compression tests were carried out; one on samples of the artificially cemented material, each having a cement content of 20%, and the other on uncemented samples of the carbonate soil. Each sample was saturated and isotropically consolidated prior to shearing.

Figs 3 and 4 show the stress-strain and volumetric behaviour for the cemented and uncemented samples, respectively. It may be observed that the uncemented soils exhibit a response that is mostly ductile at all confining pressures within the range 20-500 kPa. At low confining pressures the material exhibits a slight overall dilation, while at higher confining pressures the volumetric response is entirely contractive. The maximum strength is mobilised at relatively large axial strains, in excess of 10%.

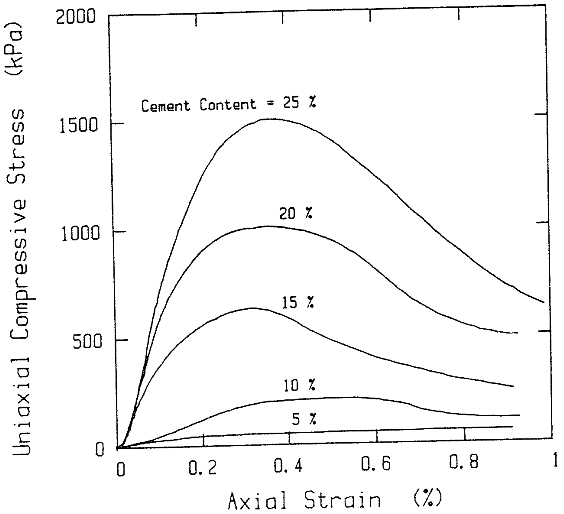

Figure 1. Variation with cement content of the behaviour in uniaxial compression

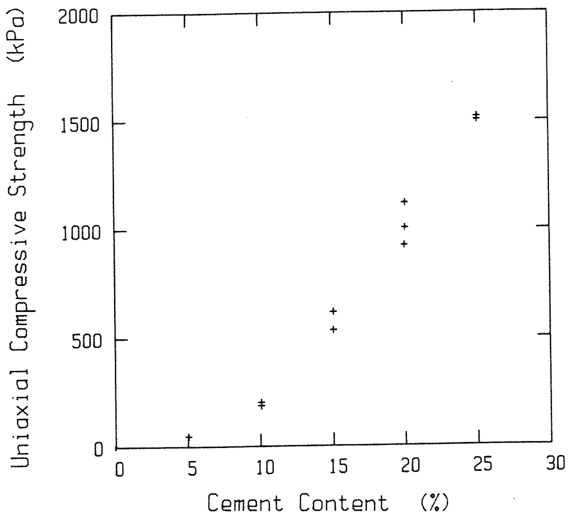

Figure 2. Effect of cement content on the uniaxial compressive strength

The behaviour of the cemented soil is somewhat different to that of the uncemented material. At all confining pressures in the range 20-500 kPa, an initial, fairly linear stress-strain response is observed almost up to the peak strength, which occurs in all samples at a relatively low value of axial strain, viz. 1-2%. At low confining pressures the material is brittle with significant post-peak softening, but the behaviour becomes more ductile as the level of confinement is increased. The volumetric response of the cemented material is similar to that of the uncemented soil, i.e. some dilation under low confining pressures and contraction during shearing at higher pressures.

4.3 Direct Shearing under Constant Normal Stiffness (CNS Tests)

Figures 5 to 9 show the results of CNS tests on samples of the artificially cemented soil (cement content = 20%). Tests were performed on essentially identical specimens using a normal stiffness of 500

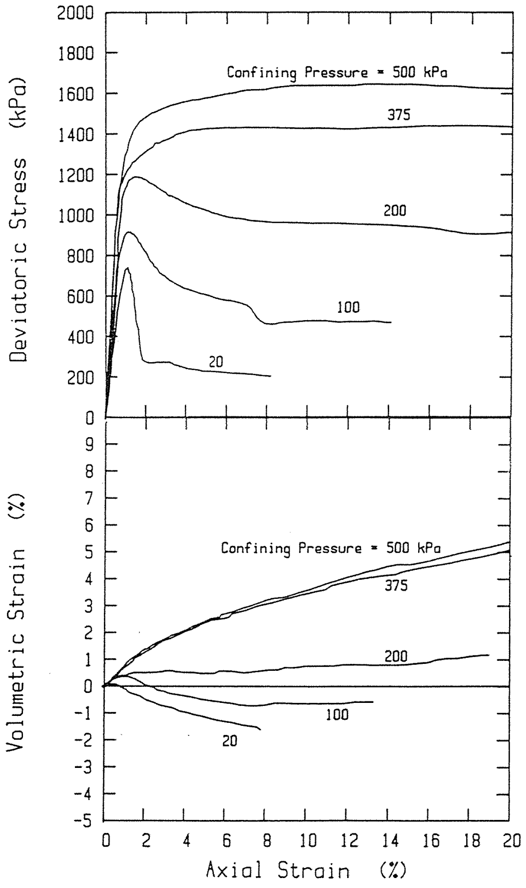

Figure 3. Typical behaviour during drained triaxial tests - cement content of 20%

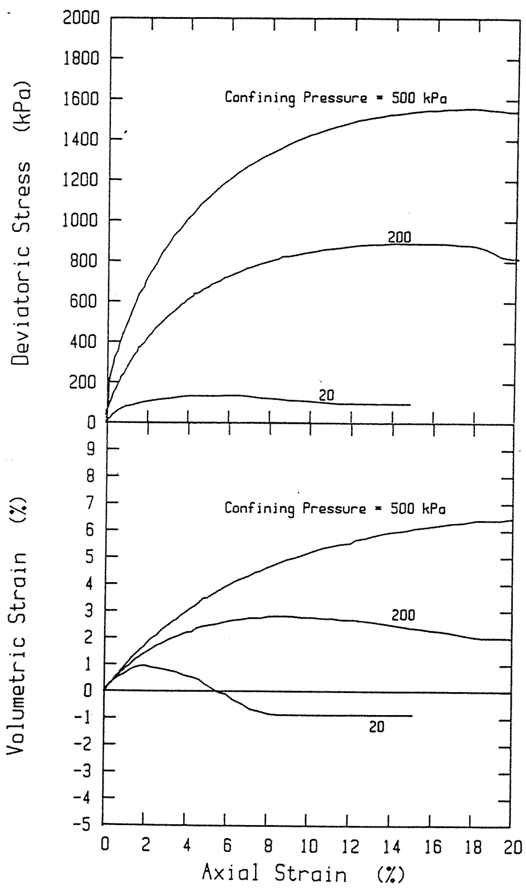

Figure 4. Typical behaviour during drained triaxial test - uncemented soil

kPa/mm in each test. The parameter varied in this series of tests was the initial normal stress applied to each specimen prior to shearing. Values in the range 200-1000 kPa were applied.

Figure 5 indicates that the peak shear strength of each specimen was mobilised after only very small shear displacement, typically about 0.2 mm. In each case a sharp reduction in strength occurred after the peak, followed by more gradual softening with increasing shear displacement. The amount of immediate post-peak softening (both in absolute terms and as a proportion of the peak strength) appears to be reduced as the initial normal stress is increased. This feature is more readily seen on the stress path plots of Fig. 7. However, in all cases the strength continued to reduce as shearing continued to relative displacements of 10 mm. Except in the case where the initial normal stress was smallest (200 kPa), the reduction in shear strength was accompanied by contraction of the specimen in the direction of the applied normal stress with a consequent reduction in the normal stress under the condition of constant normal stiffness. The normal stress response is shown in Fig. 6. The rate of decrease in normal stress is more dramatic as the initial normal stress is increased. Because the normal stiffness is the same in each test, this means that the contraction of the specimen with shearing is greatest at higher normal stresses.

Figure 7 is particularly useful for revealing the mechanisms involved in strength mobilisation and then loss with increasing shear displacement. In all samples tested the stress paths rise to a peak at almost constant normal stress, i.e. the pre-peak shear behaviour is reasonably linear, and therefore largely uncoupled from the behaviour in the direction of the normal stress. Very soon after the peak, the stress paths lie closely along a line of constant slope indicating purely frictional behaviour, with a residual angle of friction defined by τ/σ of approximately 33 degrees. In general the peak strengths lie well above this line and therefore the material has both a frictional component of peak shear strength and a component due to the cementation. The cementation component provides a much greater proportion of the peak strength at lower normal stress levels than at higher normal stresses. Indeed it appears that the cementation provides little, if any, contribution to the peak strength at an initial normal stress of 1000 kPa. Reference to Fig. 2 reveals that this material (with 20% cement content) has an unconfined compressive strength on the order of 1000 kPa, and so it could be postulated that the cementation may be damaged due to application of an initial normal stress of 1000 kPa and thus its contribution to the peak shear strength may be largely removed before shearing commences. At the other extreme, friction will contribute effectively nothing to the shear strength at zero normal stress. A specimen was tested with an initial normal stress of 0 kPa and it had a strength of approximately 200 kPa. This is probably the maximum contribution that can be made by the cementation to the peak strength of this material.

Peak and residual strengths (the latter measured at large shear displacements) for a large number of CNS tests on samples with a 20% cement content are plotted in Figs 8 and 9. Fig. 8 shows the more

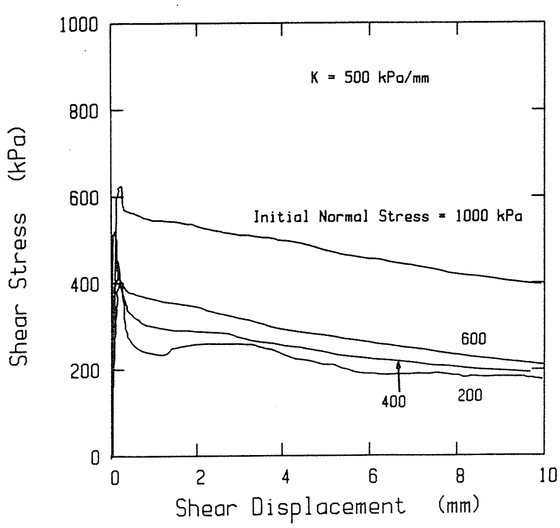

Figure 5. Effect of initial normal stress on behaviour during CNS tests

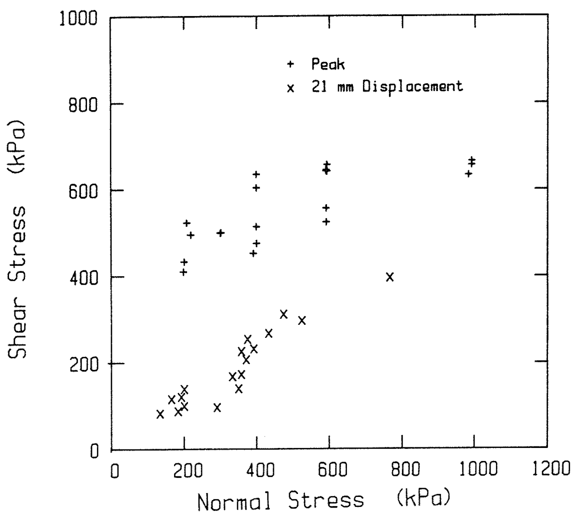

Figure 8. Strength envelopes for CNS tests

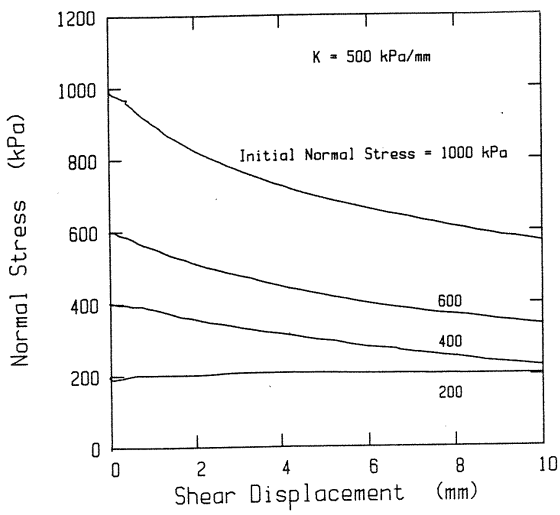

Figure 6. Changes in normal stress during CNS tests

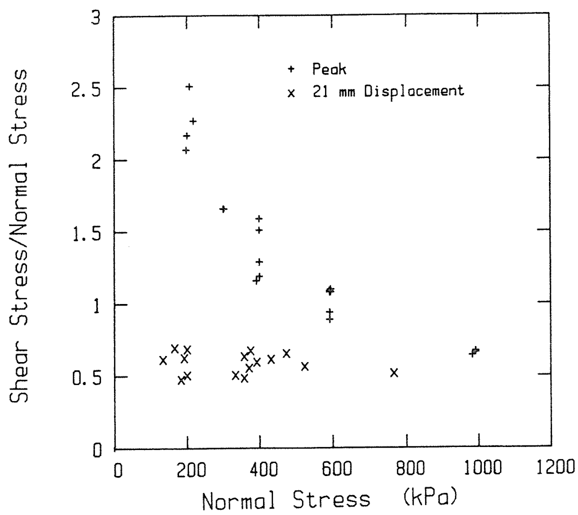

Figure 9. Stress ratio versus normal stress in CNS tests

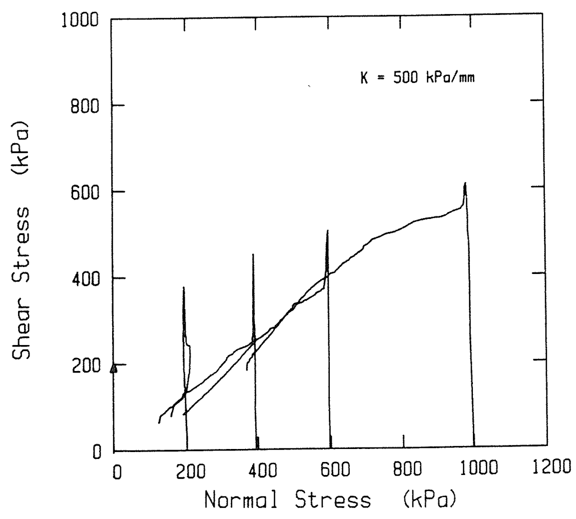

Figure 7. Typical stress paths in CNS tests

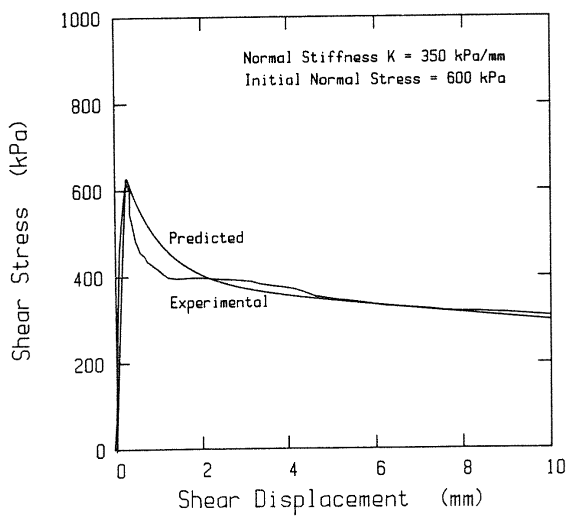

Figure 10. Comparison of experimental results and model predictions

conventional plot of shear stress τ versus normal stress σ, while Fig. 9 shows stress ratio τ/σ versus σ. Strengths measured in tests involving different initial normal stresses and a range of different normal stiffnesses have been plotted. The results indicate that the strength envelopes are largely independent of the normal stiffness and therefore are independent of the path taken to failure, i.e the same envelopes should be determined in more conventional constant normal stress, direct shear tests, though this is yet to be validated for this material. Fig. 8 indicates that the residual strength envelope is fairly well defined and linear over the range plotted. The peak envelope is less well defined on this plot and could well be curved. Less scatter is obtained when peak stress ratios are plotted as in Fig. 9. It is very interesting to note that the peak and residual shear strength envelopes apppear to be converging in Fig. 9 at a normal stress of about 1000 kPa. Again it is noted that this normal stress is approximately equal to the unconfined compressive strength of the material. It is therefore impossible to obtain shear strength data beyond about this value of initial normal stress, as the test apparatus allows a portion of the sample to be in unconfined compression during application of the initial normal stress.

5. PREDICTIONS OF DIRECT SHEAR RESPONSE

An elastoplastic constitutive model to predict the direct shear behaviour of cemented carbonate soils has been developed. In order to be concise only the essential details of the model are summarised here, but full details are contained in the paper by Ooi et al (1988).

The basic assumptions made in this model are as follows:

(1) The behaviour prior to the peak is linear elastic and thus the shear and normal modes of behaviour are completely uncoupled before the peak.

(2) Plastic yielding (slip) of the interface will commence whenever the Mohr-Coulomb criterion is satisfied, i.e.

$$\tau = c + \sigma \tan \varphi \qquad [1]$$

where c is the cohesion intercept and φ is the friction angle.

(3) Once yielding commences the strength parameters c and φ will vary with the accumulated plastic shear displacement. The forms of variation are such that the cohesive component of strength (presumably arising in part from the cementation and in part from the strength of the grains) will decay exponentially with plastic shear displacement. This implies that the interface will become more damaged as it is sheared. In contrast, the frictional component of strength is mobilised from zero quite rapidly after shearing commences. This assumption recognises that some rupture must occur and a shearing plane develop before the full friction angle can be mobilised.

(4) During yielding of the interface plastic flow will occur according to a flow rule that depends on both the amount of accumulated plastic shear displacement and the level of normal stress acting on the interface. In particular, this flow rule assumes that the interface will contract on shearing and that the rate of contraction is highest at low plastic shear displacements and large normal stresses. This type of behaviour is suggested by the test data shown in Fig. 6.

This elastoplastic model requires the specification of 8 parameters to fully describe the behaviour and all may be derived from the results of direct shear tests. An example of the quality of fit of the experimental data that may be made obtained with this model is shown in Fig. 10. The test considered here was conducted using an initial normal stress of 600 kPa and shearing was carried out under a constant normal stiffness of 350 kPa/mm. A good fit to the peak strength is shown on this plot. Obviously the model predicts a much smoother softening than was observed in the experiment, but the overall agreement is encouraging.

6. CONCLUSIONS

Results of the mechanical testing of artificially cemented carbonate soil have been presented. The testing included uniaxial compression, triaxial compression and direct shearing. The recorded data indicate that reproducible behaviour can be obtained using the sample preparation techniques described here and furthermore that the behaviour of the artificially cemented material is very similar to that of naturally occurring cemented carbonate soils. The manufactured material is thus suitable for a study of the fundamental mechanical behaviour of calcarenites and it has the particular advantage of removing from such studies the inherent variability of the natural materials. The results of these studies should be of assistance in understanding the behaviour of foundations in calcareous sediments.

It has also been demonstrated that the behaviour of shear interfaces in these materials can be captured well by a specially developed elastoplastic constitutive model.

7. ACKNOWLEDGEMENTS

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