Investigations for Rock Socketted Piles in Melbourne Mudstone

by

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SUMMARY. The proposed Johnson Street Bridge has called for some of the deepest rock-socketted foundations yet used in Melbourne. Properties of the mudstone, as revealed in laboratory tests, are discussed, together with some current concepts of socket design.

1 INTRODUCTION

The Johnson Street Bridge Project has been conceived to provide a new arterial route across the Yarra River, adjacent to the Melbourne Docks. Foundation arrangements have been complicated by the need to provide for construction of a high level bridge on the same alignment at a later date, and also by the Melbourne Main Sewer, now in a much deteriorated condition, which intersects the bridge axis at an acute angle.

Bored cast-in-situ piles, 5-1/2 ft. (1.68m) diameter and socketted into mudstone, have been proposed to support the superstructure. Driven piling was discounted because of the risk of vibrational damage to the nearby sewer, and under-reaming was not favoured because of the severe difficulty associated with de-watering and inspection.

2 GEOLOGY

The site is located within the Yarra delta, where Silurian bedrock underlies some 30 to 40 metres of Tertiary and Quaternary sediments (Fig. 1). These sediments range from soft silty clays to sandy gravels, with frequent sandy seams, and there is also a flow of Tertiary basalt over part of the site.

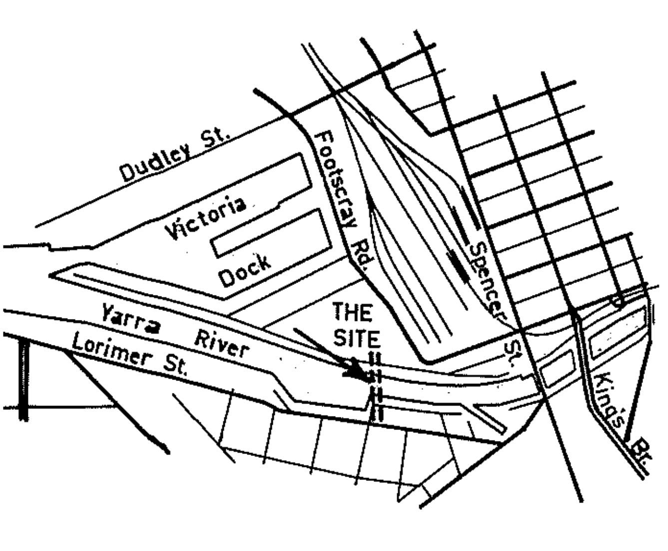

Fig. 1 Locality plan

The bedrock consists of deeply and erratically weathered interbedded mudstones, siltstones and sandstones. Dip varies from 10^o to 60^o along a strike parallel to the bridge axis, indicating moderate to severe folding, and some faulting near the southern abutment. The rock is of blue-grey color throughout, indicating non-typical weathering by the absence of the brown colouration that is usually associated with the more weathered zones. At the outset, it was not known what effect this might have on established methods of assessing core, such as Neilson (1970), or on previous correlations of strength - modulus - moisture, such as Parry (1970).

Despite the unusual weathering, drill core has been classified into the basic five weathering zones, following normal practice based on scratch hardness. These range from totally weathered rock (zone 1) to fresh unweathered rock (zone 5). It was, however, found to be both possible and advantageous to introduce half-zones (e.g. zone 3-4), which enabled the mechanical properties to be defined within closer limits than those otherwise possible. The practicability of this procedure has been tested by independent examinations of core by both the writers and MMBW geologists, which rarely differ by more than one half-zone (Donald and Parkin, 1973).

Interpretation of the borehole logs for pile design was difficult, in respect to the many variations in rock quality detailed therein. Therefore, a colour coding was introduced, ranging from red for zone 1, sequentially through the spectrum to violet for zone 5, and this proved to be of considerable assistance at the design stage.

3 STRENGTH - MODULUS - MOISTURE RELATIONSHIP

The extensive jointing and weathering of Melbourne mudstone has meant that it is normally difficult to recover sufficient intact core for mechanical testing, Thus, in the King's Bridge investigation (Wilson, 1970), a relationship between moisture content and mechanical behaviour was developed, to allow extrapolation to fragmentary material on the basis of moisture content. Subsequently, this relationship has served as a basis of comparison in other investigations, notably Soilmech (1967), Learmonth and Garrett (1969) and Parry (1970), although generally higher strengths have been obtained at corresponding moisture contents.

The classification of mudstone into weathering zones, as defined by Neilson (1970), has also been proposed and used as an aid to the extrapolation of test data. It is, however, less precise, due to the subjective element in classification.

Parry's essential conclusions were that the mudstone could, for practical purposes, be regarded as a $\phi_u = 0$ material, and that bedding and jointing have only minor influence on strength under confined conditions*. These factors have enabled the King's Bridge correlation to be adopted at least within the western city area and within the range of normal moisture contents. Whilst it should, therefore, apply to the Johnson Street Project, it was felt that this should not be assumed, in view of the unusual weathering conditions, and a fairly extensive triaxial investigation appeared necessary.

In the triaxial programme, some 60 samples of NX core were selected to cover a spectrum of hardness from zone 2 to zone 4-5. After diamond sawing, sample ends were faced square in a lathe to yield test specimens with a length-diameter ratio of 2:1. Samples were then soaked for 24 hours prior to testing in a Hoek cell. Single stage tests were performed at cell pressures of 50, 100 and 200 lb./in.^{2} (0.34, 0.69 and 1.38 MPa), controlled via a dead load piston and hydraulic accumulator. Axial load was applied through a fairly stiff proving ring, fitted with an LVDT displacement transducer, to produce a continuous record on a chart recorder. Loading was at the rate of 0.012 ins./min. (0.5 x 10^{-3} cm./sec.).

In comparison with the mean lines for King's Bridge, the results, shown in Figs. 2 and 3, indicate higher strength, as in the other investigations quoted, but a lower elastic modulus. This latter has not been reported previously and may be an effect of the different weathering at this site.

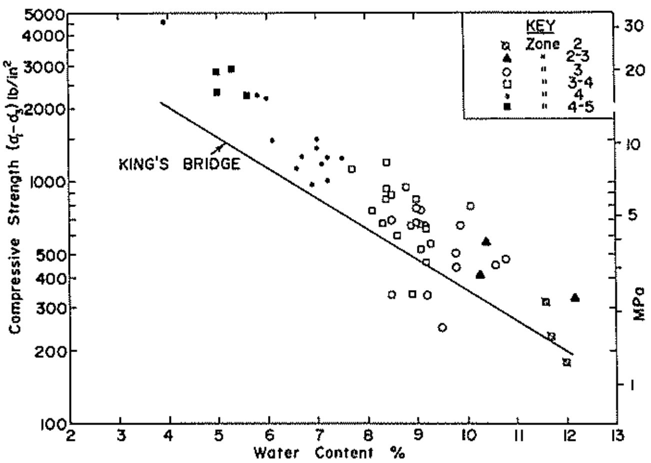

Fig. 2 Triaxial tests on Melbourne mudstone - strength.

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* The unconfined situation may be quite different, e.g. Bamford (1968), with strengths varying by a factor of 10 or more, according to dip. This however, is to be expected in a situation where tensile failure is possible, and does not conflict with Parry.

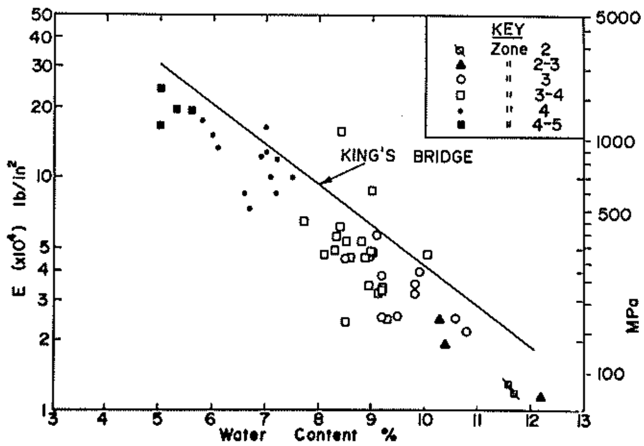

Fig. 3 Triaxial tests on Melbourne mudstone - modulus.

By extracting typical values from Figs. 2 and 3, strength is found to increase by a factor of 1.6 and modulus by a factor of about 1.9 between each half-zone. There is not, therefore, a characteristic E/c ratio comparable with those often quoted for clays (e.g. Skempton and Henkel, 1957), although the ratio $E^{0.7}/c$ gives a fairly constant value for each of the sites investigated. With E and c in ton/ft.^{2}, the values are about 25 and 16 for King's Bridge and Johnson Street respectively.

4 SUPPLEMENTARY CONSIDERATIONS OF SHEAR STRENGTH

(a) Effect of Confining Pressure

Available evidence indicates a trend to increasing strength with cell pressure, but the data are not plentiful, and are often erratic. Parry (1970), for example, found ϕ_u to be generally insignificant, but ranging up to 23° in some instances (which were not specifically identified). The writers' tests (Donald and Parkin, 1973) also suggested that there is an appreciable ϕ_u, although their programme was not designed to evaluate this in depth.

A testing programme to evaluate the effect of confining pressure was subsequently carried out by Fletcher and Parker (1973), under the authors' direction. Stage testing was considered essential, because of the great variability of mudstone core, and the difficulty of selecting matching samples based on scratch hardness. In these tests, 33 in all, cell pressure was applied in three increments of 100 lb/in^2 (0.69 MPa), using the same equipment and preparation as previously. The third stage was continued past peak to ultimate, and the residual envelope was defined by stage unloading.

Typical results for hard and soft samples are shown in Fig.4, in the form of p-q plots. In this case, the slope of the failure envelope, ψ, is related to the angle of shearing resistance ϕ_u by the expression

$$\sin \phi_u = \tan \psi$$

Appreciable values of ϕ_u were indicated for both peak and residual conditions, although peak values were less clearly defined. Values of c_r were evidently small.

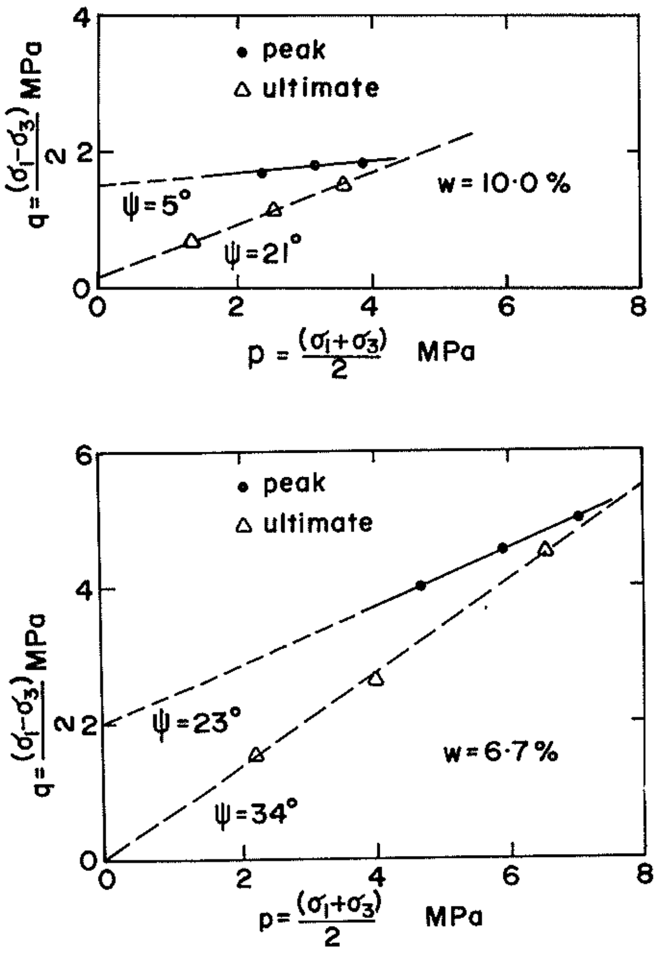

Fig. 4 Stage Triaxial Tests on Mudstone.

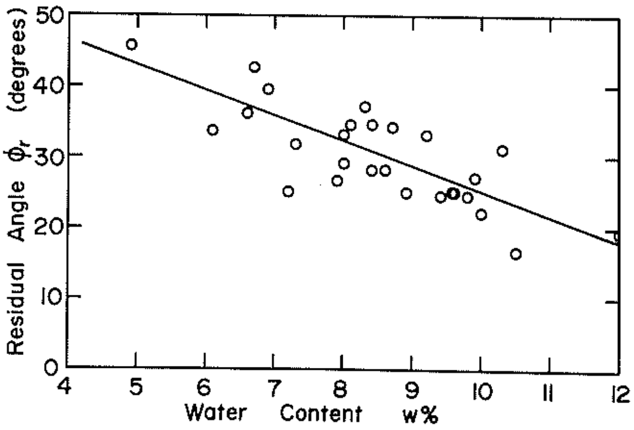

Fig. 5 Residual Strength - Moisture Relationship.

Peak angles of shearing resistance, when plotted against water content, showed a trend from values of 25° to 30° at low water contents down to 0° to 5° at high water contents, with a mean of about 15°. Both c_u and ϕ_u appear to increase with hardness, although c_u is the dominant strength component at stress levels up to 100 lb/in^2 (0.69 MPa).

Residual angles of shearing resistance were generally about 30°, and were subject to less scatter. Fig.5 shows that ϕ_r decreases somewhat with increasing water content, which probably indicates the increasing significance of weathering products as weathering proceeds.*

It would be expected that undrained tests should yield $\phi_u = 0$, and the reason for the commonly observed ϕ_u values is that the pore-pressure parameter B is significantly less than 1 due to a combination of incomplete specimen saturation and relatively high mineral skeleton rigidity. Adachi and Mesri (1973) for example recorded B values as low as 0.2 for a sandstone, even after using substantial back pressures before testing. As there are no special procedures to ensure saturation in the routine testing of mudstone, B values may well approach zero. In this case, the undrained test would become essentially a drained test, with a significant frictional component of strength.

It may well be that pore-pressure dissipation around a pile socket in mudstone is rapid, such that drained strength is applicable. At the same time, however, c-ϕ analysis is complex and unproven, and it is a valid simplification to treat the mudstone as a ϕ=0 medium, provided strength tests are carried out at appropriate values of cell pressure.

(b) Effect of Dip Angle

In examining the effects of dip angle, it is assumed (as, for example, in the theoretical analysis of Jaeger and Cook, 1969) that there will be a diminished shear resistance in the plane of bedding . A minimum strength, of the order 3/4 maximum, is thus predicted at some particular orientation of bedding. In examining this experimentally, however, it is necessary that extraneous failure mechanisms (viz., tension) are not permitted, and hence unconfined tests are excluded.

Whilst work such as Chenevert and Gatlin (1965) on sedimentary rocks has closely followed the predicted behaviour of Jaeger and Cook, there is no evidence that this is so for Melbourne mudstone. Fletcher and Parker (1973) found from their stage tests on samples of varying dip and water content that there was no correlation between strength and direction of bedding planes. From this it is concluded that the effect of dip is insignificant for this material, and that bedding does not introduce planes of inherent weakness, as might be implied from commonly observed shear failures in the direction of bedding (Learmonth and Garrett, 1969).

5 PROBLEMS IN THE DESIGN OF BORED PILES

In the design of bored socketted piles in materials such as Melbourne mudstone, the establishment of laboratory parameters is one of the lesser problems. Several important questions, which are not usually significant for softer materials, require judgement in the absence of adequate field measurements or experience at this stage.

* The dependence of ϕ_r on mineralogy, for various mineral mixtures has been examined by Kenney (1970).

(a) Variations in Rock Quality.

Melbourne mudstone in general, and at this site in particular, exhibits erratic and frequent variations in quality, including totally weathered clay seams to depths of 65 metres and beyond. Often, contrasting rock occurs in alternating seams of only about 10 cms. thickness. Sockets, on the other hand, may extend over a depth of 6 metres or so, and the location of a reasonably homogeneous rock zone can be difficult.

In the absence of any definite evidence, Morgan (1974) has offered guidelines based on arbitrary modifications to socket length and position. Thin seams, aggregating less than d/2 (d being pile diameter) are considered insignificant, whereas thick clay seams, exceeding d/2, should be excluded from the socket altogether. This should, however, be considered against the comments of Hobbs (1966).

An alternative approach has been used by Donald and Parkin (1973), in which it is considered that the ratio (σ/E) should be made compatible between contrasting rock zones. To ensure this, a reduction factor of about 15% should be applied to the working stress in the softer material for each half-zone difference in weathering, reflecting the non-constant (E/c) ratio.

(b) Shatter Zones and Jointing.

Recovered core is usually somewhat broken to highly shattered, and poses two questions. Firstly, it must be judged whether a similar degree of shattering exists in-situ, or whether it has been caused during drilling. For example, severe degradation of core from over-water borings was known to have been due to the penetration of gravel down the outside of casings. Intense shattering is also a factor causing core loss, which does not always indicate seams of highly weathered material. Secondly, there must be some assessment of the significance of breakage in relation to socket performance, although there is little factual experience for guidance.

The design of sockets in rock, as opposed to soil, is made difficult by the fact that no failure mechanism is known (Thorburn, 1966). Adjustments for jointing (and dip), as required by the British Code of Practice, are therefore rather academic. One of the few attempts at a rational design procedure for a jointed material is a paper by Peck (1969), which combines indentation theory with a joint strength reduction factor (C). However, even apart from the fact that the validity of the combination is open to question, the joint reduction factor, derived from Duncan and Hancock (1966), has a very flimsy basis. Using a supposed mean value of C = 0.12 the allowable working pressure under a pile, q_w, is given by $q_w \doteqdot c$ for rocks having $\frac{E}{c} > 600$*. For Melbourne mudstone although $\frac{E}{c}$ is much less q_w values of this order are in common use, but the agreement with Peck's formula seems fortuitous.

A more conventional approach was adopted by Parry (1970), in relation to the design of the Kings Bridge caissons, which are mostly belled into mudstone. These were treated as near surface footings, with $N_c = 6$, as for cohesive soils, and a total factor of safety of 5 was used to incorporate the effects of fracturing. This led to $q_w = 1.2$ c.

One of the best physical simulations of fractured rock is sintered marble, as tested in triaxial compression by Rosengren and Jaeger (1968). This material is not seriously incompatible with mudstone in the matter of hardness, and whilst a high ϕ was evident in the low stress range of interest to rock socket design, the strength was shown to be about 30% of that of intact rock. Taking $N_c = 9$ and a safety factor of 2.5, an allowable stress of $q_w = 1.1$ c results.

On the evidence, as reviewed by Cullen (1974), these criteria are undoubtedly conservative. There is also an implication in Peck's paper that jointing may not be as serious for small footings as analytical methods might suggest. Meigh (1966), referring to recommended bearing pressures for near-surface footings in the British Code of Practice, notes a substantially greater allowance for jointing in the case of harder rocks than in the case of softer rocks. In the latter instance, there appears to be a negligible reduction as he states that nominal loadings are normally of the same order as the unconfined compressive strength (which may be rather less than the confined strength), i.e. $q_w = 2c$.

From the above considerations, a design criterion of $q_w = 1.3$ c would seem justifiable for large bored piles in the Melbourne situation. However, because of the critical nature of these structures, which is such that the tolerable risk of failure is extremely small, major up-rating of bearing pressures would be imprudent unless fully supported by load tests or experience in-situ.

(c) Shaft Adhesion

Available skin friction is normally assessed in terms of the adhesion factor, α, in the equation

$$f_s = \alpha . c_u$$

It is generally understood that α decreases with increasing stiffness of the medium. Quoted values for stiff clays, however, vary from 0.15 - 0.25 (Derrington, 1966) to 0.30 - 0.45 (Skempton, 1959).

In mudstone, additional factors must be considered. Smoothness and smear of the socket walls may significantly impair bond, as might concrete shrinkage. Also, stress-strain behaviour is different from clay soils. Hobbs (1966) has noted that adhesion is fully developed at very small displacements, and that brittle failure will normally ensue. In tight rock, sockets may continue to function by virtue of friction and Poisson's ratio effect under axial load, but support capacity could not be so maintained in open rocks. In rock containing weak layers, significant base movement might be required in order to develop socket action. Accordingly, Hobbs questions whether shaft and base load capacities should be additive in the normal manner (see also Cullen, 1974).

* It is difficult to choose a reliable mean value of C from Duncan and Hancock's data. There is also a contradiction in Peck's recommendations for q_{ult} and q_w (both equal to 2 c).

The adhesion strength of Melbourne mudstone was examined by laboratory pullout tests. Cores of varying hardness were first grouted into steel shells, to prevent shattering, and a 1/2 inch dia. hole was then bored on the axis. After grouting in a 3/8 inch rod and curing for 10 days, pullout tests were performed. Subsequently, the outer shell was removed and undrained triaxial compression tests were performed. Fig. 6 indicates that a linear relationship exists between bond strength and c_u, equivalent to $\alpha = 0.14$. A survey by Cullen (1974) of current world practice suggests α values of up to 0.5 are in use. Because of the difficulty of adequately simulating field conditions in laboratory samples it was felt that the measured α value represented a lower limit and $\alpha = 0.2$ was therefore considered a reasonable design figure. Higher values are possible but would have to be justified by field testing.

(d) Groundwater Aggression*

Although substantial sulphate is present, the principal corrosion hazard in the South Melbourne area arises from aggressive carbon dioxide, which can cause significant reductions of concrete compressive strength. Traditional investigation has been found to be inadequate for a definitive assessment of the corrosion risk, and, in the light of Johnson Street experience, attention is drawn to the following points. Firstly, there must be assurance that water samples are truly representative and uncontaminated, which may require the installation of piezometers for this specific purpose. Secondly, a total chemical analysis of all ions is required, since aggressivity is not so much associated with specific ions, as with combinations (Biezok, 1967). In the South Melbourne area, for example, there is corrosion risk arising from high dissolved CO_2 associated with low hardness (calcium).

6 CONCLUSION

In this paper, laboratory test data have been presented, relating to Melbourne Silurian Mudstone, which is to be the bearing medium for the proposed Johnson Street Bridge. These tests confirm that dip and normal stress have minimal effect on strength, which is of a similar order to that of other investigations. The larger ϕ values were obtained on harder samples, which may reflect the greater difficulty in maintaining saturation, but a similar trend in ϕ_r values probably reflects the increasing significance of weathering products. The derived E/c ratio for mudstone is shown to be not constant as is normally the case for clays.

Subsequently, some of the design principles of large diameter socketted piles are examined. An approach to the problem of variable layering is suggested, based on elastic compatibility, and a bearing criterion of $q_w = 1.3$ c is suggested after an examination of the problems of shattering and jointing, which are of considerable significance in this area. A shaft adhesion factor of 0.2 has been adopted, aided by pullout tests, and comments are made relating to groundwater aggressivity, which mainly originates from aggressive CO_2 in the South Melbourne District.

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* The authors gratefully acknowledge the expertise of their colleague, Assoc. Prof. B.W. Cherry.

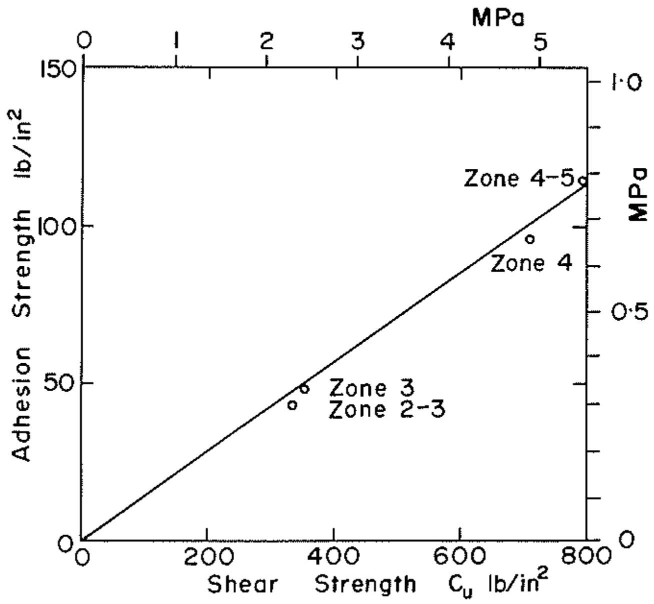

Fig. 6 Pullout tests on mudstone

Pile load tests are the main means of advancing knowledge on this subject, but these are likely to remain rare events, as the practical difficulties are considerable. Conservative design can then scarcely be avoided, although, in the harder rock zones, it is doubtful whether failure, in the traditional sense, can occur, and design and testing concepts in the future might therefore be best based on the control of deformation.

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