THE EFFECT OF ALKALI-AGGREGATE REACTION ON THE STRENGTH AND DEFORMATION OF A REINFORCED CONCRETE STRUCTURE

G E Blight and M G Alexander
Department of Civil Engineering
University of the Witwatersrand
Johannesburg, South Africa

W K Schutte and T K Ralph
Soderlund & Schutte Inc.,
Johannesburg, South Africa

1. ABSTRACT

A portal that forms part of a major double deck road structure in Johannesburg has deteriorated, as a result of alkali-aggregate reaction, to an extent that is visually alarming. The upper portions of the columns and the upper beam of the portal show the greatest degree of deterioration, and an ultrasonic pulse velocity survey indicated that the visible surface cracking deeply penetrates the concrete.

It was decided that an assessment of the safety of the structure was required prior to taking a decision to either repair and seal the surface of the concrete against moisture ingress or partially demolish and rebuild the portal. To assess the structural safety, it was decided to carry out a full scale load test.

Prior to performing the load test, a finite element analysis of the portal was carried out on the basis of elastic moduli for the deteriorated concrete that had been measured in the laboratory on cores taken from the portal.

This is believed to be the first time that the effects of alkali-aggregate reaction on the strength of a major structure have been assessed quantitatively by means of a full scale field loading test.

KEY WORDS: Reinforced Concrete, Deterioration, Strength Deformation.

2. DESCRIPTION AND CONDITION OF PORTAL

An elevation of the portal is shown in Figure 1. The western half of the top beam of the portal and the cantilever projection to the east have experienced the worst degeneration.

Figure 2 shows the results of an ultrasonic pulse velocity survey of the west end of the upper portal beam. The numerical values represent measurements of the ultrasonic pulse velocity made with a portable 'Pundit' instrument. Pulse velocities greater than 3,5 km/s are taken to represent sound concrete. Those less than 3,5 km/s represent concrete that has undergone some degree of deterioration. The limit of 3,5 km/s had been determined earlier /1/ by comparing ultrasonic pulse velocities measured in the laboratory on cores that had suffered deterioration, either from alkali-aggregate reaction or by deliberately cracking them under stress. It is clear from Figure 2 that almost the entire beam had deteriorated to some extent. Figure 3 shows the severe deterioation of the western face of the top of the west column.

3. RESULTS OF FINITE ELEMENT ANALYSIS

The finite element analysis was performed on an elastic basis. Second moments of area for the portal were determined for the gross concrete section. Tests on cores from the portal and other related structures showed that the elastic modulus for the concrete varied from 32 GPa for the sound material to 11 GPa for badly deteriorated concrete. An intermediate value of 18 GPa was chosen for the analysis.

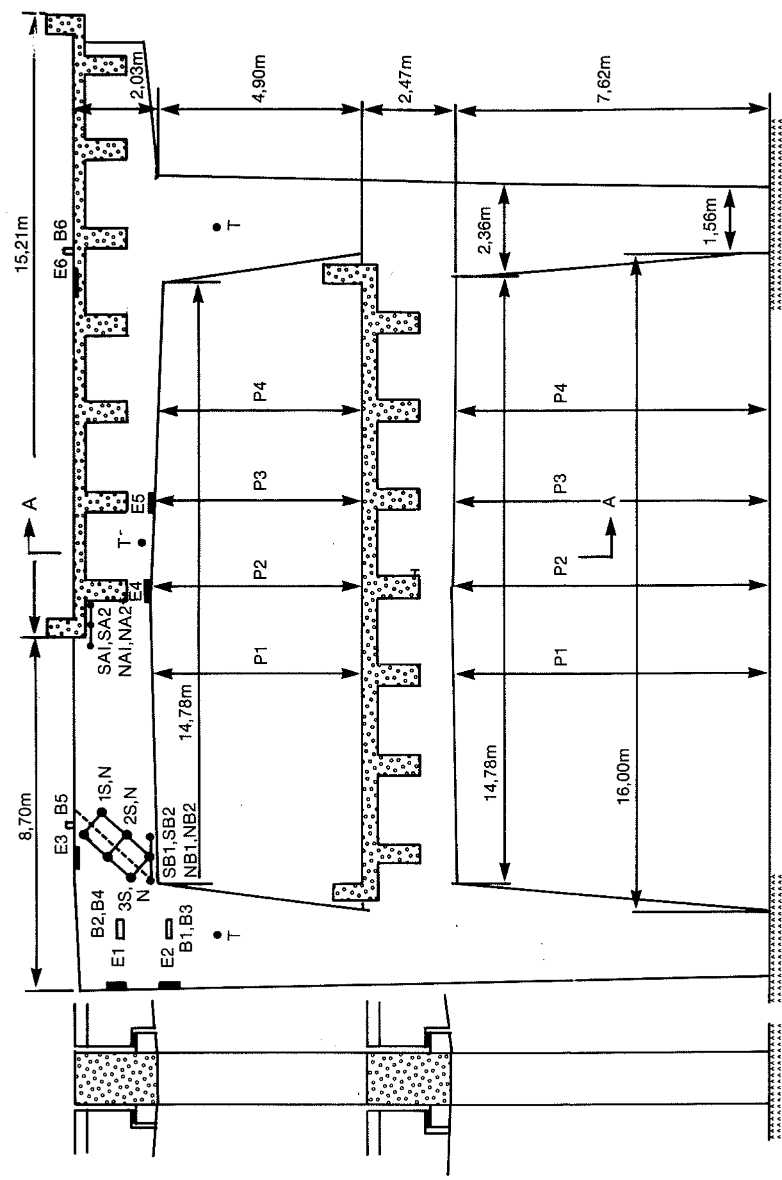


FIGURE 1. Elevation of portal frame looking north and showing positions and types of instruments:

E = electric resistance strain gauges on reinforcing steel
P = displacement gauge
B = bubble slope gauge
T = thermocouples

S, N, SA, NA, SB, NB = demec strain targets

The results of the computed deflections are shown in Figure 4. In the first series of analyses full continuity was assumed at every joint of the portal.

Because of the severe deterioration at the west end of the top portal beam, some concern was felt lest moment continuity had been partially or completely lost at this point. (J in Figure 4). For this reason, the analysis was repeated assuming no moment continuity at J.

Note that loss of continuity at J has the effect of causing the portal to sway to the west.

What is measured in a load test is the difference between displacements or

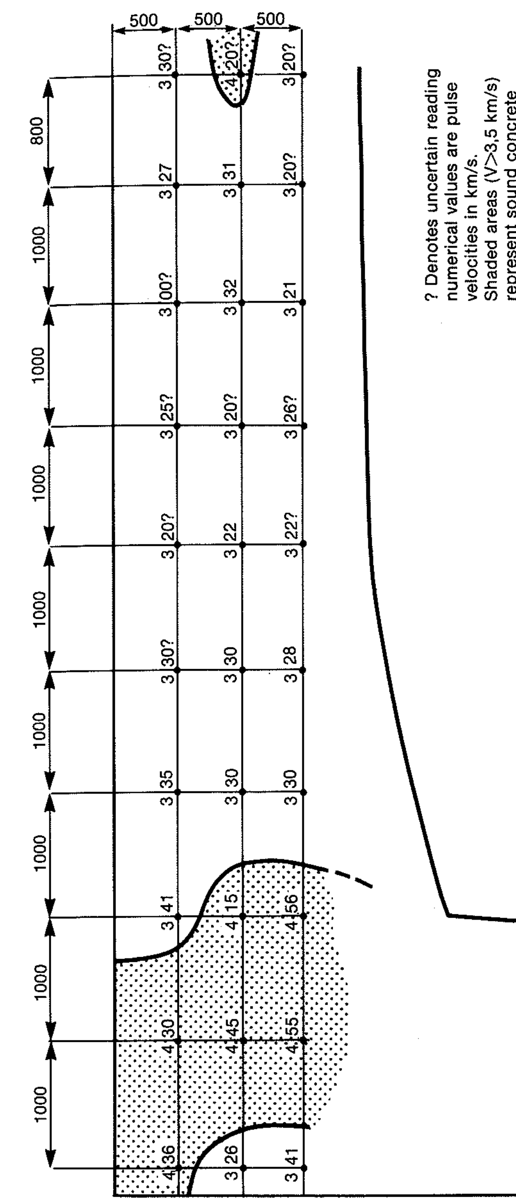


FIGURE 2. Ultra-sonic pulse survey on west side of upper portal beam.

FIGURE 3. Deteriorated condition of western face of west portal column. The scale can be judged from the pair of demec targets below the pointed figure 9 at the right. These are 100 mm apart.

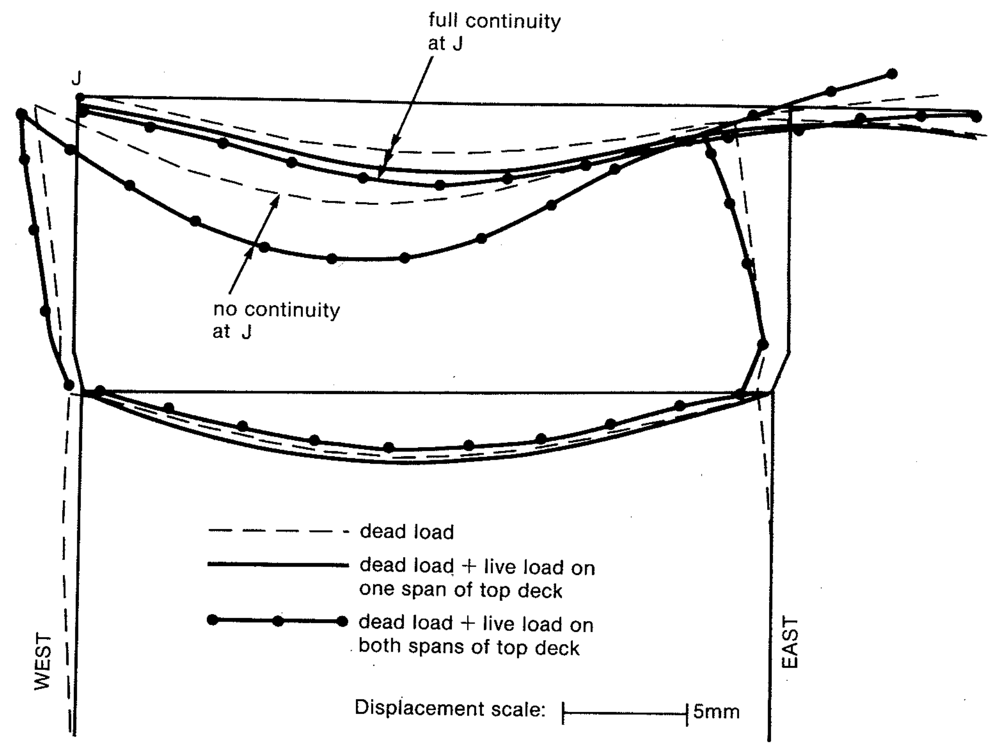


FIGURE 4. Calculated displacements of the portal members under load.

slopes under dead load and those under dead plus live load. In numerical terms, these differences are very small. The maximum expected displacement of the top beam was only 1,2 mm. The maximum rotation at point J was expected to be only 70.10^{-6} radians if full moment continuity existed and 365.10^{-6} radians if continuity had broken down.

4. THE LOAD TEST

Because of the small magnitude of the movements it was intended to measure, temperature differences between different parts of the structure had to be avoided as far as possible. For this reason, it was decided to set up the instrumentation during daylight and to complete the test between midnight and dawn when temperature conditions could be expected to be stable. Loading was to be applied by means of trucks loaded with sand to their maximum legal weight of either 20T or 10T. When the loading was planned, it was found to be impossible to fit the full design HA loading on the deck using legally loaded vehicles. Placing the trucks nose-to-tail in each lane resulted in a maximum possible load of 84% of the design value.

4.1 Instrumentation for the test

The instrumentation was selected and placed so as to monitor those aspects of the behaviour of the portal frame that the finite element analysis had suggested might be critical. These were:

4.1.1 Displacement of the beam

Displacement was measured at four points of the beam (gauges P1 to P4 in Fig-

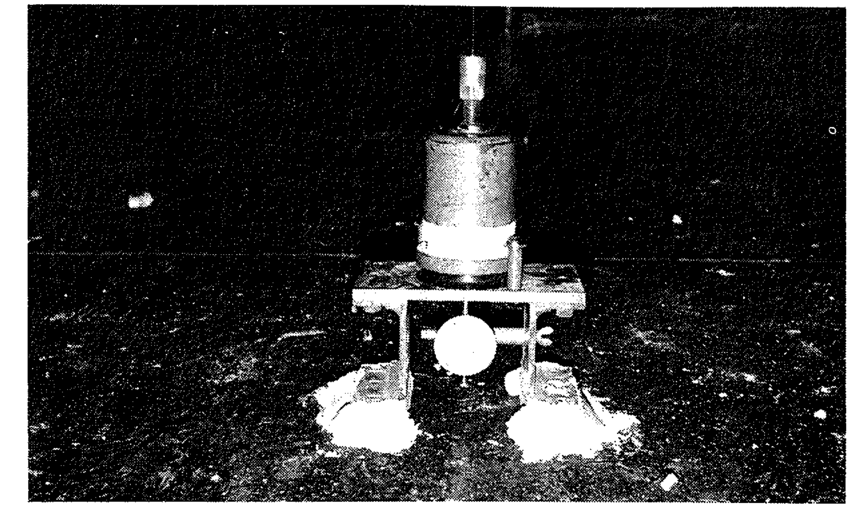


FIGURE 5. Displacement gauge bedded on lower deck of portal.

Figure 6. Main reinforcing bars in column exposed. Strain gauges mounted on second bar from right.

ure 1) by means of the instruments illustrated in Figure 5. An invar wire was attached to the soffit of the beam at the point of measurement by means of a masonry bolt. The lower end of the wire was attached to a heavy weight that was able to slide freely in a vertical direction guided by a ball bushing. The bushing, in turn, was attached to a base that was bedded down on the road surface of the lower deck vertically below the point of measurement. Figure 5 shows the lower extremity of the invar wire attached to the cylindrical weight. The bushing was attached to the base, the channel legs of which were bedded down in the white plaster of paris visible in the photograph. The underside of the weight was attached to a spindle, the vertical movement of which was measured to the nearest 0,01 mm by means of the dial gauge visible in Figure 5. Because of possible displacements of the lower deck, the arrangement of wires and displacement gauges was repeated between the soffit of the lower beam of the portal and the road surface at ground level.

4.1.2 Rotation of the joints

Rotations were measured at points B1 to B6 (Figure 1) by means of bubble slope gauges. Each gauge consists of a sensitive bubble that can be levelled to an accuracy of 50.10^{6} radian and the change in slope measured to the same accuracy. Gauges B1 to B5 all measured the rotation at or near J. If moment continuity at J had been partially or completely lost, B2 and B4 would register different rotations to B1 and B3. Gauge B6 was mounted to measure out of plane rotations of the portal when only one span of the deck was loaded.

4.1.3 Tensile strain in the main reinforcing

Because of the deterioration of the concrete it was considered essential to measure the strain in the tensile reinforcing of the portal. The main reinforcing bars were exposed by chipping away the cover at the points E1 to E6 shown in Figure 1. Temperature compensated pairs of electric resistance strain gauges were then bonded to two exposed steel bars at each location.

Figure 6 shows the main vertical reinforcement exposed in the west column near the beam-column junction J. (Note the deteriorated condition of the concrete around the exposure). Strains were recorded manually via a strain bridge.

Gauges E1, E2 and E3 were placed to measure tensile strains at J. A significant difference between the readings of E3 and E1 and E2 would indicate a loss of moment continuity at J. Gauges E4 and E5 measure bending strains at midspan of the beam and gauges E6 the negative moment at the east end of the beam.

4.1.4 Temperature in the structure

Because of the possibility that temperature movements might be of the same order of magnitude as load-induced movements, it was essential to monitor temperatures in the structure during the test loading. The points selected were points T in Figure 1.

In addition to surface temperatures, it was decided to monitor temperatures within the concrete. A 10 mm diameter hole 500 mm deep was drilled at each of the selected points. Copper-constantan thermocouples were taped to a 10 mm diameter wooden dowel stick so that when the dowel was pushed into the hole thermocouple junctions were located 500 mm, 250 mm and 5 mm from the surface of the concrete. The dowel, being a good insulator, thermally isolated one junction from another and being a tight fit in the drilled hole, prevented movement of air within the hole.

4.1.5 Compressive strains in the concrete

Surface strains on the concrete were measured at selected points shown in Figure 1 by means of a 400 mm gauge length demountable Demec mechanical strain gauge.

Points SA1, SA2, NA1, NA2 were placed to measure midspan compressive strains while points SB1, SB2, NB1, NB2 were to measure compressive strains near the column. Points 1S to 3S and 1N to 3N were to measure diagonal compressive strains near the column.

5. RESULTS OF LOAD TEST

Figure 7 is a general view of the site during the course of the test. The

caravan was used to house the instrument readouts. Figure 8 shows the loaded trucks being arranged on the upper deck.

5.1 Temperatures

The difference in temperature between the three surface thermocouples was a maximum of 3,9°C, but the maximum difference in temperature change between the deeper thermocouples was only 0,7°C. It is therefore considered unlikely that thermal movements would have influenced any of the measured movements.

5.2 Displacements

Comparisons of measured and predicted displacements at point P2 are illustrated in Figure 9. If full continuity at J is assumed, the measured displacements are slightly larger than predicted. A decrease in the elastic modulus of the concrete from 18 GPa to 15 GPa would account for the difference. Results for points P1, P3 and P4 were similar and all indicated that full continuity existed at J.

5.3 Rotations

The results of the rotation measurements at J are compared with predicted rotations in Figure 10. Although the slope gauges were being used at the limit of their accuracy, the results appear consistent and show that full amount continuity exists at J. The measured and predicted results can be reconciled by reducing the modulus of elasticity of the concrete from 18 GPa to 12 GPa - again a perfectly possible value.

5.4 Tensile strains in reinforcing

Figure 11 shows strains measured in the reinforcing at the west beam to column joint (J). As far as continuity at J is concerned, a lack of continuity would be indicated by low or zero strains at E1 and E3 and a difference in strains between E1 and E2. In fact, the readings taken at all three locations cannot be distinguished, although strains are lower than predicted if full continuity applies. Measured strains were also lower than predicted at midspan, thus supporting the conclusion that full continuity exists at J.

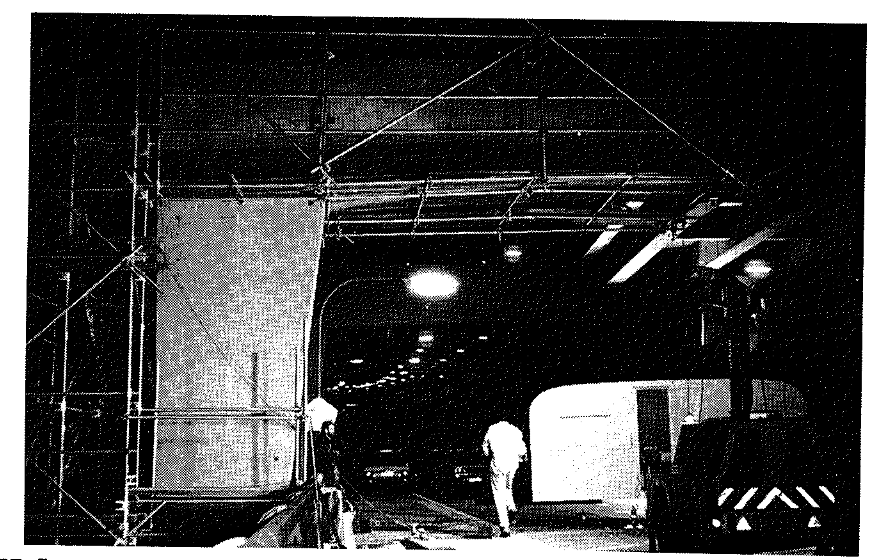


FIGURE 7. General view of site during test.

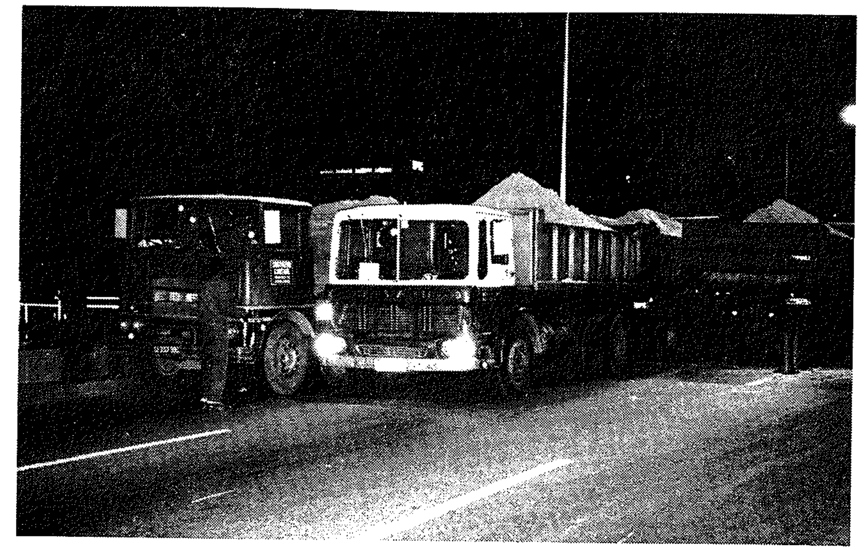


FIGURE 8. Arranging the loaded trucks on the upper deck.

The measured strains cannot easily be reconciled with the predicted values. The predictions were based on the usual straight-line, no tension theory. The most likely explanation of the discrepancy is that although the concrete is very obviously cracked, the cracking does not penetrate to the neutral axes of the beam and column sections and the structure is behaving as if partly cracked.

5.5 Surface compressive strains in concrete

Comparisons of measured surface compressive strains and compressive strains predicted on the basis of the straight line, no tension theory for conditions at midspan are shown in Figure 12.

Of particular note are the alarmingly large strains of about 500.10^{6} that took place during the first loading increment. Strain changes that took place in subsequent load increments were small, however, and very close to predicted values.

The initial strain increment appears to have resulted from a closing up of the surface cracks in the concrete. This view is supported by the observation that there was a large residual strain on unloading. If the initial 'closing-up' strain is discounted, the measured compressive strains are remarkably close to those predicted on the basis of an elastic modulus of 18 GPa for the concrete. Similar results were found at the west column-to-beam joint J.

5.6 Permanent deformation of structure

After applying each load increment a set of measurements was taken and the load was then left in place for 30 minutes before taking a second set of measurements. The second set agreed almost exactly with the first set in all cases. In other words, there was no discernable creep of the structure over the five hour duration of the loading. Figures 9, 10, 11 and 12 all show residual deformations after unloading. However, these are typically 15% to 20% of the maximum which is quite acceptable and to be expected in a reinforced concrete structure.

FIGURE 9. Comparison of measured and predicted displacements.

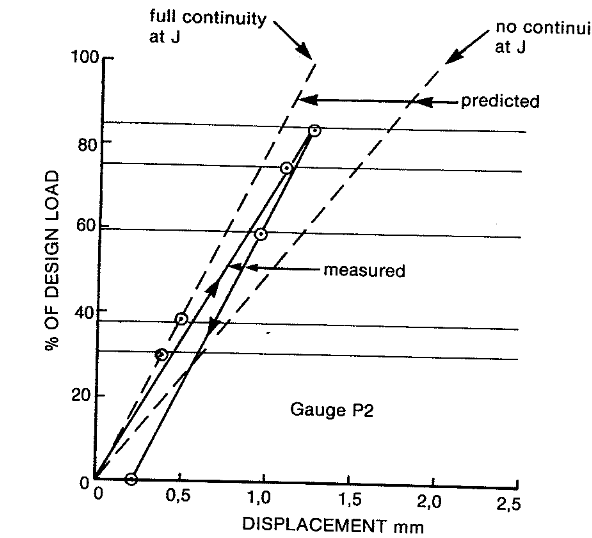
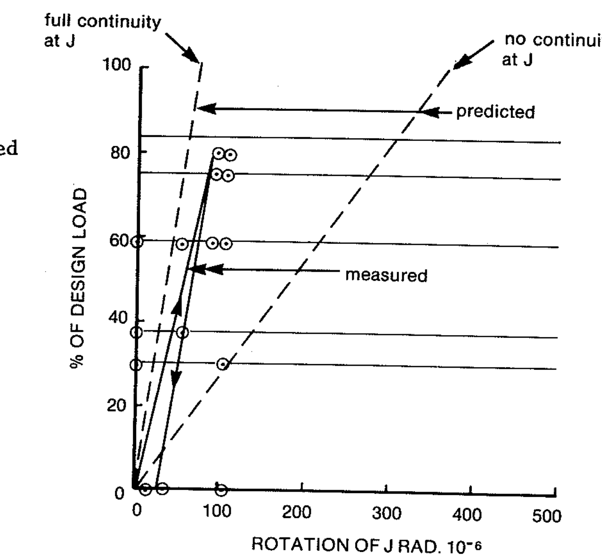


FIGURE 10. Comparison of measured and predicted rotations at point J.



6. CONCLUSIONS

The load test supports the view, formulated earlier on the basis of laboratory tests /1/, that deterioration of concrete as a result of alkali-aggregate reaction is alarming in appearance but not necessarily structurally dangerous. Moreover, structures that have deteriorated behave predictably and their elastic behaviour can be predicted on the basis of laboratory tests on cores taken from the structure.

The major effect of alkali aggregate deterioration appears to be to increase the deformation of a structure by reducing the elastic modulus of the concrete. With normal design practice where the design load usually far exceeds loads actually applied to a structure, safety appears not to be a problem. (With certain classes of structure, silos and bins for example, this conclusion may not be valid).

FIGURE 11. Comparison of measured and predicted tensile strains in reinforcing at J.

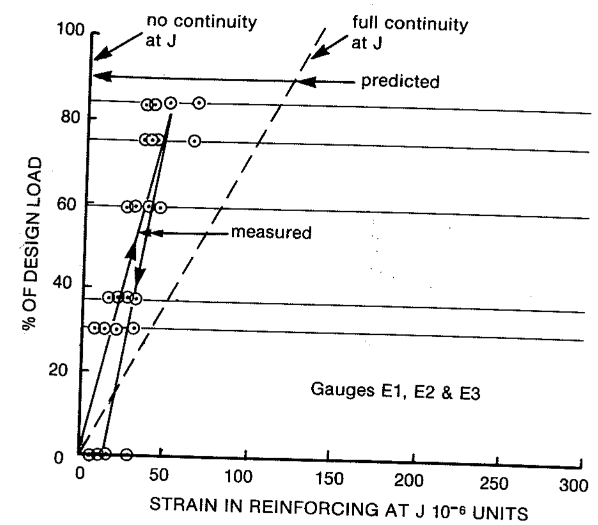
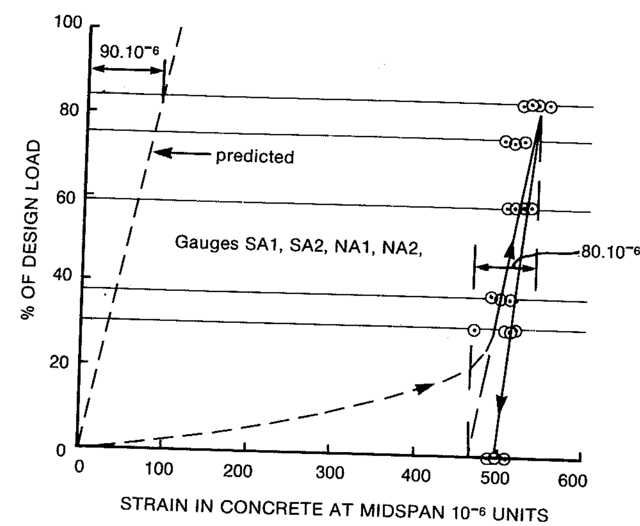


FIGURE 12. Comparison of measured compressive surface strains and predicted compressive strains in concrete.



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SERVICE LIFE PREDICTION AND ALKALI-SILICA REACTIONS

Anders Nielsen
civ.ing.,tekn.dr.

Technological Institute, Building Technology
Taastrup, Denmark*)

ABSTRACTS

Based on Danish experience the hypothesis is set forward that damage due to alkali-silica reactions only occur in structural members in which cracks previously are created during the production. The water penetrates along the cracks in an initiation period. The cracks open in a propagation period. At last a resting period enters, where nothing more happens. - Two cases are treated, one including a moisture analysis.

Keywords: Moisture analysis, service life, bridges, prestressed beams.

1. INTRODUCTION

Experience from Denmark shows that many structures of concrete made with alkali-silica reactive aggregate are sound, although the according to the textbooks ought to have cracks. Often it is seen that one part of a structure is whole the other cracked, or that only some of several obviously identical structural members in a building are cracked.

Further to this it is seen that cracked structural elements may stand for years in the cracked condition without any change of the crack pattern.

In the following I shall connect these observations to the possibility of moisture movements in the structures.

2. THEORY

To declare the above mentioned obviously contradictory observations I will put forward the following hypotheses. Cracking due to alkali-silica reactions (ASR) mainly occurs in structural elements, which have initial cracks from e.g. insufficient mixing, bad vibration, casting joints, bleeding, plastic shrinkage and thermal stresses from wrong curing. These initial cracks have two effects. Firstly the water moves much faster through cracks than through sound concrete. Secondly the cracks have less tensile strength than the solid concrete, and therefore the particles near the crack get the possibility of taking up water and hereby expanding the already existing cracks. This last effect should be seen as an example of the stress dependence of the water absorption capacity of the gel /1/.

From the above mentioned theory the following may be deduced concerning the crack development in an initially cracked structure.

*) From 1983-09-01: Building Materials Laboratory, Technical University, Lyngby, Denmark.