INVESTIGATION AND ASSESSMENT OF THE CONDITION OF THE NEW KELANI BRIDGE

by

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Synopsis

A recent inspection of the New Kelani Bridge by the Road Development Authority had raised doubts about the structural condition of the Bridge and decided to have a thorough investigation carried out on the entire structure and recommendations made for carrying out necessary repairs and strengthening. This was Phase I of the Project.

This was to be followed up with the preparation of designs, bill of quantities, specifications and an estimate for the work - Phase II.

Chandrasena & Partners were commissioned by the Road Development Authority, after obtaining Cabinet approval, to carry out this work.

The paper deals with the work carried out under Phase I.

A detailed close investigation of the structure was carried out and all measurements and other data necessary for the preparation of a bill of quantities were recorded. Typical areas and types of damage were photographed and a hundred of these were selected for presentation with the Report to the Road Development Authority.

The underwater examination of the substructure and piles was carried out by a firm of Divers.

Bridge

The New Kelani Bridge is the largest bridge in the island and the second longest, the longest being Kallady Bridge in Batticaloa.

Tenders were invited by the Public Works Department for the construction of this bridge, on a Turn-Key basis. The construction was carried out by Germon (England) Limited whose tender was the lowest acceptable. The bridge was completed and opened to traffic on the 3rd of February, 1959.

The superstructure of the Bridge is of beam and slab construction in reinforced concrete and has 10 spans varying in length from 70 ft. to 107 ft. and incorporating 3 suspended spans. The suspended spans have bearings of steel plate with a copper plate sandwiched between them. The superstructure is carried on reinforced concrete roller and rocker bearings.

The substructure consists of mass concrete piers and abutments on 13½ in. dia. cast-in situ piles driven to rock. The piles for the piers in the river are driven in cylindrical formation. The maximum design load on the piles is 50 tons.

It was unfortunate that no drawings of the Bridge were available with the Road Development Authority although a complete set of as-built drawings as well as contractor's working drawings were provided to the P.W.D. on completion of the work. These have apparently got misplaced during the various transformations the P.W.D. went through at different times.

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Mr. Chandrasena is a past President of the Institution.
Defects generally found in reinforced concrete structures are:-

a. cracking due to overstressing of reinforcement in tension (due to excessive loading or thermal effects).
b. cracking caused by shrinkage
c. corrosion of reinforcement due to insufficient cover, permeable concrete, exposure to marine atmosphere
d. spalling of concrete due to corrosion of reinforcement

2.0 Procedure

In a detailed investigation of this nature, the normal practice is to examine a significant proportion of the concrete surface. This proportion will vary with the nature of the structure. One composed of repetitive elements may be examined using a smaller proportion than one with diverse elements.

However, the assignment included the preparation of a Bill of Quantities, Specifications and an Estimate for the repairs. Therefore, the investigation was carried out in great detail and the extent of damage assessed as accurately as possible so that quantities for repairs can be computed to a reasonable degree of accuracy to enable a contractor to prepare a fair bid.

It may be mentioned that generally, repair work of this nature is carried out on a "cost-plus" basis as it is not normally possible to determine the quantities of the work to be carried out to the necessary degree of accuracy required for a Bill of Quantities, without actually breaking up entire areas of damaged concrete. This is not practically possible as it may affect even the safety of the structure. The breaking up and repairs have to be carried out in stages.

Close examination of the concrete of the superstructure required access between beams up to 12 ft. deep at the supports and 7 ft. deep at mid span, to reach up to the soffit of the deck slab. The ground under the shore spans was soft with water stagnating. Access was gained by using ladders placed in a trailer of a farm tractor-trailer unit. But this got bogged down under the second span and a wheel loader with large tyres was used with the bucket serving as a working platform. This too got bogged down and a tracked loader had
to be used.

For the river span a specially assembled adjustable telescopic tower carrying a platform was mounted on a timber barge. With this arrangement it was possible to reach the soffit of the deck slab at heights of 18 to 28 ft. above water level.

A fishing boat with an inboard engine was used for towing the barge and for ferrying personnel.

For the purpose of recording damage the bridge structure was divided as follows -

1. Pile Caps
2. Abutments and Piers
3. Bearings
4. Roadway and Kerb Beams
5. Parapet Beams
6. Cross Beams
7. Deck Slab
8. Footways and Ducts
9. Wearing surface
10. Parapets and End Features and
11. Piles

3.0 Investigations - General

Close visual examination of the entire concrete surface of the bridge was carefully carried out followed immediately behind by investigations for honeycombing and weak concrete using scaling hammers. Initially almost every square foot of surface of the concrete was checked for weak concrete and honeycombing in this manner.

It was found that generally the deck slab and the upper three-fourths of the beam ribs which showed no visible signs of defects were quite sound and had no weak concrete or honeycombs. These areas were therefore checked at 1 m to 1 m intervals.

All damage was recorded on forms designed for the purpose, giving type of damage, the exact location, extent of damage and comments and reference to the photographs taken. The locations were also marked with paint. Cracks were examined in detail, the crack width and length recorded and sketches were made. Notes were also made where necessary, to assist in deciding on the type of repairs to be carried out.

Photographic records in colour were made of the damage that had taken place.

Representative sections of honeycombed areas and areas of weak concrete were broken up to determine the depth of the defective concrete to enable a reasonable assessment of the extent of repairs necessary. For obvious reasons it was not prudent to break up all areas of defective concrete.

3.1 Pile Caps

Pile caps of the abutments and the shore span piers are below ground level and these were not examined. The abutments and the shore span piers are founded on piles driven individually (as opposed to the piles driven in cylinder formation as in the river piers).

The shore span piers and the Kandy and abutment showed no signs of damage at the ground level and it was presumed that the pile caps are not damaged. At the Colombo and abutment, however, the vertical construction joint at the middle has opened out to about one inch and the separation was practically of the same width from top to bottom. It appears from the records that this joint did not extend to the pile cap. It is possible that the pile cap has cracked at this point. The opening out of this construction joint and possible cracking of the pile cap would not affect the strength or stability of the abutment.

The pile caps in the river had a horizontal crack at mid-depth running practically through the full length and appearing on both faces. The cracks were of varying thickness up to a maximum of about 1/4 inch.

The piles caps in the river are 3 ft. deep and 9½ ft. wide and of length 70 ft. The original design of the contractors provided 1/2 in. bars at 12 in. c.r.s. each face. They claimed that the pier and pile cap were a monolithic unit and had been designed as a deep beam. But the reinforcement was changed on instructions from the Department to 1 1/8 in. bars at 12 in.c.r.s. on each face with 3/4 in. bars at 18 in.c.r.s. transversely in order to tie up the pile cylinders together. It is recorded in the Paper on the construction of the Bridge, presented at the Annual Sessions of the Institution in 1960, that the concreting of the pile caps had been carried out in horizontal layers. The only
possible cause for the cracks is the differential shrinkage of the concrete and the presumed absence of any vertical reinforcement. (It is not recorded either in the Paper on in any of the drawings that were found with the R.D.A. that any vertical reinforcement was actually used.).

The cracks are not considered as affecting the strength of the pile cap or the pier as the upper half of the pile cap with the reinforcement will act together with the pier as a deep beam. The lower half of the pile cap will also serve as a tie for the pile cylinders. There are no cracks in the piers at pile cap level. The repairs recommended are to wash out the cracks and grout with shrinkage controlled neat cement grout.

3.2 Abutments and Piers

As mentioned above, the construction joint at the middle of the Colombo end abutment has opened out about one inch uniformly from top to ground level.

This joint which is actually a separation or contraction joint had been painted with bitumen from 4 ft. below ground level to the top.

This joint had opened out after the filling at the end features was completed. This was caused by the lateral earth pressure on the retaining walls of the end features, the soil at the site being soft material (the vertical loads are carried by piles). To my recollection, when this occurred ties were provided connecting the retaining walls on either side.

The joint has opened out and the conditions have perhaps stabilised now. If ties were in fact provided, they would have suffered elastic deformation before taking load and caused the opening of the cracks to increase, the ties being about 120 ft. long.

The repairs recommended are to fill the gap with concrete and keep it under observation and remedial action taken if considered necessary. An interesting phenomenon that was observed in the Piers except Pier No. 6 were the cracks that propagate from the top of the Pier cap downwards disappearing after a few feet. Similar cracks were also observed in the bearing beams at these Piers, the cracks propagating from the soffit upwards.

The only possible cause of these cracks would be the differential thermal movement between the superstructure and the Piers which is prevented due to the fact that there is no provision for the lateral thermal movement of the Bridge which has a width of over 80 ft. The differential movement is caused due to the fact that the superstructure which has a larger surface area and smaller mass compared to the pier which has a small surface area and a large mass, reacts more quickly to variations in the ambient temperature than the pier. This movement is prevented by the friction between the rollers and the bearing beam at the top and rollers and the capping beam at the bottom. This results in tensile stresses being set up in the capping beam and compressive stresses in the bearing beam with rise in the ambient temperature and reversal of these conditions with a fall in the ambient temperature.

As the bearing beam as well as the capping beams are very lightly reinforced they cannot take up the tensile stresses. The cracking would have increased from initial fine cracks progressively with time.

In the case of the rocker bearings which are fixed in position by reinforcing bars extending from the bearing beam through the rocker and into the capping beam fixing the bearing to the capping beam firmly, the conditions would have been worse.

In the abutments both of which carry rollers, there were no cracks of this nature. The bearing beams at the abutments too had no such cracks. But lateral movement of the rollers has taken place. Signs of movement are clearly visible.

The pier No. 6 had no cracks of this nature but the bearing beam above had a number of cracks up to 1/8 in. in width propagating from the soffit upwards indicating that the bearing beam here has taken up all the thermal effects. The probable cause of this would be that the bearing beam slipped on the rollers with the movement due to increase in the ambient temperature but for some reason the slippage did not take place when the bearing beam tried to move back when the ambient temperature dropped. The recommendations are to grout the cracks with epoxy grout and provide reinforcement on either side of the capping beams and the bearing beams to withstand stresses caused due to a reasonable possible differential
in temperature between the superstructure and the piers. In the case of the abutments, as no cracks are observed either in the abutment capping or the bearing beam no remedial action appears necessary as sliding has taken place and would probably continue without any harmful effects. However, brass strips can be inserted at the top of the rollers to facilitate movement.

3.3 Bearings

The reinforced concrete roller and rocker bearing were found to be generally in good condition. The damage observed were a small area of concrete in a roller in abutment 1 and pier 3 spalled off and a crack about 6" in a rocker bearing in pier 5. Damage was observed in the bearing beams which transmit the loads from the superstructure on to the bearings, at the contact surface where concrete in the beams has spalled off.

The roller and rocker bearings have been designed with cylindrical contact surfaces of different radii. In this type of design the difference in radii determine the load carrying capacity - smaller the difference the greater the load carrying capacity. In the Kelani Bridge rollers and rockers the difference has been too small and the rollers have been in contact with the edge of the bearing beam due to this, causing spalling of concrete in the bearing beams and in two of the rollers.

The sliding bearings of the suspended spans were found completely seized. As the provision for thermal movement of this 900 ft. long structure has been made at the suspended spans, the Bridge has been converted into one continuous structure with no provision for thermal movement. Variations in temperature will therefore be causing stress conditions for which the structure has not been designed.

Therefore the first and most important item of repair to be carried out will be the release of the sliding bearings and replacing them with more satisfactory bearing which are not likely suffer the same fate. It is recommended that stainless steel plate bearings totally enclosed and permanently grease packed should be used. Elastomeric bearings cannot be used as the space available is only about 11 in. and such bearings require a greater depth to cater for the required thermal movement.

Replacing of the sliding bearings requires the suspended spans to be jacked up at least 1 ft. to 14 ft. This will be a very difficult operation as

a. the movement at each beam has to be same at all times during the operation as otherwise severe stresses can be caused in the cross beams and deck slab
b. jacking has to be carried out from the supporting cantilevers without any change in the loading conditions existing on these cantilevers.

c. the Bridge cannot be closed to traffic as the only other bridge over the Kelani Ganga is the Victoria Bridge and this is closed to heavy traffic on account of its poor condition. However, it may be possible to have the actual jacking up operation which may take about two hours, carried out during the time when the traffic volume is a minimum and the Bridge can be closed for about two hours.

d. the dead weight of a suspended span is over 500 tons.

The existing bearings held in place by 1 in. deep lugs and therefore their removal should not pose a serious problem. The new bearings will be set in epoxy mortar and grouted in epoxy resin keeping the suspended span at the correct level with the help of the jacks until the epoxy mortar and grout have set.

The concrete rollers and rockers have to be eased by cutting into the contact surface to widen the gap between the faces to enable free movement to take place.

3.4 Roadway and Kerb Beams

The most prevalent defects in the Roadway and Kerb beams (main beams) are the honey-combing and weak concrete at the bottom section of the beam rib and the presence of cavities, all due to the congestion of reinforcement which has restricted the flow of concrete. 88 areas of honey-combing were found.

Spalling of concrete and poor concrete were found in 59 areas. The reinforcement in some of these areas was exposed and corrosion had set in. However, the extent of corrosion was small and does not warrant additional reinforcement. There were areas where the bottom of the beam appeared to be good but the surface broke up on the impact of a hammer and cavities were present round the main steel. The surface
of the beam was a thin layer of cement mortar from the concrete. Surprisingly in these areas the reinforcement showed very little or no corrosion.

50 cracks were observed on these beams. Of these 40 propagate from the soffit of the deck slab downwards to about the middle of the rib and others extend downwards but stop well above the soffit of the beams. These cracks have invariably occurred, before the deck slab was cast, due to shrinkage. (the deck slab had been cast several days after the beam ribs).

The other cracks were found to propagate from the soffit upwards. These were very fine cracks and may have developed from hair cracks sometimes found in reinforced concrete members, possibly aggravated by stresses caused due to thermal movement of the structure being prevented.

The honey-combing, weak concrete and the cavities at the bottom of the beams have been caused due to the congestion of steel and the large depth of the rib, making compaction of the concrete very difficult.

The repair work in the areas of poor concrete will be carried out by cutting out in sections, cleaning the reinforcement and re-building the section with high strength shotcrete or collapsed slump high strength concrete. These required properties of the concrete are achieved by the use of suitable admixtures. Where the repair areas are small, polymer modified cementitious mortars will be used. Where the reinforcement is badly corroded and the area is reduced more than 10% additional steel will be provided (the stress in the steel used in the design is 18,000 lbs/sq.in.).

The cracks will be grouted with epoxy grout.

3.5 Parapet Beams

The Parapet Beams were found to be the worst affected. These beams are only 12 ins. wide but with depths up to 12 ft. and with heavy reinforcement, all of which would have made it very difficult to ensure a well compacted concrete at the bottom section. This has resulted in severe honey-combing and spalling of concrete in this area causing heavy corrosion of steel.

Numerous cracks were observed in these beams, the worst being at the ends of the suspended spans and the supporting cantilevers. The conditions have been aggravated due to these beams being fully exposed direct to the sun on full face of the beams, the orientation of the Bridge being practically North-South, and also fully exposed to rain.

The damage on the downstream parapet beams was much worse than that in the upstream beam, perhaps due to the effects of the afternoon sun being stronger and of longer duration. Cracks were the most prevalent damage in the parapet beams where 192 cracks were found in the 20 beam units. Of these 85 do not extend through to the beam flange or to the beam soffit. These cracks are most probably shrinkage cracks. 69 of the cracks are tension cracks which would have, in all probability, been hair cracks initially. These may have been aggravated due to thermal movement being prevented. Some may even have been caused due to the same reason. 38 cracks cannot be explained as due to any other cause than to thermal movement being prevented.

92 areas of honey-combing, 30 areas of weak concrete and 19 areas of spalled concrete were found. Reinforcement corrosion was found in 61 areas some of which were severe. Some bars have lost 50% of the area. The worst damage suffered by the parapet beams is at the ends of the cantilevers supporting the suspended spans. On the downstream side the end of two of the cantilevers has virtually collapsed, the concrete being shattered due to severe corrosion of the reinforcement and the bearing has actually slipped down and the end of the suspended spans has dropped down about 1 in. The suspended beams are resting wedged against the cantilevers. The reinforcement at the ends of the cantilevers shows a large reduction in section. This reinforcement has to be cut out and new reinforcement welded as there is no possibility of using additional steel due to the heavy congestion of reinforcement.

On account of the nature and extent of the damage in the parapet beams it was decided to reduce the loading on them by using prestressed concrete slabs which will be of smaller thickness, for the footway slabs which are now in reinforced concrete and which are to be replaced as most of them are damaged. The hand rails too are to be replaced as a large number of them are damaged. They will be replaced with lighter prestressed concrete rails.
The repairs to the parapet beams will be carried out in the same manner as in the case of the roadway beams except at the ends of the suspended spans and the ends of the cantilevers supporting the suspended span beams. Here the entire cracked section will be demolished and recast with badly corroded reinforcement cut out and new sections welded, using high strength flowing concrete of low water:cement ratio and giving a high early strength.

3.6 Gross Beam including Bearing Beams

117 cracks, 91 areas of honey-combing, 42 areas where reinforcement is corroded, 23 areas of spalled concrete and 34 areas of weak concrete were identified.

6 of the cracks appear to be due to prevention of thermal movement and/or shrinkage. Majority of the other 111 cracks propagate from the soffit of the deck slab downwards. These cracks have invariably been caused by shrinkage of the beam rib prior to casting of the deck slab, which had been cast several days later. The other cracks are fine cracks confined to the middle of the beam rib and are, in all probability, surface cracks caused by shrinkage. These do not penetrate through the rib.

Repairs are to be carried out in the same manner as for the roadway, kerb and parapet beams.

3.7 Deck Slab

The deck slab was found to be in very good condition compared to the rest of the structure. Damage was formed only in three of the ten spans. Spalling of concrete was found in 7 areas, corrosion of reinforcement due to insufficient cover in 11 areas and honey-combing in 3 areas. Two transverse construction joints have opened out and seepage of water was observed.

Repairs are to be carried out as in the other sections. The construction joints which have opened out will be cleaned out, grouted and sealed at the top.

3.8 Footways and Ducts

A large number of the precast footway slabs were found to be damaged and some completely broken. The damage has been caused by heavy vehicles belonging to the armed services, which were found parked on the footways at various times during the emergency. The footways had not been designed for a wheel load in the event of a vehicle mounting the kerb. The designers had claimed that the kerb being 9 ins. high would generally prevent a vehicle mounting it. But the height of the kerb has been considerably reduce by successive layers of road surfacing.

All the footway slabs are to be replaced with lighter prestressed concrete slabs which will reduce the load on the parapet beams.

A crash barrier will be installed at the kerb to prevent vehicles getting on to the footways.

The ducts which are under the footways have slender beams spanning the parapet and kerb beams at approximately 8 ft. centres. These beams have been designed to carry two 30 in water mains. They also support precast duct floor slabs which have been designed for incidental loading.

Some of the slabs were found to be damaged due to corrosion of reinforcement but the others appeared to be in fair condition. It was not possible to examine them carefully for damage as they were covered with dirt and debris. About 50% of the duct beams were found to be damaged due to corrosion of the reinforcement. These beams have been precast and built in. The corrosion of the steel has been caused mainly due to insufficient cover. In some beams the area of the reinforcement has been reduced by about 50%. The damage was found to be more in the downstream duct.

The duct slabs will be removed, washed down and examined.

All the duct beams will be demolished, additional reinforcement will be provided where necessary and the beams re-cast in high strength concrete. The duct slabs which are found to be damaged will be replaced. If a large number are found to be damaged the entire lot will be replaced with lighter prestressed concrete slabs.

3.9 Wearing Surface and Expansion Joints

The original wearing surface on the Bridge was to be 3 in. reinforced concrete. The Department changed the thickness to 2 3/4 in. with a 1 in. surfacing of bituminous concrete. Subsequent
layers bituminous concrete have been added and the thickness was found to vary from 2 in. to 5 in. and the surface as it stands is uneven. The result is that sharp vibration are felt on the bridge. As the vibrations would be harmful to the repair work the surface will be levelled before any repair work is commenced. The entire surface will be stripped after all repair work is completed and a new surface of bituminous concrete 1 in. thick will be provided. The expansion joints have all been found to be damaged. All the joints will be replaced with new prefabricated joints.

3.10 Parapets and End Features

The original design provided for a light precast railing with vertical members. But the Director of Public Works at the time on the advice of the Chief Architect decided on the present design which is much heavier. On account of this additional steel was required in the narrow parapet beams where the steel was already congested.

Many of the rails, which are precast, were found damaged due to corrosion of the reinforcement. It has therefore been decided to replace all the rails with prestressed concrete rails which will be lighter.

The end features were found to be damaged and in a neglected condition. These will be restored to the original condition.

3.11 Piles

An under-water examination of the piles was carried out by a firm of Divers. The piles were scanned using a television camera and watched from a boat through closed circuit television, a recording also being made simultaneously. Instructions were given to the Diver through two way communication. The first investigation was not successful due to the water being turbid and a second attempt was made recently which proved to be much better.

The damage observed was in a few piles only where there was a gap in the concrete, exposing the reinforcement. It is recorded that the reinforcement cage in some piles got lifted up during extraction of the pile tube indicating a separation in the concrete. When this happened replacement piles had been driven inside the cylinder formation. Only three piles in one pier and two in another showed such damage confirmed by the Diver in his commentary. This is a negligible percentage as the number of piles for a pier was over 100. Actually it is recorded that all the piers in the river, which were in cylinder formation had several replacement piles.

4.0 Tests carried out on the Concrete

Non-destructive tests were carried out on sections of the structure selected at random, by the Research and Development Division of the Road Development Authority using a Schmidt Hammer. They gave results varying from 6,000 lbs/sq.in. to 8,000 lbs/sq.in. The strength required in the design is 3,000 lbs/sq.in. Sample cores of concrete were cut, two from the parapet beams and one from the deck slab. Tests on these core gave strengths of 5,075 lbs/sq.in. to 6,670 lbs/sq.in. Density of the cores was found to be 150 lbs/cu.ft.

Tests were carried out at the site on freshly cut concrete on the downstream parapet beams which have been worst affected, for carbonation and the results indicated that hardly any carbonation had taken place and that the alkaline barrier to corrosion was fully available.

5.0 Order of Repairs

Prior to commencement of repair work the unevenness of the surfacing on the Bridge will be corrected so as to minimise vibration of the Bridge.

The second step will be the replacement of the sliding bearings followed up by the release of the roller and rocker bearings. This is essential in order to relieve the structure of the internal stresses caused due to thermal movement being prevented. Movement of the bridge during these operations will be closely monitored. For the purpose inclinometers will be attached to two outer rollers at each pier and abutment.

The other repairs can then be carried out as approved by the Engineer.

6.0 Comment

The advantages claimed for a reinforced concrete structure are, amongst others, that it requires no maintenance and lasts longer than steel. It would therefore be a matter of surprise that the New Kalani Bridge has been found to require fairly extensive repairs after
a comparatively short period of 30 years.

The main cause of this situation can be attributed to the method by which it was decided to have this bridge constructed - viz., by inviting tenders on a design and construct or turn-key basis. This had resulted in the tenderers cutting down their designs to provide the barest minimum in every aspect. The dimensions of the various elements have been selected to cut down quantities irrespective of conditions. Durability of the structure has not been given sufficient consideration. The contractor's designers in their anxiety to produce a very economical design, have not given sufficient thought or consideration to the practical problems involved in construction, as for example the placing an compaction of concrete in 12 in. wide beams up to 12 ft. in depth with congested reinforcement at the bottom. Cover on the parapet beams fully exposed to weather and to some extent marine atmosphere has been the minimum given in the code for R.C. structure at the time.

The sliding bearings which have seized due to corrosion, mainly as a result of water leaking on to them through the expansion joints, will be responsible for a major portion of the repair costs.

Attention has not been given to the lateral thermal movement of this 80 ft. wide bridge.

Had the Bridge been designed in accordance with the current Code of Practice for R.C. Structures the damage would be far less.

7.0 Acknowledgements

The author wishes to place on record the valuable contribution Mr. Rasaratnam, who was mainly responsible for preparing the Paper on "The New Kelani Bridge", has made in presenting a very detailed record of the construction of the New Kelani Bridge from the tender stage to completion, which was of immense value in the investigations and recommendations for repair of the Bridge.

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