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REVIEW OF THE PRESENT CONDITION OF THE LUCINDA BULK SUGAR TERMINAL AT LUCINDA IN NORTH QUEENSLAND AUSTRALIA

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ABSTRACT

This paper describes the present condition of the 5.76 km long Lucinda Bulk Sugar Terminal commissioned at Lucinda in North Queensland in 1978. The substructure of this facility consists of steel piles cathodically protected in the tidal zone and coated above water. The superstructure consists of a single lane roadway made out of prestressed concrete beams and a conveyor for the sugar made of coated steel and aluminium components. The roadway is only used to transport workmen to the ship loading facility off shore and is not used to transport sugar. This structure is currently 25 years old in 2003 and was first detected to be suffering alkali-silica reaction distress in 1982. Since 1987 significant information has been collected from this structure under a scheme initially devised by Gunnar Idorn of Denmark. The information collected on an annual basis consisted of transverse and longitudinal expansion data, concrete surface potentials, chloride ion profiles in the cover concrete and selected load testing of several spans of the roadway structure. The components suffering ASR distress were determined to be the prestressed roadway beams over the full length of the structure and the cast in place headstocks in the tail end structure. This structure consists of a tail end section of approximately 60 metres which ramps down to the high water mark and the balance of the structure over 5.5 km in length has at least 4 metres clearance above high water. It is concluded from this monitoring that the height above sea water and the early protection of the top surface of the roadway beams with a polymer modified sealer are two of the main factors reducing the rate of degradation of the beams. In addition, the load testing of the roadway spans indicates the structure is performing similar to an uncracked structure at this stage.

Keywords: Alkali, aggregate, concrete, testing, structures

1 INTRODUCTION

The main purpose of this paper is to confirm the Lucinda Jetty structure is currently in a very sound condition and fully functional in relation to its original design intent. Even though the roadway beams are significantly cracked due to alkali-silica reaction expansion, they continue to easily carry their design loads and will continue to do so for many years to come [1]. Fig. 1 shows a general view of the structure consisting of 5.76 km long roadway and adjacent conveyor and the off shore sugar loading facility.

Since 1987, this structure has been monitored to plot its rate of deterioration due to ASR and ensure it remains in a safe condition to carry out normal operations. The information collected has also been used to optimise the ongoing maintenance strategies. Fig. 2 shows a general view of the Tail End Structure which accepts the sugar from the 3 sheds for transport along the conveyor. This part of the structure has the lowest clearance above the high water mark and hence is the location where significant corrosion has occurred due to the ASR cracking in the jetty beams. It is fortunate that the Tail End Structure is only 60 metres in length which

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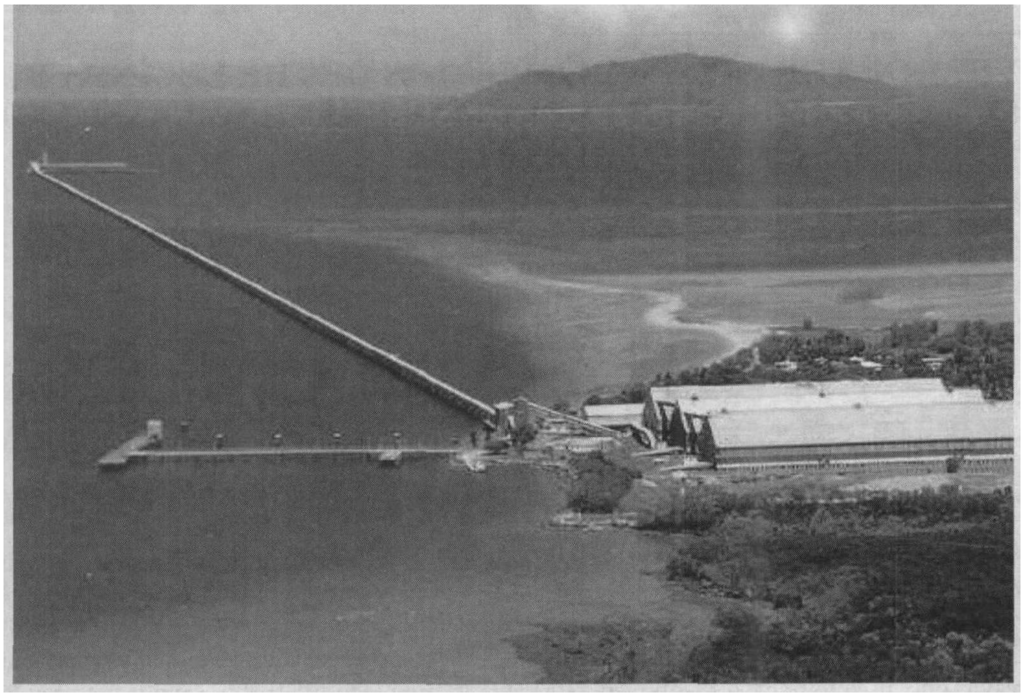

Fig. 1: General view of Lucinda Jetty structure

represents 1% of the Jetty length. Hence, 99% of the Jetty is well above the high water mark, which significantly reduces the risk of chloride ion penetration into the soffits of the beams.

In 1996, delegates to the 10th ICAAR in Melbourne were able to select a site visit to Lucinda Jetty as one of their options. About 30 delegates inspected this Jetty and hence this paper will bring them and the rest of the Industry up to date with the present condition of this structure in 2003. A petrographic analysis of the extrusive volcanic aggregates used on this project has previously been described [2]. Shortly after the Lucinda Jetty was determined to be suffering ASR distress the Queensland Department of Main Roads detected approximately 100 road bridges also suffering ASR distress [3].

2 REVIEW OF MONITORING DATA

2.1 Expansion Measurements

2.1.1 Longitudinal Expansion Data

Table 1 and Fig. 3 detail the longitudinal measurements taken by the author over a representative sample of expansion joint gaps. It can be seen that the mean expansion joint gap of 77 mm varies by -4 and +3 mm over the 15 year measuring period. If alkali-silica reaction (ASR) was causing any significant longitudinal expansion of the prestressed beams then the expansion joints should be reducing in width. The data in Table 2 shows this is not the case and hence it can be concluded that ASR is not causing any significant longitudinal expansion of the beams.

2.1.2 Transverse Expansion Data

Table 2 contains a summary of the transverse expansions measured on the beam soffits at a selection of cross over locations. Typically 12 gauge lengths have been measured at each cross over location and the minimum, average and maximum results (in microstrain) are given in Table 2. The period of measurement is approximately 14 years from June 1986 to August 2000.

Examination of the results in Table 2 shows there is a gradual increase in the maximum expansion at most locations. The maximum expansion at Pier 62, in August 2000 was 4541 microstrain compared with 4795 microstrain in August 2002 representing an increase of 254 microstrain in 2 years ie a 6% increase. The total expansion over the monitoring period of about 0.48% is very significant.

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Fig. 2 View of Tail End Structure showing proximity to high water mark

Table 1 Comparison of Expansion Joint Gaps

Pier No.	Expansion Joint Gaps (mm)				
	1985	1990	1995	2000	2002
31L	74	70	70	79	73
67L	102	97	102	100	100
91R	84	92	95	90	89
115R	80	82	96	92	88
173R	62	52	55	55	60
181L	80	72	85	85	83
283R	50	45	55	48	48
Mean	76	73	80	78	77

Table 2 Transverse Expansions Measured on Beam Soffits During the Period 6/86 to 8/2002

Pier No.	Expansion (μ)		
	Minimum	Average	Maximum
17	+442	+2095	+4667
62	+12	+2363	+4795
107	-48	+1516	+3384
152	+329	+1846	+4249
194	+380	+1819	+3996
272	+41	+1459	+2929

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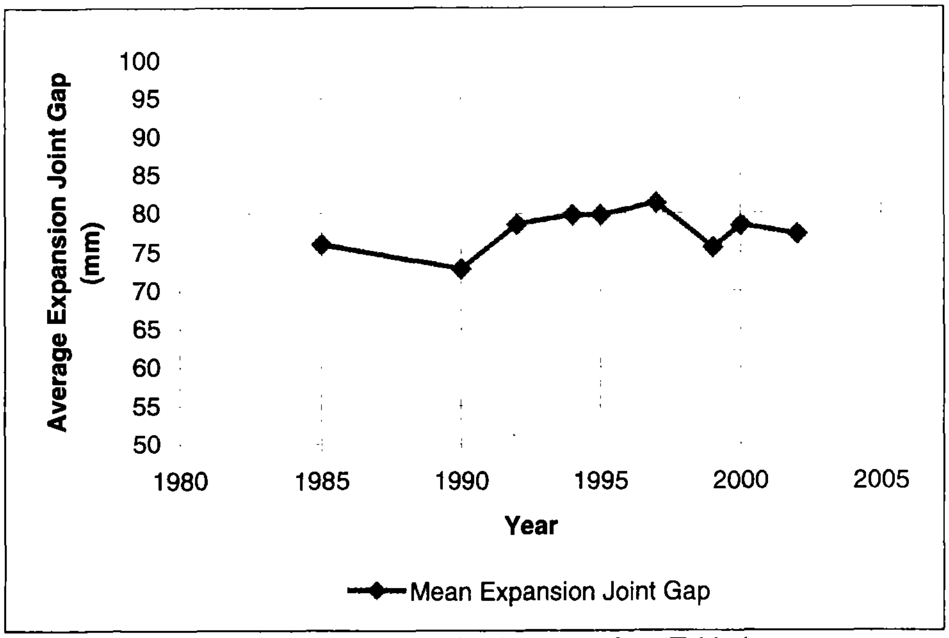

Fig. 3 Average expansion joint gaps from Table 1

Table 3 Efficiency of Sealers Measured by Reduction in Transverse Expansion on Top of Beams

Location (Span)	Sealer	Average Expansion Unsealed (μ)	Average Expansion Sealed (μ)	Sealed/ Unsealed
78	Dynasylan, No Bit	2522	2780	1.10
79	Dynasylan, Bit	1754	1210	0.69
102	Dynasylan BSM40, No Bit	1964	1275	0.65
103	Dynasylan BSM40, Bit	1177	627	0.53
126	Hydrozo, No Bit	2609	2426	0.93
127	Hydrozo, Bit	809	637	0.79

Table 4 Current Maximum Transverse Expansions in Sealer Trial Area on Top of Beams

Span	Bitumen Removed	Expansion (μ)		
		Unsealed	Sealed	Sealed/ Unsealed
78	Yes	6742	4471	0.66
79	No	3643	5032	1.38
102	Yes	4533	3223	0.71
103	No	5183	3724	0.72
126	Yes	6180	3647	0.59
127	No	2726	2407	0.88

2.1.3 Concrete Sealer Trials

Table 3 contains a summary of the average transverse expansions measured in selected spans to check the efficiency of various sealers in reducing the ASR expansion. The period of measurement is from December 1986 to August 2002, i.e. 16 years. From the data in Table 3 the average ratio of sealed to unsealed transverse expansion is 0.78. The average ratio for the bitumen zones is 0.67 and the bitumen removed zones is 0.89. This indicates the

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presence of the bitumen is twice as effective as the sealers on their own. Table 4 contains a summary of the maximum transverse expansions measured in the sealer trial zone. The maximum transverse expansion measured in this set of tests was 6742 microstrain in span 78, 12.2 m from the east end (no bitumen or sealer present).

Fig. 4 graphically shows the minimum, average and maximum expansion that has occurred in Span 78. The maximum expansion of 6742 microstrain occurred at a duration of 191 months since the installation of the measuring system in December 1986. Since September 1999 this maximum expansion has remained static indicating the overall potential of the concrete to expand due to ASR has probably been reached. This level of expansion is consistent with that measured on equivalent Main Roads structures over estuarine creeks.

2.2 Chloride Ion Contents

The chloride ion contents continue to provide critical information concerning the change in condition of this structure. Previous reports have shown the chloride ion levels to be ten times higher in the tail end structure beams compared with the roadway beam soffits. Fig. 5 shows a summary of the average chloride ion contents in spans 2 and 3 of the tail end structure and the roadway beams in 2002. The maximum chloride ion contents in the roadway beam soffits has also been included for comparison. The chloride ion threshold concentration for corrosion would be approximately 2 kg/m^3 for the beam concrete. Hence, Fig. 5 predicts corrosion is probably active in the tail end beams (which has been proven by physical inspection and repair) and not yet started in the roadway beams (also proven by physical inspection).

Examination of the chloride ion profiles in the soffits of the tail end beams shows they are increasing with time due to the level of ASR cracking. This is supported by the level of repair activity in this area. Since most of the structure is represented by the roadway beams the rate of increase of chloride ions in this zone is critical to the life cycle performance of the structure.

2.3 Concrete Surface Potential Results

A comparison of results in 1987 when the sealers were applied and the September 2002 results is recorded in Table 5 (measurement period 15 years). Examination of Fig. 6 which shows a plot of the half cell potentials from a typical beam indicates that the sealed and unsealed sections are performing in a similar manner. Since the last report in 2000 the results have remained static. The current potential readings are similar to the initial values in 1987.

Table 6 contains a summary of the minimum half cell potentials measured on the beam soffits in the tail end structure. The data shows the control (untreated) beams and sealed beams are behaving in a similar manner in Span 3 and in general exhibiting a positive shift in potential. In Span 2 the potentials have on average shifted in a negative direction. In Span 2 the sealers have in general reduced the amount of negative potential shift compared to the control sections. However the control beams are in a more exposed location which may result in an overestimate of the effectiveness of the sealers used on beams in a less aggressive area.

Table 7 contains a summary of the concrete surface potentials measured on the top of the roadway beams. From Table 7 it can be seen that most control sections are performing as well as the sealed sections.

2.4 Load Testing of Roadway Beams

Table 8 contains a summary of the load test results for 1993 and 2002. Table 9 identifies the predicted results for deflection, elastic modulus and stiffness assuming different average concrete compressive strengths for the roadway beams. Examination of the data in Table 8 shows all values lie within the predicted range of values given in Table 9. The maximum recorded deflection is 4.25 mm measured in Span 163 indicating an equivalent average beam compressive strength of 75 MPa which is satisfactory given the obvious level of cracking in the beams.

Spans 161, 163 and 205 are all showing a relatively constant stiffness, for measurements taken between 1993 and 2002 as recorded in Fig. 7.

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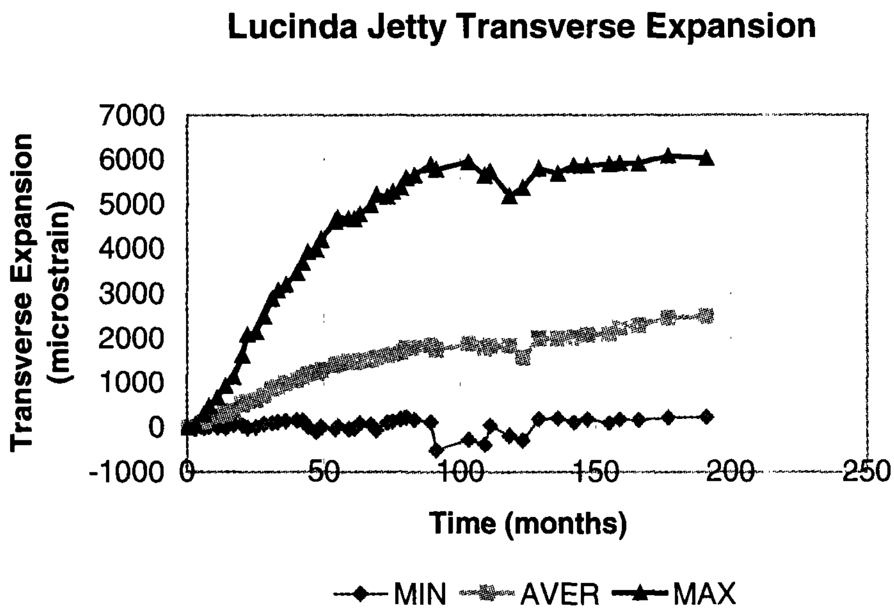

Fig. 4 Span 78 (no bitumen, no sealer) December 86 to September 2002
Maximum transverse expansion location

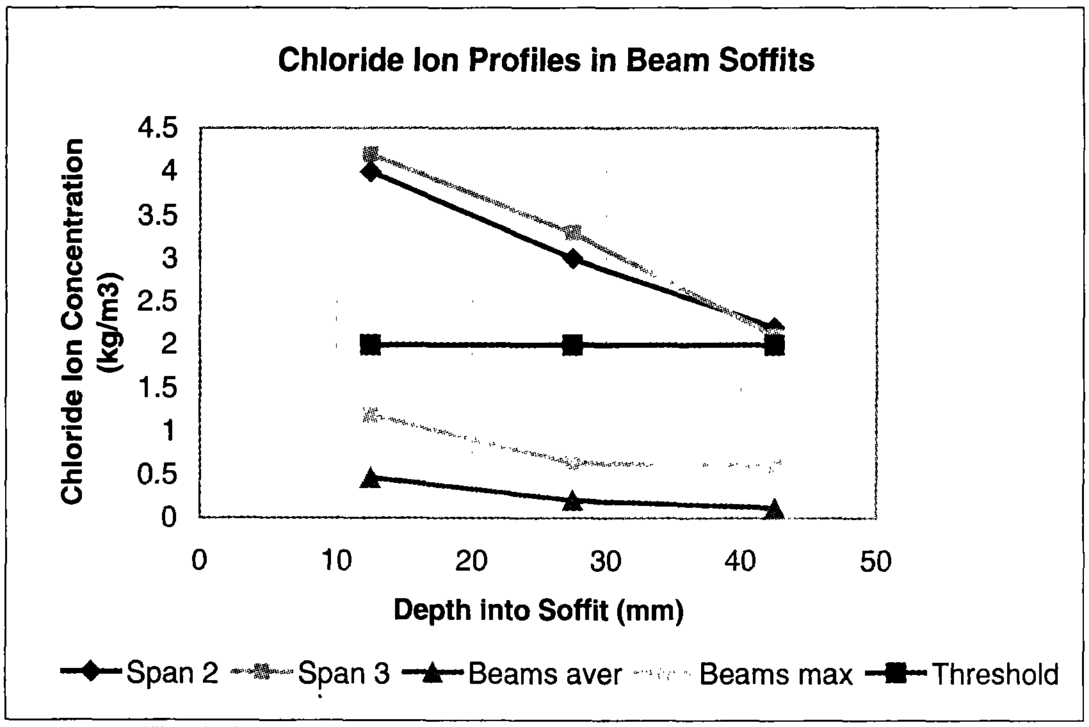

Fig. 5 Summary of the Chloride Ion Profiles in the Beam Soffits in 2002

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Table 5 Average Concrete Surface Potentials (mV) on Roadway Beam Soffits

Spans	Concrete Surface Potential (mV)					
	1987		1995		2002	
	Unsealed	Sealed	Unsealed	Sealed	Unsealed	Sealed
78 - 127	- 151	- 172	- 145	- 143	-145	-139
161 - 205	- 194	-	- 127	-	-138	-

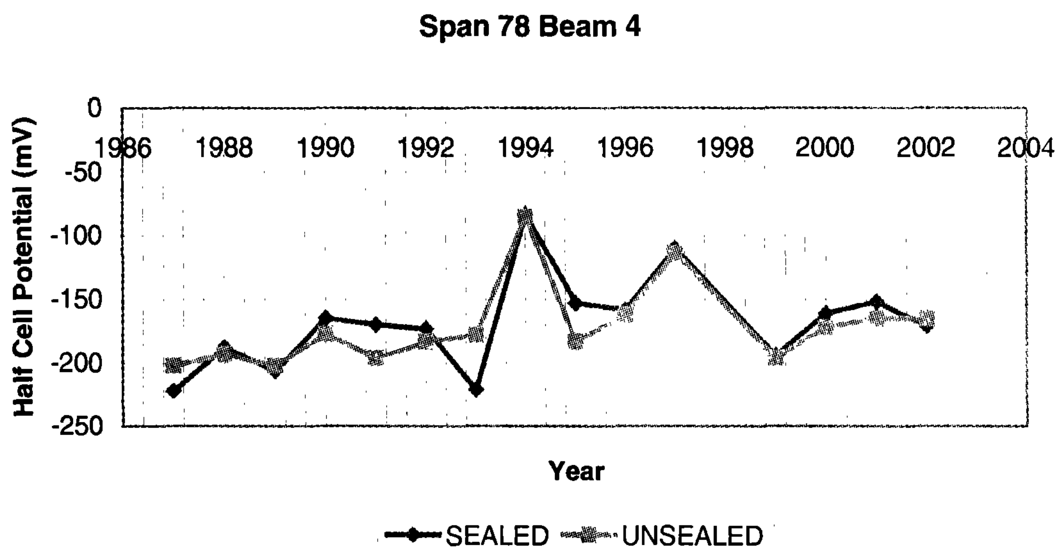

Fig. 6 Half Cell Potentials at a typical location in Span 78 Beam 4

Table 6 Minimum Concrete Surface Potentials (mV) Measured on Beam Soffits in the Tail End Structure

Span	Beam	Sealer	HCR (mV)					
			1989		1995		August 2002	
			North	South	North	South	North	South
3	B26	Control	- 266	- 283	-318	-391	-193	-196
3	B29	Control	- 193	- 195	-204	-233	-149	-137
3	B32	Control	- 360	- 335	-370	-371	-374	-350
3	B35	Control	- 268	- 252	-290	-279	-226	-235
3	B38	Control	- 579	- 494	-622	-548	-463	-494

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Table 7 Summary of Concrete Surface Potentials on Top of Roadway Beams

Span	Bitumen	Sealer	Average Concrete Surface Potential (mV)		
			January 1987	May 1994	August 2002
78	No	Control	- 198	- 233	-144
	No	Dynasylan BH-BSM	- 137	- 183	-166
79	Yes	Control	- 209	- 210	-176
	Yes	Dynasylan BH-BSM	- 224	- 260	-207
102	No	Control	- 210	- 248	-203
	No	Dynasylan BH-BSM 40	- 191	- 296	-222
103	Yes	Control	- 195	- 162	-92
	Yes	Dynasylan BH-BSM 40	- 235	- 170	-69
126	No	Control	- 137	- 227	-183
	No	Hydrozo	- 207	- 272	-164
127	Yes	Control	- 202	- 226	-197
	Yes	Hydrozo	- 234	- 216	-152

Table 8 Summary of Load Test Data for 1993 and 2002

Span	1993			2002		
	Deflection (mm)	Elastic Modulus (MPa)	Stiffness EI ($Mn.m^2$)	Deflection (mm)	Elastic Modulus (MPa)	Stiffness EI ($Mn.m^2$)
161	3.8	48500	3880	3.54	52350	4190
163	4.8	38500	3080	4.0	46350	3700
205	4.1	45000	3600	4.25	43600	3500

Table 9 Predicted Structure Response for varying Average Compressive Strengths

Average Beam Compressive Strength (MPa)	Deflection (mm)	Elastic Modulus (MPa) based on Pauw's Formula	Stiffness EI ($Mn.m^2$)
54 *	5.0	37150	2975
75	4.2	43800	3500
100	3.6	50500	4040

* Predicted strength at completion of Project

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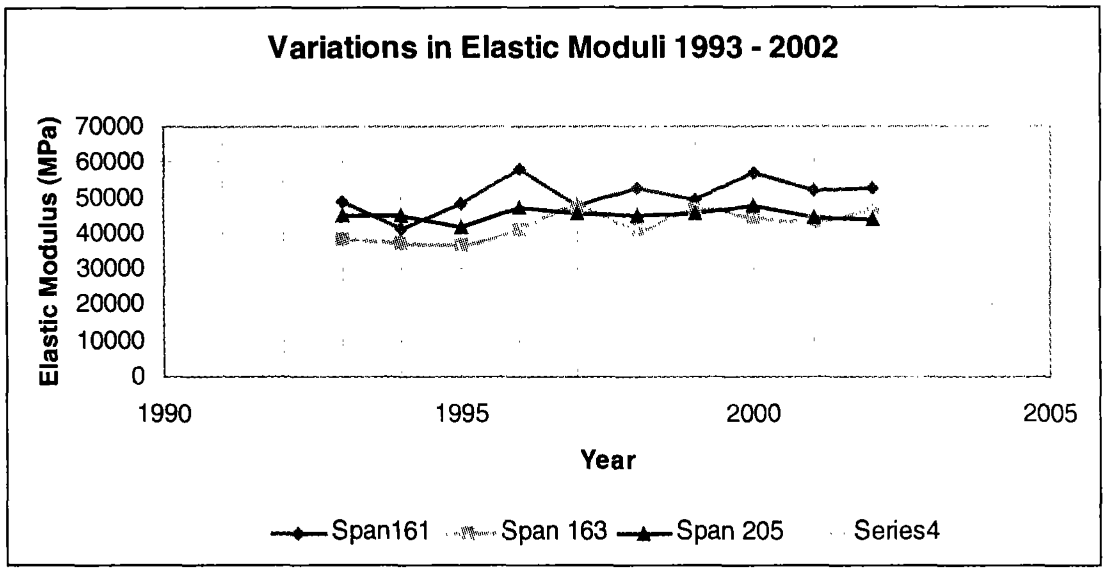

Fig. 7 Variation in Elastic Moduli

3 CONCLUSIONS

The review of the present condition of the Lucinda Jetty Structure in 2002 has identified the importance of collecting the data discussed in this paper at an annual frequency of measurement. The following conclusions are made based on the information collected to date:

3.1 Longitudinal Expansion

The structure is stable in the longitudinal direction with no measurable expansion due to ASR occurring in this direction. As shown in Table 2 the mean expansion joint gap measured in November 2002 was 77mm compared to 76mm in 1985. Hence the mean value measured over 17 years has essentially remained unchanged. This is in accordance with the nature of alkali-silica reaction distress in that the main concrete expansion will occur in the directions of least resistance which for the Lucinda Jetty beams are the transverse and vertical directions. Hence the emphasis on strain measurements in the transverse direction as detailed below.

3.2 Transverse Expansion

The transverse expansions measured at the cross over locations (see Table 3) are showing a steady increase in magnitude since 2002. The maximum expansion recorded in these zones was 4795 microstrain compared to 5195 microstrain at Pier No. 17 in 2000. In the sealer trial area the overall maximum expansion occurred in Span 78 - 12.2 m from the East (No Bitumen, No Sealer Zone) of 6742 microstrain. Fig. 2 shows the typical expansion at 16.7 m from the East appears to have peaked at approximately 6000 microstrain at a test measurement age of 103 months. However, further testing is required to confirm this conclusion.

From the data in Table 4 the average ratio of sealed to unsealed transverse expansion is 0.78. The average ratio for the bitumen zones is 0.67 and the bitumen removed zones is 0.89. This indicates the presence of the bitumen is twice as effective as the sealers on their own.

3.3 Chloride Ion Contents

The chloride ion content of the beam soffits in the Tail End Structure is increasing with time due to the level of ASR cracking (see Fig. 5). This result is supported by the level of repair activity required in this area. The chloride ion content of the roadway beam soffits in 2002 has reduced in relation to the 2000 values. Since most of the length of this structure is represented by the roadway beams the rate of increase of chloride ions in the beam soffits is critical to the life cycle performance of this structure. The chloride ion measurements taken in

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1993, 2000 and 2002 have confirmed the long term trend for the concentration of chloride ions to increase in the roadway beam soffits. However, in the short term 2000 to 2002 they have reduced which is very significant in deciding whether any intervention action is required and the timing of this action.

3.4 Concrete Surface Potential Results

3.4.1 Sealer Trial Zone Beam Soffits

All concrete surfaces are behaving in a similar manner whether sealed or unsealed, bitumen or no bitumen. (See Fig. 5)

3.4.2 Tail End Structure

The results indicate corrosion is active as evidenced by the level of repair activity required in this zone.

3.4.3 Top of Roadway beams in Sealer Trial Zone

The control sections are performing as well as the sealed sections and the results indicate a low risk of corrosion being active.

3.5 Load Testing

All tested spans indicate a normal response was obtained with no significant change in long term stiffness.

REFERENCES

[1] Carse, A.,. The asset management of alkali-silica reaction in the Queensland road bridge network an owner's viewpoint, Proc 11 Th Int. Conf on AAR in concrete, Quebec, Canada, 2000, pp 1119–1128

[2] Thaulow, N., Holm, J. and Anderson K T., Petrographic examination and chemical analysis of the Lucinda Jetty prestressed concrete roadway, Proc. 8 th Int. Conf. on AAR, Kyoto, Japan, 1989, pp 573-581

[3] Carse, A., Alkali-silica reaction in concrete structures, Univ. of Qld Res Rep., 1988, No. CE88, pp 1-24

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