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**COLUMN TENDON VIBRATIONAL TEST
PREPARATION AND DATA ANALYSIS
Contract No. BC353, RPWO#23**

Final Report to Florida Department of Transportation

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16. Abstract Vibrational data from the vertical external tendons in piers of the high level approaches of the Sunshine Skyway Bridge were analyzed to obtain tendon tension estimates. Testing was practicable for virtually all the tendon segments in 68 of the high level approach piers of the bridge. Estimated tensions for the external portions of all but one of the tendon segments evaluated approached values expected from typical design conditions. These observations are consistent with independent direct inspection of selected piers that revealed only minor to moderate corrosion of the external tendon portions, and only one instance of severe corrosion at a tendon recess in the thick wall region, far from the external portion. Dramatic tension loss was measured only for the NE tendon segment in Pier 133 NB. Upon autopsy, that part of tendon was found to have experienced severe corrosion damage and breakage of most strands in the upper internal portion. Tension at tendon segments 133 NB SW and 133 NB NW were near normal, even though severe detensioning had taken place at the opposite ends of both tendons. This observation indicates effective force transfer development in the lower, hairpin bend portion of those tendons. Although tension at the external portion of 133 NB NW was near normal, autopsy revealed two full strand breaks in the internal area of that part of the tendon near the trumpet. This result illustrates how force development can hide to some extent the signs of nearby internal corrosion when conducting measurements in the external portion.					
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NOTICE

The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation or the U.S. Department of Transportation. Prepared in cooperation with the State of Florida Department of Transportation and the U.S. Department of Transportation.

EXECUTIVE SUMMARY

Under this project vibrational data from the vertical external tendons in piers of the high level approaches of the Sunshine Skyway Bridge were analyzed to obtain tendon tension estimates. These pier columns have been subsequently further reinforced and post tensioned, and some of the external lengths investigated are now partially or completely internal. The results provide archival information of the state of tension of the tendons prior to strengthening of the piers.

The estimated tension values were on average ~110 kN/strand or ~24.8 kips/Strand, which is in the order of that expected under normal conditions. Of the entire set tested, dramatically low estimated tension (about 1/4 that of the average) was obtained only in one tendon (Pier 133 NB, NE tendon segment). Subsequent direct inspection in another investigation revealed 15 broken strands out of 17 in that tendon segment, showing that the vibration test could reveal the presence of extensive deterioration in the internal zone. The other tendon segment tested in Pier 133 NB (133 NB SW) also had estimated tension close to the average, and autopsy revealed no pitted or failed wires and no failed strands. However, tests on tendon segment 133 NB NW yielded estimated tension close to normal but autopsy revealed 2 out of 17 strands failed. Inter strand and strand/grout force development are expected in that case to have been efficient enough there so that tension in the external portion of that tendon end was little affected. It is noted also that near-normal tensions obtained for 133 NB NW and 133 NB SW, even though the mating opposite ends were severely detensioned, indicate efficient friction and force development at the grouted hairpin-bend sections of the tendons. The results for Pier 133 NB illustrate both the strengths and limitations of the vibrational test method as a means of revealing hidden corrosion damage and structural condition.

Outside Pier 133 NB the estimated tensions of nearly of all tendon segments showed near normal values. It is concluded therefore that in this group, no serious detensioning conditions existed in the external portions of the tendons amenable for vibration testing and that if any strand breaks existed in the internal portions, efficient force transfer to the rest of the tendon had taken place in those regions. Independent direct inspection of selected piers in this group revealed only minor to moderate corrosion of the external tendon portions, and only one instance of severe corrosion (three strands failed) at a tendon recess in the thick wall region. The estimated tension for the corresponding tendon segment did not deviate appreciably from the average, consistent with the expectation that force development was efficient at the long internal portion of that tendon.

NOTE ON SCOPE OF THIS REPORT

The main findings of this program were obtained using a simplified tendon force calculation procedure, and conveyed to the Florida Department of Transportation (FDOT) in an interim report dated February 6, 2002. Those preliminary results are listed in Reference [1]. Preparation of a final report was postponed however, as it was anticipated that a more advanced force calculation method would be developed in an upcoming FDOT project. An initial version of such method has now become available from progress in the FDOT Project "Validation and Practical Procedure for Vibrational Evaluation of Tendons", BC353 RPWO #44, and has been applied to recalculate the force estimates of all the regular bridge spans. The results essentially confirm the preliminary findings and are presented in this Final Report which supersedes any previous version.

BACKGROUND

The objective of this project was to plan, prepare for, and analyze vibrational data to be acquired from the vertical external tendons in columns the high level approaches of the Sunshine Skyway (SSK) Bridge by a separate contractor. These pier columns have been subsequently further reinforced and post tensioned, and some of the external lengths investigated are now partially or completely internal. The results provide archival information of the state of tension of the tendons prior to strengthening of the piers.

The University of South Florida (USF) prepared in cooperation with FDOT a vibrational test plan for the tendons. USF prepared test equipment based on that used for a previous investigation conducted for FDOT for the Niles Channel Bridge [2]. In addition, USF trained both in Tampa and at the SSK Bridge the separate contract operators on the operation of the test equipment.

The tests were conducted adapting the methodology described in the report "Initial Development of Methods for Assessing Condition of Post-Tensioned Tendons of Segmental Bridges" [2], except that each tendon segment was subject to four test impacts; two in a direction perpendicular to the inner column wall and two parallel to the wall, yielding four data files and four manual records of test results. Each manual result record consisted of Mode 1 and Mode 2 frequencies, read by the operator from the computer screen; precise tendon segment length and air temperature measured by operator, and supplementary information such as observations of tendon vibration obstructions. Manual records and data files were conveyed regularly to USF for analysis. The data processed (including some earlier observations) were acquired between January and May, 2001 and delivered to USF. Data consisted of (i) the manually recorded data forms (frequencies for the Mode 1 and 2 peaks for each tendon were used for simplified calculations not reported here) and (ii) electronic files in .wav format containing the accelerometer output records of each measurement.

For the calculations the mass per unit length was estimated to be 20.351 kg/m (13.66 lb/ft) and 19.744 kg/m (13.26 lb/ft) for 18 and 17 strand tendons respectively, based on

standard 0.5-inch diameter strand mass per unit length, assuming densities of $1,650 \text{ kg/m}^3$ (103.1 lb/ft^3)^A and $1,000 \text{ kg/m}^3$ (62.4 lb/ft^3) for the grout and duct polymer respectively, and assuming that the polymer duct in all tendons had an outer diameter of 88.9 mm (3.5 in) and a wall thickness of 4.76 mm (3/16 in). Tendon segment 133 NB NE and 133 NB NW had been stripped of its polymer duct and grout before vibration testing, so its mass per unit length was estimated at 13.09 kg/m (8.79 lb/ft) based on the mass of the steel only.

The force calculations were made assuming also that the free vibrating length was the clear measured concrete-to-concrete distance between ends, reflecting any noted concrete or grout accumulation at the ends. In some of the longest columns the tendon ran at midlength through a narrow concrete opening that partially restricted vibration. A wooden wedge was inserted at the opening acting effectively as a fret in that position. The resulting free length was used for the test and the calculations were conducted assuming that the end condition was the same as that of a fully clamped end. That approximation is expected to have introduced little error as the resulting free length was still quite long and the effective non-dimensional stiffness was correspondingly small. The freely vibrating tendon lengths and number of strands used in the calculations are reported in Appendix 1.

RESULTS

Tendon segments were designated by their position in the bridge high level approaches as exemplified by 098 NB NW where 098 is the pier number, NB indicates the column for the Northbound lane, and NW indicates the tendon segment in the NW quadrant of the column cross section. Within each column, the NE and NW segment pair are parallel ends of the same hairpin tendon that makes a U-bend near the bottom of the column. A similar configuration exists for the SE and SW tendon pair. Reference [1] contains detailed information on the column configurations.

Force estimates were calculated from the available data in nearly all cases. Exceptions were the tendons in piers 88, 89, 134 and 135 for which the free tendon lengths were less than 1m (3 ft) (too short for straightforward calculation^B) and tendons 124 SB NE,

^A The $1,650 \text{ kg/m}^3$ (103.1 lb/ft^3) value adopted for the grout density in this report is smaller than the one ($1,850 \text{ kg/m}^3$ (115.5 lb/ft^3)) used for analysis of data from the tendons in the high level approaches of the SSK bridge [3]. The present value, used also for analyses elsewhere where grout density was found to be relatively low [4], was adopted to reflect the observation of chalky grout in Pier 133 NB and other grouting conditions documented in the main inspection report [1]. Use of $1,850 \text{ kg/m}^3$ (115.5 lb/ft^3) would result in mass per unit length values of 20.98 kg/m (14.09 lb/ft) and 20.40 kg/m (13.69 lb/ft) for 18 and 17 strand tendons respectively, a relatively small change. If desired for comparison purposes, estimated force values in Table 1 can be modified accordingly by simple prorating by the corresponding mass per unit length.

^B Tension estimates may be obtained for these short tendons in the future if further analysis methods are developed and validated under ongoing Project BC353 RPWO #44. An update to this report would be prepared in that case.

097 SB NW and 093 SB SE where obstructions to vibration prevented acquiring reliable data. In four other cases (100 NB SW and SE; 097 NB NW, 093 NB SE) partial obstructions resulted in unusually large vibrating length uncertainty; the calculated estimated tensions were so noted. It is noted that tendon segments 133 NB NW and NE were cut out for direct examination after the vibration tests were conducted. Tendon segment 133 NB SE had experienced severe corrosion before the vibration tests had been initiated and was not suitable for vibration testing. Estimated tensions are reported in Table 1A and 1B as Force per Strand both in SI (kN/Strand) and in English units (kips/Strand; kips=1,000 pound-force) respectively. Note that 1 kN = 0.225 kips.

The results in Table 1 are displayed following a plan arrangement similar to that of the bridge and a NW, NE, SW and SW scheme for each column as indicated in the key. The estimated tension results are presented using 0.1 kN/Strand and 0.1 kips/Strand precision to minimize the effect of roundoff errors in any later numeric manipulation by users of the results. Experience to date shows that typical reproducibility of results (keeping input parameters the same) using the present data acquisition and calculation procedures is in the order of 1% to 2%, or about one significant digit less than the tabulation precision. Test uncertainty (e.g. mechanical complications in vibrational response, spectrum peak identification problems) could occasionally result in larger variability. If improved calculation models become available in the future, tensions may be recalculated accordingly. To that end, the original electronic data files from these field tests are archived permanently at USF.

Table 1 shows also the disparity of estimated tension of both ends of each hairpin tendon, calculated as the E minus W tensions, divided by the average of both and expressed as a percentage. Either unusually overall low tension, or unusually large disparity between tensions at both ends of a given tendon would be indicators of potential distress from corrosion induced strand breakage or other sources.

It is emphasized that regardless of the reproducibility of the measurement results, the force estimates are also subject to global or individual inaccuracy from uncertainty in the input parameters used. Important among those is the mass per unit length, which depends on actual in place grout density. It may have differed globally from the assumed value, or varied from tendon to tendon due to grouting irregularities as were observed during inspections [1]. The grout contributes typically about 1/3 of to the total tendon mass per unit length, to which the force estimate is proportional. Thus, an uncertainty of 30% in grout density could translate in a potential error on force estimation of about 10%. Some uncertainty may exist also in the effective vibrating length, from undetected hidden vibration obstructions.

DISCUSSION

The results were in general agreement with those obtained during the preliminary calculations (taking into account that moderately smaller mass per unit length values were used at that time). The estimated tension values were on average ~110 kN/Strand or ~24.8 kips/Strand. This value corresponds to the force on regular "0.5 inch" strand

having a stress of 60% of the nominal strength (1.86 GPa or 270 ksi), which is in the order of that expected for the terminal condition after set and relaxation of tendons that were initially stressed to 75% of nominal strength and considering the potential sources of uncertainty indicated above.

Of the entire set tested, dramatically low estimated tension (about 1/4 that of the average) was obtained only in the 133 NB NE tendon segment. This result was consistent with subsequent direct inspection which revealed 15 broken strands out of 17 in that tendon segment, the breaks being located in the trumpet area [1]. In the absence of any frictional force redistribution, the residual tension would have been simply proportional to the fraction of sound strands left (2/17). However, the estimated tension was about twice as greater than that, comparing to the average estimated tension in the presumably mostly sound tendons in the other columns. Thus, appreciable friction between strands and force development against the grout partially filling the anchorage must have been taking place. The other end of the same tendon (133 NB NW) had an estimated tension very close to the average and yet later autopsy revealed 2 out of 17 strands failed and numerous wires failed in the trumpet and nearby internal region of that segment [1]. Inter strand and strand/grout force development are expected to have been efficient enough there so that tension in the external portion of that tendon end was little affected. The other tendon segment tested in Pier 133 NB (133 NB SW) also had estimated tension close to the average, and autopsy revealed no pitted or failed wires and no failed strands [1]. It is noted also that the near-average tensions obtained for 133 NB NW and 133 NB SW, even though the mating opposite ends were severely detensioned, indicate efficient friction and force development at the grouted hairpin-bend sections of the tendons. The results for Pier 133 NB illustrate both the strengths and limitations of the vibrational test method as a means of revealing hidden corrosion damage and structural condition.

Outside Pier 133 NB the estimated tensions of nearly of all tendon segments deviated little from the average value. The average absolute value of the observed E-W tension disparity was about 3% and very few tendons showed absolute value disparities exceeding 10%, a value deemed to merit some note.^c Of those tendons, only one (segment pair 130 NB NW - NE) was not noted otherwise for vibration length uncertainty, and had a disparity of only 11.3 %. It is concluded therefore that in this group no serious detensioning conditions existed in the external portions of the tendons amenable for vibration testing and that if any strand breaks existed in the internal portions, efficient force transfer to the rest of the tendon had taken place in those regions. Direct inspection of selected piers [1] in this group revealed only minor to moderate corrosion of the external tendon portions, and only one instance (Pier 131 SB, NE tendon) of severe corrosion (three strands failed) at a tendon recess in the thick wall region. The estimated tension for the corresponding tendon segment (131 SB

^c An absolute disparity value of 6% was chosen as a threshold to flag tendons that may merit special attention in similar analyses in other FDOT bridges [2] where segmental box tendons had intermediate deviation blocks. The threshold was raised to 10% in the present case as the hairpin configuration at the bottom of the tendon is expected to introduce relatively large frictional forces which would allow greater differences between the tensions at opposite ends under normal circumstances.

NE) did not deviate appreciably from the average, consistent with the expectation that force development was efficient at the long internal portion of that tendon based on the findings in Pier 133 NB.

CONCLUSIONS

1. Vibrational testing was practicable for virtually all the tendon segments in 68 of the high level approach piers of the bridge.
2. Estimated tensions for the external portions of all but one of the tendon segments evaluated approached values expected from typical design conditions. These observations are consistent with independent direct inspection of selected piers (except Pier 133 NB noted below) that revealed only minor to moderate corrosion of the external tendon portions, and only one instance of severe corrosion at a tendon recess in the thick wall region, far from the external portion.
3. Dramatic tension loss was measured only for the NE tendon segment in Pier 133 NB. Upon autopsy, that part of tendon was found to have experienced severe corrosion damage and breakage of most strands in the upper internal portion.
4. Tensions at tendon segments 133 NB SW and 133 NB NW were near normal, even though severe detensioning had taken place at the opposite ends of both tendons. This observation indicates effective force transfer development in the lower, hairpin bend portion of those tendons.
5. Although tension at the external portion of 133 NB NW was near normal, autopsy revealed two full strand breaks in the internal area of that part of the tendon near the trumpet. This result illustrates how force development can hide to some extent the signs of nearby internal corrosion when conducting measurements in the external portion.

REFERENCES

- [1] Parsons Brinckerhoff Quade & Douglas, Inc. in association with Concorr Florida, Inc. and Kisinger Campo & Associates Corp., "Sunshine Skyway Bridge Post Tensioned Tendon Investigation", FPN: 411135 1 32 01, Report to Florida Department of Transportation District Seven, February 6, 2002, revised February 13, 2002.
- [2] Final Report to FDOT "Initial Development Of Methods For Assessing Condition Of Post-Tensioned Tendons Of Segmental Bridges" Contract # BC374, A. A. Sagüés and S. C. Kranc, May 17, 2000.
- [3] Final Report to FDOT on Task 4 of Project Validation and Practical Procedure for Vibrational Evaluation of Tendons, BC353, RPWO #44: "Results of Detailed Analysis of

Vibration Testing of External Tendons in the High Approaches of the Sunshine Skyway Bridge” A.A. Sagüés, T.G. Eason, C. Cotrim and A. Saradhi, December 22, 2003.

[4] Final Report to FDOT on Project, BC353, RPWO #17: “Test Preparation and Data Analysis for Vibrational Testing of Mid Bay Bridge Tendons” A. A. Sagüés, S.C. Kranc, V. Balakrishna, A. Saradhi, April 15, 2004.

Table 1A - Estimated tensions (kN/Strand) and E-W tension disparity. Part 1, North high approach piers. N.A.: calculations not available. N.T.: not tested. Obstr.: obstruction to vibration preventing test.

% (E-W)	SOUTHBOUND		PIER	NORTHBOUND		% (E-W)
	N.A.	N.A.	135	N.A.	N.A.	
	N.A.	N.A.		N.A.	N.A.	
	N.A.	N.A.	134	N.A.	N.A.	
	N.A.	N.A.		N.A.	N.A.	
2.7	110.5	107.6	133	111.3	24.8	127.1
3.0	109.9	106.7		105.0	N.T.	
5.1	118.5	112.6	132	105.9	113.4	-6.8
-5.4	106.6	112.5		112.3	116.7	-3.8
4.6	111.7	106.7	131	105.9	104.2	1.6
0.4	110.6	110.1		108.5	108.2	0.3
-6.2	105.1	111.9	130	103.5	115.9	-11.3
-3.1	112.3	115.8		106.2	110.3	-3.7
-1.1	105.1	106.3	129	103.5	102.5	0.9
-1.1	106.7	107.9		104.4	109.3	-4.6
-3.1	104.4	107.7	128	106.8	108.0	-1.2
7.7	112.8	104.4		108.8	107.6	1.1
3.4	113.7	109.9	127	113.8	105.4	7.6
1.4	106.6	105.1		108.9	107.0	1.8
3.4	113.5	109.7	126	113.0	108.9	3.6
-2.2	108.8	111.3		114.1	110.6	3.1
-5.5	105.6	111.6	125	107.1	107.5	-0.4
-2.1	103.9	106.1		107.8	106.6	1.1
	114.0	Obstr.	124	107.7	111.0	-3.0
0.4	108.9	108.4		105.9	110.4	-4.1
0.9	118.9	117.9	123	113.2	114.6	-1.2
2.3	116.8	114.1		116.2	115.0	1.1
-5.7	106.8	113.0	122	107.0	112.2	-4.8
-3.0	108.4	111.6		108.9	111.0	-1.9
-1.5	106.9	108.5	121	107.1	109.7	-2.5
-0.1	112.1	112.2		105.5	110.3	-4.4
1.5	107.0	105.4	120	107.2	108.5	-1.2
-0.4	110.0	110.4		109.4	108.0	1.2
2.2	111.0	108.6	119	111.9	113.4	-1.3
-2.0	104.3	106.4		106.4	109.2	-2.6
-3.3	114.8	118.7	118	112.1	108.8	3.0
1.0	112.1	111.1		110.7	110.2	0.4
0.8	111.1	110.2	117	111.1	115.8	-4.2
-2.3	104.4	106.9		103.0	100.2	2.8

Key:

NW	NE
SW	SE

Table 1A - Estimated tensions (kN/Strand) and E-W tension disparity. Part 2, South high approach piers. N.A.: calculations not available. N.T.: not tested. Obstr.: obstruction to vibration preventing test. Italics: uncertain vibration length.

% (E-W) OFF	SOUTHBOUND		PIER	NORTHBOUND		% (E-W) OFF
1.5	109.8	108.1	106	111.6	110.2	1.3
-1.3	106.5	107.9		108.5	105.6	2.7
0.0	110.9	110.9	105	112.3	110.3	1.9
-0.3	111.1	111.4		109.0	108.5	0.5
-0.2	107.6	107.8	104	110.0	105.6	4.1
1.8	109.5	107.6		113.6	116.8	-2.8
-3.7	108.3	112.4	103	113.7	109.6	3.6
3.5	108.7	105.0		113.1	113.4	-0.2
4.6	106.1	101.4	102	104.3	109.0	-4.3
-2.9	109.2	112.4		110.1	108.3	1.7
2.6	114.3	111.3	101	110.8	109.5	1.2
0.0	106.2	106.2		109.0	107.4	1.5
-3.4	108.6	112.4	100	106.2	111.7	-5.1
2.9	111.7	108.5		97.2	120.2	-21.2
8.3	115.1	105.9	99	110.9	108.6	2.1
-0.4	105.4	105.9		106.3	109.1	-2.6
-1.7	114.0	115.9	98	114.6	113.0	1.3
0.3	108.3	107.9		110.5	107.9	2.3
	Obstr.	110.5	97	103.1	110.9	-7.3
1.1	112.1	110.8		103.8	107.9	-3.9
5.1	115.1	109.4	96	112.1	107.3	4.4
-5.1	102.6	107.9		105.8	108.0	-2.1
5.9	113.9	107.3	95	114.8	107.5	6.5
4.2	108.3	103.8		108.1	107.1	0.9
1.3	112.6	111.2	94	112.2	109.7	2.2
-1.3	105.3	106.6		105.9	107.1	-1.1
4.6	114.0	108.9	93	111.7	109.6	1.9
	114.7	Obstr.		105.3	98.3	6.9
9.6	118.1	107.3	92	111.6	109.2	2.2
-4.2	103.8	108.3		108.5	105.8	2.5
-8.0	112.5	121.9	91	105.6	102.2	3.3
3.1	105.4	102.1		102.6	105.2	-2.5
1.9	100.2	98.3	90	102.5	103.7	-1.2
-0.9	97.2	98.0		109.9	107.0	2.6
	N.A.	NA	89	N.A.	NA	
	N.A.	NA		N.A.	NA	
	N.A.	NA	88	N.A.	NA	
	N.A.	NA		N.A.	NA	

Key:

NW	NE
SW	SE

Table 1B - Estimated tensions (kips/Strand) and E-W tension disparity. Part 1, North high approach piers. N.A.: calculations not available. N.T.: not tested. Obstr.: obstruction to vibration preventing test.

% (E-W)	SOUTHBOUND		PIER	NORTHBOUND		% (E-W)
	N.A.	N.A.	135	N.A.	N.A.	
	N.A.	N.A.		N.A.	N.A.	
	N.A.	N.A.	134	N.A.	N.A.	
	N.A.	N.A.		N.A.	N.A.	
2.7	24.9	24.2	133	25.0	5.6	127
3.0	24.7	24.0		23.6	N.T.	
5.1	26.6	25.3	132	23.8	25.5	-6.8
-5.4	24.0	25.3		25.3	26.2	-3.8
4.6	25.1	24.0	131	23.8	23.4	1.6
0.4	24.9	24.8		24.4	24.3	0.3
-6.2	23.6	25.1	130	23.3	26.1	-11.3
-3.1	25.2	26.0		23.9	24.8	-3.7
-1.1	23.6	23.9	129	23.3	23.0	0.9
-1.1	24.0	24.2		23.5	24.6	-4.6
-3.1	23.5	24.2	128	24.0	24.3	-1.2
7.7	25.4	23.5		24.5	24.2	1.1
3.4	25.6	24.7	127	25.6	23.7	7.6
1.4	24.0	23.6		24.5	24.1	1.8
3.4	25.5	24.7	126	25.4	24.5	3.6
-2.2	24.5	25.0		25.6	24.9	3.1
-5.5	23.7	25.1	125	24.1	24.2	-0.4
-2.1	23.4	23.9		24.2	24.0	1.1
	25.6	Obstr.	124	24.2	25.0	-3.0
0.4	24.5	24.4		23.8	24.8	-4.1
0.9	26.7	26.5	123	25.4	25.8	-1.2
2.3	26.3	25.7		26.1	25.9	1.1
5.7	24.0	25.4	122	24.0	25.2	-4.8
3.0	24.4	25.1		24.5	24.9	-1.9
1.5	24.0	24.4	121	24.1	24.7	-2.5
0.1	25.2	25.2		23.7	24.8	-4.4
1.5	24.0	23.7	120	24.1	24.4	-1.2
0.4	24.7	24.8		24.6	24.3	1.2
2.2	24.9	24.4	119	25.2	25.5	-1.3
2.0	23.4	23.9		23.9	24.5	-2.6
3.3	25.8	26.7	118	25.2	24.5	3.0
1.0	25.2	25.0		24.9	24.8	0.4
0.8	25.0	24.8	117	25.0	26.0	-4.2
2.3	23.5	24.0		23.2	22.5	2.8

Key:

NW	NE
SW	SE

Table 1B - Estimated tensions (kips/Strand) and E-W tension disparity. Part 2, South high approach piers. N.A.: calculations not available. N.T.: not tested. Obstr.: obstruction to vibration preventing test. Italics: uncertain vibration length.

% (E-W)	SOUTHBOUND		PIER	NORTHBOUND		% (E-W)
1.5	24.7	24.3	106	25.1	24.8	1.3
1.3	24.0	24.3		24.4	23.7	2.7
0.0	24.9	24.9	105	25.3	24.8	1.9
0.3	25.0	25.0		24.5	24.4	0.5
0.2	24.2	24.2	104	24.7	23.7	4.1
1.8	24.6	24.2		25.5	26.3	-2.8
3.7	24.3	25.3	103	25.6	24.6	3.6
3.5	24.4	23.6		25.4	25.5	-0.2
4.6	23.9	22.8	102	23.5	24.5	-4.3
2.9	24.6	25.3		24.8	24.3	1.7
2.6	25.7	25.0	101	24.9	24.6	1.2
0.0	23.9	23.9		24.5	24.1	1.5
3.4	24.4	25.3	100	23.9	25.1	-5.1
2.9	25.1	24.4		21.8	27.0	-21.2
8.3	25.9	23.8	99	24.9	24.4	2.1
0.4	23.7	23.8		23.9	24.5	-2.6
1.7	25.6	26.1	98	25.8	25.4	1.3
0.3	24.3	24.3		24.8	24.3	2.3
	Obstr.	24.8	97	23.2	24.9	-7.3
1.1	25.2	24.9		23.3	24.3	-3.9
5.1	25.9	24.6	96	25.2	24.1	4.4
5.1	23.1	24.3		23.8	24.3	-2.1
5.9	25.6	24.1	95	25.8	24.2	6.5
4.2	24.3	23.3		24.3	24.1	0.9
1.3	25.3	25.0	94	25.2	24.7	2.2
1.3	23.7	24.0		23.8	24.1	-1.1
4.6	25.6	24.5	93	25.1	24.6	1.9
	25.8	Obstr.		23.7	22.1	6.9
9.6	26.6	24.1	92	25.1	24.5	2.2
4.2	23.3	24.4		24.4	23.8	2.5
8.0	25.3	27.4	91	23.7	23.0	3.3
3.1	23.7	23.0		23.1	23.7	-2.5
1.9	22.5	22.1	90	23.0	23.3	-1.2
0.9	21.8	22.0		24.7	24.1	2.6
	N.A.	N.A.	89	N.A.	N.A.	
	N.A.	N.A.		N.A.	N.A.	
	N.A.	N.A.	88	N.A.	N.A.	
	N.A.	N.A.		N.A.	N.A.	

Key:

NW	NE
SW	SE

APPENDIX 1: - Freely vibrating tendon lengths (m). Part 1, North high approach piers. N.T.: not tested. Obstr.: obstruction to vibration preventing test. All tendons 17 strands except 18 Strands for Piers 126 to 130.

SOUTHBOUND		PIER	NORTHBOUND	
0.84	0.84	135	0.84	0.84
0.84	0.84		0.84	0.84
0.84	0.84	134	0.84	0.84
0.84	0.84		0.84	0.84
3.88	3.88	133	3.87	3.87
3.88	3.88		3.90	N.T.
3.87	3.87	132	3.89	3.88
4.00	3.87		3.88	3.88
6.91	6.91	131	6.91	6.91
6.91	6.91		6.92	6.91
6.93	6.93	130	6.90	6.90
6.93	6.93		7.05	6.91
9.96	9.96	129	9.96	9.93
9.96	9.96		9.82	9.94
12.87	12.90	128	13.03	13.00
12.87	12.90		13.01	13.00
13.02	13.02	127	13.00	13.00
13.02	13.02		13.01	13.01
13.00	13.00	126	13.00	13.01
13.00	13.00		13.01	13.01
13.01	12.96	125	13.02	13.02
13.01	13.01		13.00	13.02
15.92	Obstr.	124	16.00	16.05
15.92	15.89		16.00	15.89
13.03	13.03	123	13.03	13.03
13.03	13.03		13.03	13.03
15.98	15.98	122	16.07	16.07
16.22	16.01		16.07	16.07
15.97	16.00	121	16.08	16.08
15.98	15.98		16.08	16.08
25.20	25.20	120	25.19	25.19
25.20	25.20		25.19	25.19
25.22	25.22	119	25.19	25.19
25.22	25.22		25.19	25.19
28.43	28.23	118	28.16	28.16
28.23	28.23		28.12	28.13
22.07	22.07	117	22.00	22.00
22.07	22.07		21.93	21.93

Key:

NW	NE
SW	SE

APPENDIX 1:- Freely vibrating tendon lengths (m). Part 2, South high approach piers. N.T.: not tested. Obstr.: obstruction to vibration preventing test. Italics: uncertain length. All tendons 17 strands except 18 Strands for Piers 93 to 97.

SOUTHBOUND		PIER	NORTHBOUND	
22.00	22.07	106	22.15	22.15
22.10	22.15		22.15	22.15
28.19	28.19	105	28.20	28.20
28.19	28.19		28.20	28.12
25.08	25.08	104	25.14	25.65
25.08	25.21		36.21	36.21
25.19	25.19	103	34.49	34.49
25.19	25.19		34.49	34.49
16.07	16.07	102	15.98	15.98
16.07	16.07		16.03	16.06
16.07	16.07	101	15.93	16.05
16.07	16.07		15.98	15.98
13.02	13.02	100	13.01	13.01
13.02	13.02		13.01	13.01
15.99	15.99	99	15.94	16.14
15.99	15.99		16.00	16.00
12.95	12.95	98	12.93	12.93
12.93	12.93		12.93	12.93
Obstr.	12.82	97	9.07	12.88
12.95	12.87		12.88	12.88
12.93	12.93	96	12.94	13.05
13.00	12.93		12.94	12.94
12.93	12.93	95	13.03	13.03
13.00	12.98		13.03	13.03
9.90	9.90	94	9.91	9.91
9.90	9.90		9.91	9.91
6.94	6.89	93	6.85	6.85
6.94	Obstr.		6.85	6.01
6.80	6.78	92	6.78	6.73
6.78	6.83		6.78	6.78
3.81	3.89	91	3.80	3.75
3.81	3.81		3.80	3.80
3.81	3.81	90	3.78	3.78
3.76	3.76		3.89	3.89
0.84	0.84	89	0.85	0.85
0.84	0.84		0.85	0.85
0.84	0.84	88	0.85	0.85
0.84	0.84		0.85	0.85

Key:

NW	NE
SW	SE

UNIT CONVERSIONS TABLE

CONVERSION FACTORS, US CUSTOMARY TO METRIC UNITS

<i>Multiply</i>	<i>by</i>	<i>to obtain</i>
inch	25.4	mm
foot	0.3048	meter
square inches	645	square mm
cubic yard	0.765	cubic meter
pound/cubic yard	0.593	kg/cubic meter
inch ² /year	2.046 10 ⁻⁷	cm ² /sec
gallon/cubic yard	4.95	liter/cubic meter
standard cubic feet/hour	466.67	ml/minute
ounces	28.35	gram
pound	0.454	kilogram
pound (lb)	4.448	newtons
kip (1000 lb)	4.448	kilo newton (kN)
pound/in ²	0.0069	MPa
kip/in ²	6.895	MPa
ft-kip	1.356	kN-m
in-kip	0.113	kN-m