SIXTH STREET VIADUCT OVER LOS ANGELES RIVER
(Bridge No. 53C-1880)

Field Sampling and Testing Program Final Report

Submitted to
The City Of Los Angeles
Bureau Of Engineering

Prepared by
W. Koo & Associates, Inc.
Wiss, Janney, Elstner Associates, Inc.

February 7, 2002

TABLE OF CONTENTS

EXECUTIVE SUMMARY

For the performance of Phase II material testing of The Sixth Street Bridge over the Los Angeles River (Bridge No. 53C-1880), W. Koo & Associates, Inc. (WKA), under contract with the City of Los Angeles and with close supervision of City staff, collaborated with Wiss, Janney, Elstner Associates, Inc. (WJE). The material testing program included a preliminary visual survey, extraction and documentation of 88 concrete core samples and six rebar samples, nondestructive testing (NDT) of six bridge locations, and laboratory testing. The laboratory testing included compression strength testing, Young's modulus testing, petrographic examination of core samples, and tensile testing of the rebar samples. This report contains the findings and recommendations from this testing program.

The results of the material testing program indicate that there is significant cracking due to alkali-silica reaction (ASR) throughout the Sixth Street Viaduct. Though the severity of cracking varies depending on location, cracking occurs along the entire length of the bridge. Cracking was observed in all bridge elements including the railings, the deck, the girders, the bent caps, the columns, and the foundations. Severe cracking due to ASR was observed in numerous bridge elements.

The severe cracking was observed as a network of cracks extending throughout the element–not only on the surface but throughout the interior of the element. Cracking at the interior appeared to be more severe than cracking at the surface of the elements. Areas of the viaduct with high moisture exposure had the most distress. Locations of high moisture exposure with severe cracking include the areas below the bridge deck expansion joints, the exterior columns and girders, the foundations, and the lower portion of the columns near grade. The areas protected from rain and away from the base of columns, where moisture is present from the soil, generally had less deterioration.

The preliminary visual survey found significant surface cracking in most portions of the bridge. In general, severe surface cracking exists between Bents 12 to 30, moderate to severe surface distress was noted between Bents 1 to 11, and light distress was noted between Bents 30 to 39. Approximately 40% of all substructure elements show severe distress, 30% show moderate distress, and 30% show light distress. The visual survey ratings correlated well with the severity of cracking observed in the 88 cores removed from the structure. Generally, the visual surface condition is a conservative predictor of the severity of the internal cracking.

Petrographic examination of the concrete core samples confirmed that cracking in the viaduct is caused by ASR. Evidence of ASR was found in all 21 cores examined petrographically. Even cores from lightly distressed locations had moderate levels of ASR activity. The petrographic studies found that most of the aggregates in the concrete are reactive. The reactive aggregates, listed from most to least reactive, are rhyolitic tuff, graywacke, granitic gneiss, and quartzite. For a particular rock or mineral type, the intermediate-sized particles appeared to be the most reactive.

The petrographic studies found that ASR appears to be active and is continuing to cause distress in the concrete. The field observations of new cracks forming next to epoxy injected cracks confirm this conclusion. Given that some of the cracks were injected within the last ten years, new cracks have formed in a relatively short time. In addition to the new surface cracks, new internal cracking is probably occurring.

The cracking has reduced the compressive strength and Young's modulus of the concrete. The compressive strength tests of 51 cores from the Sixth Street Bridge show that the sound portions of severely cracked cores have an average of 25% less compressive strength (3,000 psi with severe cracking vs. 4,000 psi uncracked) than uncracked cores. The average Young's modulus of the eight cores tested was 2.5×10^6 psi. The compressive strength and Young's modulus values for the severely cracked cores do not reflect that the heavily cracked portions of the cores were not testable and that several core samples were too cracked to test.

Epoxy injection of cracks has been used as a repair for cracking in the bridge, but has proven to be ineffective. The network of cracking extends completely through the members and is worse at the interior of the members, which are over seven feet thick for columns. The deepest penetration of injected epoxy in the core samples was roughly 6 inches, so the internal cracking is not addressed. In addition, the epoxy injection has not stopped the cracking due to ASR as is evidenced by new cracks forming next to injected cracks on the surface of elements.

Though the cracking has reduced the compressive strength and Young's modulus of the concrete, the preliminary visual survey did not identify members with severe cracking patterns which were indicative of structural failure. In addition, significant cracking in the bridge was noted as early as 1953. Considering this, and the lack of obvious indications of structural failure, the structure appears adequate to remain in service for the near future, absent seismic considerations. Given its distressed condition, a thorough load rating analysis of the bridge should be performed to evaluate its current gravity load capacity. In addition, selected elements of the bridge could be instrumented to measure stress levels during service.

Since ASR appears to be still active in the structure, the most effective means to address further deterioration due to ASR is to keep the concrete dry. However, in practice, this is difficult to do. Surface coatings and penetrating water repellents can reduce the rate of ASR but do not stop the deterioration completely. Barrier coatings have proved to be ineffective and are maintenance-intensive repairs. Also, coatings and sealers are problematic for below-grade elements.

Rehabilitation of the viaduct could include:

1. Providing a drainage system for the deck.
2. Improving waterproofing of the deck and deck expansion joints to prevent water infiltration into the substructure.
3. Replacing the deck.
4. Adding jackets to mildly damaged columns and bent caps to improve ductility.
5. Replacing severely cracked members.

Severely cracked members should be monitored and considered for replacement in the next five to ten years, since ASR is still active and continuing to deteriorate already heavily cracked members. Continued cracking will cause failure of the members. Repair instead of replacement of the severely cracked members may be possible, but there are significant unknowns on how the severely cracked members would perform under seismic loading.

PROJECT BACKGROUND

Introduction

W. Koo & Associates, Inc. (WKA) and Wiss, Janney, Elstner Associates, Inc. (WJE), under contract with the City of Los Angeles and in collaboration with City staff, have completed the Phase II Material Testing Program for the Sixth Street Viaduct over the Los Angeles River (Bridge No. 53C-1880). The Phase II Material Testing Program is part of a multi-phased program focused on documentation, evaluation, analysis and repair design of the bridge. The Material Testing Phase is the first phase of the investigation, and the findings will be used for future phases of the work. This report presents a summary of the objectives, scope of work, findings, evaluations and recommendations of the Phase II Material Testing Program.

Description of Bridge

Sixth Street Viaduct was completed in 1932 to create an entrance to the City of Los Angeles from burgeoning East Los Angeles. At that time, the Sixth Street Viaduct, which connects to Whittier Boulevard, was part of the State Highway system carrying Los Angeles traffic towards Whittier, Fullerton, and San Diego. The total length of the structure is 3,168 feet long running from San Mateo Street in the west to the I-5 Freeway in the east. Sixth Street Viaduct crosses several local streets including Santa Fe Avenue, Mesquite Avenue, South Mission Road, South Anderson Street, and South Clarence Street, as well as the 101 Freeway in the east. The structure also crosses the Los Angles River and a myriad of heavy rail and Metrolink tracks that parallel the river. To cross these streets and railroad tracks, the structure rises from San Mateo Street to about 60 feet at the river piers and maintains this height to the bluffs on the east.

The two main spans cross the Los Angeles River and consist of asymmetrical steel through arches. The portion of the structure crossing over the 101 Freeway is owned and maintained by Caltrans. The approach structure utilizes cast-in-place concrete girders with variable depth supported by concrete columns. The girder depth varies parabolically, deep at the supports and shallow in the center of each span. In addition, each girder's thickness differs across the deck. A wide center girder was built to accommodate light rail, two slightly narrower girders on each side carry highway traffic, and the outer girder is just wide enough to carry the sidewalk. Figures 1 through 4 show overall views of the bridge. A summary of the bridge is shown below:

Superstructure Type	Approach spans- cast-in-place concrete "TEE" girders
	LA River spans- through steel overhead arch ribs with suspended deck
Substructure	Tapered concrete columns on concrete pedestals
Foundation	Approach Spans- spread footing, 15' to 20' +/- below ground
	LA River spans- timber piles (downward capacity= 20 tons +/-)
Total Span Length	3,178' (West Abutment to East Abutment)
Number of Spans	45
Spans within CT ROW	Bent 43 to Bent 46
Length within CT ROW	235'
Average Span Length	80'
River Spans	2 Spans at 180'each span
Width	46' curb to curb with 5' wide raised walkways on both sides
	Total out to out width = 59'-10"
Average Column Height	Approach Spans West – 30 'above adjacent ground
	Approach spans East – 55' above adjacent ground
	LA River span – 61' above river invert

General Condition

Over the past 70 years, the concrete has deteriorated as evidenced by map-type and longitudinal cracking throughout the structure. In the past, the City of Los Angeles has repaired cracks with epoxy injection that highlight the cracks and leave discoloration's. In 1951, monuments at the center river pier were removed due to the poor condition of the concrete. Approximately 20 years ago, the deck asphalt was stripped and a waterproof coating was applied to the concrete deck to try to prevent future deck cracking. The City resurfaced the deck area with an asphalt overlay paved directly over the new waterproofing. Additionally, the City has continued using epoxy injection to repair various surface cracks. Epoxy crack repair is evident throughout the structure; on the deck soffit, girders, bent caps, and most especially on the columns. Despite the frequent crack repair, the concrete continues to deteriorate as new cracks occur adjacent to past epoxy repairs. The pattern of cracking in the bridge was recognized as an indicator of alkali-silica reaction (ASR) as early as 1953[1].

ASR is caused by the presence of aggregate with high silica content, such as opal, chert, and flint. The silica reacts with the calcium, sodium, and potassium hydroxide alkalis in portland cement concrete to form a gel-like material that becomes expansive. This gel undergoes extensive expansion in the presence of water or water vapor (a relative humidity of 60 to 80 percent is usually required), creating cracks around the aggregate and expansion of the concrete. Petrographic examination is an effective testing procedure to determine the presence of ASR.

Previous Seismic Retrofit Strategy

In 1997, the City of Los Angeles completed a seismic retrofit design of the Sixth Street Viaduct in response to the Category I seismic vulnerability rating established by the County of Los Angeles Department of Public Works. The retrofit strategy, as approved by the County, calls for constructing in-fill shear walls linking the three individual columns to create a continuum structure in the transverse direction. This results in reduced vulnerability of the structure because of the reduction of the joint moment in the columns. The design did not advance to construction because of the concern of the continuing deterioration of the bridge substructure due to ASR.

[1] State of California Department of Natural Resources Division of Mines, "Special Report 27: Alkali-Aggregate Reaction in California Concrete Aggregates," January 1953

Previous Material Testing Program

In late 2000, the City began a material sampling and testing program by extracting two core samples out of two columns from Bent 17 and Bent 30 of the Sixth Street Viaduct to conduct petrographic and strength testing. The core samples exhibited wide cracks parallel to the surface at 4 to 6 inch depth intervals, as well as signs of leaching at the crack interfaces. Petrographic examination confirmed that alkali-silica gel was present and is the main cause of the concrete cracking. Conceptual seismic retrofit strategies were developed as part of the material testing to evaluate feasibility of alternative retrofit concepts not employing the in-fill walls. Results of the material testing and the alternative concept for seismic retrofit were presented to the City in a Report titled "HBRR Study Report For Sixth Street Bridge over the Los Angeles River"[2] on April 2001. The report presented four retrofit alternative concepts as described below:

- ➔ *Alternative 1-* The baseline alternative using the in-fill wall design developed by the City of Los Angeles.
- ➔ *Alternative 2-* The baseline alternative combined with repairs of columns and beams by epoxy injection into cracks, and historic rehabilitation of the barrier rails.
- ➔ *Alternative 3-* Replacing all exterior columns and partial replacement of the bent caps and girders, and adding grade beams and pile cap overlays. The purpose was to replace portions of the structure that had suffered serious concrete deterioration due to ASR.
- ➔ *Alternative 4-* This alternative included structure widening in order to remove itself from the "functional obsolete" designation in the Bridge Inspection Report. The widening will be combined with seismic retrofitting measures similar to Alternative 3 by replacing the seriously damaged columns and beams, and to reconstruct foundations due to the anticipated higher loads.

[2] City of Los Angeles Bureau of Engineering, "HBRR Study Report For Sixth Street Bridge over Los Angeles River," April 2001

Phase II Material Testing Program

In October 2001, W. Koo & Associates (WKA), under contract with the City of Los Angeles, began the Phase II material sampling and testing program on this bridge. WKA, in cooperation with Wiss, Janney, Elstner Associates, Inc. (WJE), has performed a comprehensive sampling and testing program by collecting core samples throughout the 3,178-foot viaduct, including the center pier of the Los Angeles River span. In addition, impact echo and pulse velocity tests were conducted by WJE at six locations to determine the presence of sub-surface cracks. The primary objectives of the testing program were:

1. To confirm the extent of deterioration via a preliminary visual survey of the viaduct and by core sampling along the length of the viaduct.
2. To determine the depth of cracking in representative elements of the bridge foundations, substructures, and superstructures.
3. To test representative core samples for compressive strength and Young's modulus for use in structural analyses.
4. To conduct petrographic testing to verify the presence of ASR, identify the reactive aggregates, and assess the potential for future deterioration due to ASR.
5. To perform non-destructive testing using impact echo and pulse velocity methods in limited areas to assess the viability of these methods for future testing.
6. To identify options for repairs or removal and replacement of affected portions of the structure.

SCOPE OF MATERIAL TESTING PROGRAM

The Phase II Material Testing Program consisted of a preliminary visual survey of the bridge, core extraction and material testing, non-destructive testing and petrographic evaluations of cores. The scope of each of the five primary tasks is described below:

Preliminary Visual Survey

WJE performed a preliminary, visual, walk-through survey of all accessible portions of the bridge superstructure. The purpose of the survey was to identify the various types of surface deterioration and their approximate distribution along the length of the bridge and also among the various elements of the bridge. The walk-through survey was performed entirely from ground level, below the bridge deck, without binoculars or other visual aids. The survey consisted of viewing the soffit of the deck, girders, bent caps, and columns of each bent and assigning an overall rating of light, moderate, or severe to classify the visual condition of the members. Overall photographs were taken at each bent. The entire visual survey was performed by one person in one day, taking roughly 5 to 10 minutes per bay. Appendix A contains a summary of the visual survey findings

Core and Rebar Extraction and Strength Testing

Cores were removed from the concrete columns, girders, bent caps, deck soffits and several below-grade elements. Cores were typically four inches in nominal diameter except at the bridge deck soffit where 3-inch nominal diameter cores were removed. The cores were extracted using a wet coring process with Diamond core drilling bits. Cores were removed from locations of the various elements that appeared to be representative of the overall visual condition of the element. All cores extended to roughly the mid-depth of the member in order to examine the conditions in the center as well as near the surface of the concrete. Immediately after extraction, all cores were visually examined, photographed and given a visual condition rating of no cracking or light, moderate, or severe distress, based on the extent of cracking. A total of 88 cores were extracted. The core log is included in Appendix B.

Concrete testing included compressive strength testing using ASTM C42 and Young's modulus testing per ASTM C469. The distribution of testing for the different bridge elements sampled is shown in Table 1.

TABLE 1- Distribution of Testing by Bridge Element

	Compression Test	Young's Modulus	Petrography	Retained (No Test)	Total *
Columns	14	3	7	1	25
Bent Caps*	7	3	6	-	16
Girders	15	-	3	-	18
Decks	7	4	3	4	18
Below-Grade	8	2	2	-	12
Total *	51	12	21	5	89

* Core P17, taken from a bent cap, had both compression and petrography testing.

In addition to the concrete cores, six steel reinforcing samples were removed and tested. The rebar samples were removed from the base of columns and consisted of 1/2-inch or 3/8-inch ties that were 3-feet long. The samples were tested for tensile strength using ASTM A370.

Non-Destructive Testing

Two types of non-destructive testing (NDT) were performed at six locations in order to determine the presence of subsurface cracks and the effectiveness of NDT to evaluate the condition of the concrete. Impact echo (IE) and ultrasonic pulse velocity (UPV) methods were used. The NDT was performed on representative locations of columns, bent caps, girders, and deck soffits. Two of the locations were near core samples so that a direct comparison of NDT results and core samples could be made for correlation of NDT readings. The NDT locations are shown in Table 2 on page 12.

TABLE 2- Non-Destructive Test Locations

Test No.	Bent No.	Test Location	Type of Test	Number of Readings
NDT-1	21	Bent Cap: C through E	UPV	12 UPV Readings
NDT-2	21	North Column, South Elevation, near Core P1	IE & UPV	21 IE readings 7 UPV @ 5' spacing
NDT-3	26 through 27	Beam B @ Core C22	IE & UPV	18 UPV readings 18 IE readings
NDT-4	26	Bent Cap: A through C	IE & UPV	12 UPV readings 9 IE readings
NDT-5	26 through 26.25	Deck Soffit: B through C	IE	Approximately 9 IE readings
NDT-6	26	South Column, North Elevation	IE & UPV	8 UPV readings 16 IE readings

Petrographic Examination

Core samples representative of the various areas of the structure were examined by WJE using petrographic methods in general conformance with ASTM C856. A total of 21 cores were examined. The primary purpose of the petrographic studies was to verify the presence of ASR, identify the reactive aggregate, and assess the potential for future deterioration due to ASR. Petrographic examination also provides information on the paste quality, aggregate types, paste aggregate bond, and water-cement ratio, among other things. This information is used to evaluate the overall quality of the concrete.

FIELD OBSERVATION AND FINDINGS

The following is a summary of field observations and preliminary findings from the visual survey of the superstructure and the 88 concrete core samples taken in locations throughout the viaduct. Observations and findings from the laboratory tests are presented in a separate section below.

Distress At Columns

The visual survey rated 115 columns, including the river piers, and 25 core samples were removed from the columns. Cracking was observed at columns throughout the viaduct, typically as vertical cracks along the height of the columns, as shown in Figures 5 and 6. The outside (north and south) columns had the most cracking and the middle columns were noticeably in better condition. The column cracking tended to be more severe below joints in the bridge deck. There was no significant spalling observed at columns and no evidence of rebar corrosion on the column surface or within the cores. In addition to the vertical cracks, many of the outside columns had horizontal cracks on the outside face near the top of the bent cap, as shown in Figure 7. Nearly all columns have a vertical crack extending up 10 to 15 feet above-grade on each face. A summary of the visual survey ratings for the columns is shown in Table 3 and a summary of the visual and core ratings for cored columns is shown in Table 4.

TABLE 3 - Summary of Visual Survey of Columns

	Number of Columns with Various Distress Ratings				
	Severe	**Moderate to Severe**	**Moderate**	**Light**	**Total**
North Columns	14	4	8	13	39
Middle Columns	2	2	11	21	36
South Columns	12	6	12	10	40
Total	28	12	31	44	115

TABLE 4 - Visual and Core Ratings for Cored Columns

	Number of Columns with Various Distress Ratings					
	Severe	**Moderate to Severe**	**Moderate**	**Light**	**No Cracking**	**Total**
Visual Survey Rating	11	2	6	6		25
Core Survey Rating	16	1	2	1	5	25

Conditions at Bent Caps and Girders

The visual survey encompassed 40 bent caps and 35 bays, containing approximately 175 girders. The coring program included 15 cores from bent caps and 18 cores from girders. The most common condition noted was longitudinal cracking along the length of the bent cap or girder, as shown in Figures 8 and 9. Girders also had transverse cracks on the soffit and vertical cracks in the webs. The north and south girders often had horizontal web cracks at the ends on the outside faces, as shown in Figure 10. Bent caps tended to have vertical cracking in the webs and the cracking was more severe at bent caps below expansion joints. The severely deteriorated bent caps had map cracking patterns on the web, as shown in Figure 11. Many of the cracks have been injected with epoxy at several different times in the past. The bent caps also show heavy water staining and efflorescence at joint locations. Edge spalling at the top edge of bent caps and falling of concrete has occurred in only a few locations. The bent caps and girders between Bents 30 through 39 are in noticeably better condition than other parts of the structure.

The visual survey found that roughly one-third of girders had severe surface deterioration; one-third had moderate deterioration; and approximately one-third had light deterioration. The visual survey of the bent caps found 50% with severe distress; 30% with moderate distress; and 20% with light distress. Table 5 shows the breakdown of survey ratings by element type.

TABLE 5 - Summary of Visual Survey of Bent Caps and Girders

	Number of Elements with Various Distress Ratings.				
	Severe	**Moderate to Severe**	**Moderate**	**Light**	**Total**
Girder Bays	10	1	12	12	35
Bent Caps	15	4	13	8	40

Condition of Deck

The visual survey encompassed 37 bays of deck and 18 cores were removed from the soffit of the deck. The most common form of deterioration seen in the deck is common transverse shrinkage-related cracking, as shown in Figures 12 and 13. There were some limited areas of map (pattern) cracking, particularly at the bays with solid infill soffits at the bottom of the girders, as shown in Figure 14. As with other elements, the deck soffits have been injected with epoxy at several different times in the past.

The overall visual survey ratings for the decks were 35% with severe distress; 38% with moderate distress; and 27% lightly distressed. Of the 18 cores removed from decks, roughly 40% were severely cracked; 20% were moderately cracking; and 40% were lightly cracked. The cracking observed was typically parallel to the plane of the deck (horizontal delamination) and the bottom-most crack was typically about 3 to 5 inches from the bottom of the deck. The summary of the visual survey ratings for the decks are given in Table 6.

TABLE 6 - Summary of Visual Survey for Decks

	Number of Elements with Various Distress Ratings.				
	Severe	**Moderate to Severe**	**Moderate**	**Light**	**Total**
Deck Bays	12	1	14	10	37

Below-Grade Conditions

Portions of the columns, grade beams, and footings were excavated at Bents 21, 22, and 23, as shown in Figures 15 and 16. A total of 12 cores were removed from below-grade locations, with core depths extending to roughly the middle of the element. All 12 cores had severe cracking along the entire length of the core, as described below.

Visual Evaluation of Cores

The core extraction work occurred between November 13 and December 4, 2001. During that time, 88 cores were removed from columns, bent caps, girders, the deck soffit, grade beams, and footings of the viaduct. The location of each core was recorded and the condition of the core was documented with photographs and notes immediately after extraction. All data was entered into a spreadsheet for analysis.

As part of the initial documentation, each core was assigned a distress rating based on the visual appearance of the core. The rating was primarily based on the number, size, pattern, and density of cracking. Other conditions, such as friable aggregate, weathered surfaces, and ASR aggregate reaction rings, were also considered, but to a lesser extent due to the judgmental nature of these conditions. The rating categories were severe, moderate, and light distress and no cracking.

The conditions of the cores ranged from no apparent cracking to complete rubble upon removal. The most common form of cracking observed was a three-dimensional network of raveled, generally transverse to diagonal cracks along the length of the core. A typical example of the crack pattern is shown in Figure 17. The network cracking typically begins at about a 3 to 5 inch depth below the exterior surface and gradually worsens to the bottom of the core, i.e., to the mid-depth of the member being cored. Figure 18 shows a typical example. Cores that had the network cracking also tended to break into segments when removed. The number of breaks, and length of segments, varied up to as many as six breaks in a 48-inch core, as shown in Figure 19. A number of cores had so many cracks that they completely broke apart during removal, as shown in Figure 20. Figure 21 shows the condition of a core judged to be moderately distressed.

All below-grade cores had severe distress ratings. For the above-grade cores, the columns and bent caps had the highest percentages of severe or moderate-to-severe ratings, typically about 60% of the cores. The girders and deck cores had about 30% severe or moderate-to-severe distress ratings. For all cores, about 30% had no cracks, or light cracking. A summary of the visual core distress ratings is shown in Table 7. A summary of the core locations and core distress ratings is found in Appendix C.

TABLE 7 - Summary of Core Visual Distress Ratings

	No Cracking	Light Distress	Moderate Distress	Moderate to Severe Distress	Severe Distress	Total
Columns	5	1	2	1	16	25
Bent Caps	3	1	2	2	7	15
Girders	9	1	2	2	4	18
Decks	5	2	4	2	5	18
Below-Grade	0	0	0	0	12	12
Total	22	5	10	7	44	88
	25%	6%	11%	8%	50%	

Correlation of Visual Survey and Core Distress Ratings

The data from the visual survey of the substructure elements and the visual distress ratings for the cores were compared. The comparison found that 92% of core distress ratings correlated well to the visual survey ratings. A good correlation was defined as a maximum difference of one rating category. For example, a good correlation would be L to M, or M to S. A poor correlation would be L to S, or no cracking to M. In four of the six instances with poor correlation, the visual rating was worse than the core rating. The other two cases of poor correlation had the core rating worse than the visual rating.

The comparison also showed that the visual ratings tended to be a conservative predictor of the core condition in that 55% of the comparisons were identical and 30% had a visual rating worse than the core rating. The data also showed that the correlation was best for columns and bent caps, with 93-96% correlation and somewhat less for girders and decks, with 88% correlation. The difference is likely due to the fact that the preliminary visual survey rating scheme gave individual columns and bent caps ratings, but the five girders and entire deck soffit in one bay only had one overall rating each. A summary of the correlation between the visual survey and core ratings is given in Table 8. An overview of the correlation for each core is given in Appendix D.

TABLE 8 - Summary of Correlation Between Visual Survey and Core Distress Ratings

	Good Correlation	Poor Correlation	Visual Distress **equals** Core Distress	Visual Distress **worse than** Core Distress	Visual Distress **less than** Core Distress
Columns	24	1	17	2	6
Bent Caps	14	1	9	4	2
Girders	16	2	8	9	1
Decks	16	2	8	8	2
	70	6	42	23	11

Distress Patterns

The severity of the surface distress and the internal cracking (by correlation) varied significantly throughout the structure. Overall, the greatest deterioration was noted on the east side of the river between Bents 12 and 30. Figure 22 shows a typical location. The west end, between Bent 1 to 11, was roughly one visual rating category better, approximately an overall rating of moderate. Figure 23 shows a typical bay at the west end of the viaduct. The east end of the viaduct between Bents 31 and 39 is significantly less deteriorated than the remainder of the structure. A typical area is shown in Figure 24. A summary of the overall distribution of the visual survey ratings along the length of the viaduct is given in Appendix A.

The severity of distress also varied among the structure elements. The middle columns are in substantially better condition than the exterior (north and south) columns. This is particularly true for the middle and upper portions of the column that are less affected by soil moisture. Also, the girders were generally less distressed than the columns and bent caps with the outside girders closer in condition to the column and the interior girders less distressed. However, the girders at skewed bents, over removed railroad spurs, showed extensive longitudinal cracking and efflorescence. The cause of this extensive cracking is not clear but may be related to increased restraint conditions. The bent caps were generally in the worst condition, particularly at expansion joints in the deck.

The distress patterns among the structural elements appear to be related to moisture exposure of the element. The best example is the difference between the middle columns and the exposed north and south columns. Also, the girders are generally well protected from exposure to rain, except at the north and south sides which generally are more distressed. The girders also tend to show more distress at the bents, and especially where expansion joints are present. The bent caps below leaking joints are the most "exposed" elements to water due to joints that concentrate water runoff and can leak for several hours after a rain shower. The joint fill material can also trap and hold moisture for long periods. The bent caps at heavily leaking joints are typically the most cracked and the cracks allow water to penetrate to the interior of the member, as shown in Figure 25. Also, many of the side columns have horizontal cracks aligned with the bent caps where water accumulates, as shown in Figure 26.

Another moisture-related distress pattern is the column cracking near grade level. Middle columns generally appear to be in good condition, except near grade level. One column had a poor correlation between the visual and core distress rating. It was lightly deteriorated on the outside, but had a severely distressed core removed at the base of the column. Moisture in the soil is clearly affecting the condition of the concrete; note that all below-grade cores were severely distressed.

Effectiveness of Epoxy Inj ection Repairs

The past epoxy injection repairs made at the viaduct have not been effective at preventing further cracking due to ASR or restoring strength to structural elements. Visual observation of cracking adjacent to previous epoxy injection repairs indicates that cracking is continuing to occur even in areas of heavy epoxy injection repairs. Figure 27 shows a crack with two generations of epoxy injections and a new open crack immediately adjacent.

Visual observation of concrete core samples taken at epoxy-injected cracks shows that the epoxy has penetrated up to 6 inches from the surface, but the most severe cracking is deeper than 6 inches. Figure 28 shows the depth of epoxy penetration at a typical location. The nature of ASR expansion creates a random, poorly interconnected network of cracks. This inhibits the penetration of the epoxy deep into the concrete.

MATERIAL TESTING RESULTS

The material testing performed on representative samples removed from the structure included: concrete compression strength tests, concrete Young's modulus tests, tensile strength tests on reinforcement, and petrographic evaluation of the concrete. Nondestructive testing (NDT) was also performed at six locations. A summary of the test results follows.

Concrete Compression Strength Testing

Fifty one cores were designated for compressive strength testing. The cores were distributed among the various elements of the substructure and also along the length of the structure. The cores tested also represent concrete of varying severity of distress. Table 1 shows the distribution of compression tests by structural element.

The testing was performed in accordance with ASTM C42. A number of tests were performed on portions of longer samples and, generally, the core sections with the least cracking were selected for compressive testing. The approximate location of the test sample within the overall core sample was recorded. In six cases, the cores were so cracked that they were unable to be tested. In cases where the sound length of core was less than twice the diameter, a correction factor was applied to the test values per ASTM C42.

The average compressive strength of the 45 cores tested was 3540 psi, with a standard deviation of 965 psi. The test values ranged between 1510 and 5770 psi, and ninety percent of the values were above 2260 psi (the 10th percentile). The cores were tested in the as-received (dry) condition. Dry compressive strength values are typically higher than wet testing by about 10 to 20%. Since the sound portions of severely cracked cores were tested, these results likely over-estimate the compressive strength of severely cracked members. In addition, these values do not take into account that six of the severely cracked cores were too cracked to test. The average compressive strength value for severely cracked cores was 2949 psi for the 21 testable cores, and 2140 psi for all 26 cores using zero strength for the untestable cores.

The correlation between the compressive strength and the distress rating varied considerably. The lightly and moderately distressed cores had higher averages than the cores with no visible cracking. This result is counterintuitive and best explained by the small number of light and moderate test samples and the procedure of selecting the best portion of a core for testing, which may not represent the entire core. The severely cracked cores had a lower average compressive strength than all other cores, as would be expected. The breakdown of test results for all cores, as well as for cores of different visual distress ratings, is given in Table 9. Appendix E has all of the compressive strength results.

TABLE 9 - Summary of Compression Strength Results

	No. Tests	Average Compressive Strength	Standard Deviation	High Value	Low Value
No cracking	15	3975	500	4600	3030
Light Distress	3	4217			
Moderate Distress	5	4562	932	5770	3510
Moderate to Severe Distress	1	2380			
Severe Distress	21	2949	875	5250	1510
All Cores	45	3540	966	5770	1510

Concrete Young's Modulus Testing

A total of 12 cores were designated for testing to determine Young's modulus. The testing was performed in accordance with ASTM C469. A number of the cores were severely deteriorated and only eight cores could be tested. Of the eight cores tested, two cores only had short sound sections, less than 1.5 times the diameter as specified by ASTM C469. However, we believe the test results are still representative of concrete modulus.

The test values ranged between approximately 1.2 x 10^6 and 4.1 x 10^6 psi within an average value of 2.5 x 10^6 and a median value of 2.2 x 10^6 psi. The overall trend was that the cores with lower distress had higher Young's modulus values. The values for the Young's modulus are given in Table 10. The data for the Young's modulus testing is presented in Appendix F. While the correlation between compressive strength and Young's modulus is not exact, the modulus values are slightly lower than would be predicted by ACI 318 ($Ec=57000(f'c)^{0.5}$) for an average of 3,540 psi compressive strength (E=3.4 x 10^6 psi).

TABLE 10 - Summary of Young's Modulus Test Results

Core ID	Core Distress Rating	Young's Modulus
E3	N	4,060,000
E4	N	1,878,640
E10	N	4,133,000
E6	L	2,012,000
E2	M	2,850,000
E5	M	2,407,680
E7	MS	1,401,400
E9	S	1,176,000
E1	S	Not Testable
E8	MS	Not Testable
E11	S	Not Testable
E12	S	Not Testable

NDT Evaluations

Both the impact-echo (IE) and ultrasonic pulse velocity (UPV) methods were effective for finding the presence of internal cracking. In the two cases where testing was done near a core, the test method yielded results consistent with the visual observations of the core. The NDT testing report is found in Appendix G.

The NDT of the south column of Bent 21 was performed in the area where core P1 was removed. The IE readings showed a wide range of predominant frequencies and, also, in some cases, shallow delaminations. UPV signals were able to traverse the concrete at only one location and the signal had a very low velocity. These results indicate severe flaws in the concrete. The core P1 had severe network cracking along the full four-foot depth, and portions at roughly the 4 to 8-inch depth were rubble.

The second location with NDT at a core was on girder B between Bents 26 and 27. At this location, all 18 UPV readings traversed the member, yielding an average velocity of 10,580 feet per second. This is considered to be at the lower range of acceptable values for solid concrete. Core C22 was extracted from the area and noted to have no obvious cracks. The compression strength of the core was 3920 psi.

Both the IE and UPV tests were able to distinguish flawed concrete from sound concrete; however, due to the nature of the two technologies, the amount of information gleaned from the testing differs. The IE tests transmit and receive signals from the same face of the member. By analysis of the reflected signals, the depth of a flaw can be found; however, the IE method generally cannot "see" beyond the first flaw. The UPV method transmits a signal through a member and by analysis of the signal received on the other side, information can be gained about the condition of the concrete between. Very severe cracking would tend to completely block UPV signals, but network cracking will often create a circuitous path for the signal, which would translate into a low signal velocity. This phenomenon was observed on the Sixth Street Bridge where lightly deteriorated members had readings in the 10,000 to 12,000 ft/s range and moderately distressed members had velocities in the 6,000 to 9,000 ft/s range. Very severely distressed members had no through signal.

Based on the NDT of the six locations, the ultrasonic pulse velocity method provides the most information about the internal condition of the concrete. This method could be used to gain qualitative information about the density of internal cracking within a concrete member that would be useful for repair design purposes.

Petrographic Examination of Concrete Cores

Twenty-one cores extracted from each type of the substructure elements at various locations of the bridge were examined using petrographic methods per ASTM C856. The examinations also included evaluation of thin sections of selected portions of cores and powder mounts of the paste, selected aggregates, gel product and other materials of interest. A total of 35 specimens were examined.

The cores contained significant cracking throughout; however, the cracking was more severe at the interior portions of the cores. The cracks were caused by expansion of the alkali-silica gel, which is the product of the chemical reaction of silica contained in the aggregates, and alkali from the cement paste, in a process known as alkali-silica reaction (ASR). The examination found altered and consumed aggregate particles, abundant gel precipitation in cracks, aggregate sockets and air voids. ASR deterioration was found in all cores and the severity of deterioration was rated qualitatively on a scale of no ASR to very severe ASR. Of the 21 cores, 14 cores had severe or worse ASR conditions, as shown in Table 11 below.

TABLE 11 - Summary of Petrographic Rating of ASR

No ASR	Light ASR	Light-Moderate ASR	Moderate ASR	Moderate-Severe ASR	Severe ASR	Severe - Very Severe ASR	Very Severe ASR	Total
0	0	0	4	3	10	1	3	21

Most of the aggregates present in the concrete are reactive. The most reactive aggregates, in decreasing order, are as follows: tuff, graywacke, feldspar, granitic gneiss, and quartzite. For a given rock or mineral, the intermediate-sized particles appear to be the most reactive.

The tuff, graywacke, and feldspar also contain substantial amounts of potassium (k), sodium (Na), and calcium (Ca), which may become free and serve as a source of alkali in addition to that found in the cement. The cement paste was found to be carbonated to depths of 1/32 to 1-5/8 inches with the typical value roughly 1-1/4 inches. ASR in carbonated zones was observed to be less severe than in other areas.

The ASR distress noted in the cores is severe and appears to be ongoing. There also appears to be abundant, unreacted material remaining to fuel future reactions, provided moisture is present. Evidence of new ASR reactions was seen during the 2-week transit time between extraction and receipt at the laboratory. A copy of the complete report on the petrographic examinations is contained in Appendix H.

Reinforcing Steel Tensile Testing

Six samples of the steel reinforcement were removed from the east portion of the bridge and tested for tensile strength in accordance with ASTM A370. All samples were 3-foot long portions of column ties removed from the center column of Bents 18 through 23. A description of the samples is given in Table 12 below.

TABLE 12 - Description of Rebar Samples

Sample No.	Bent No.	Structure Location	Sample Description		Comment
			Rebar Size	Length	
R1	20	Column	1/2" Square	3'	No corrosion, Approx. 2 3/4" cover, Photo D2.46
R2	21	Column	3/8" Square	3'	No corrosion, Approx. 2 1/4" cover, Photo D2.47
R3	22	Column	3/8" Square	3'	No corrosion, Approx. 2 1/2" cover, Photo D2.48
R4	23	Column	3/8" Square	3'	No corrosion, Approx. 2 3/4" cover, Photo D2.49
R5	19	Column	3/8" Square	3'	No corrosion, Approx. 2 1/4" cover, Photo D2.63
R6	18	Column	1/2" Square	3'	No corrosion, Approx. 2 1/8" cover, Photo D2.78

The test results indicate that four of the six samples had yield and minimum tensile strengths consistent with ASTM 615 Grade 40 steel. The two samples with tensile strengths below the 70,000 psi minimum for Grade 40 steel were R-2, T = 61,600 psi (12%) and R-4, T = 68,300 psi (-3%). In the case of sample R-4, the sample fractured within the jaw section of the testing machine, which may have decreased the tensile strength value and the elongation. The test report is contained in Appendix I.

EVALUATION AND DISCUSSION

The Current Extent of ASR and Potential For Future ASR

The Sixth Street Viaduct shows significant distress to the columns, bent caps, girders, and deck along most of its length. The distress consists of an internal network of cracking that manifests itself on the surface as longitudinal cracking and map cracking. The severity of the surface distress is correlated to the severity of the internal cracking, and is a relatively conservative predictor of the internal cracking. The internal cracking, where present, exists through the full thickness of the member and is typically more severe internally than near the surface.

The pattern of cracking in the structure indicates that concrete exposed to moisture accelerates the ASR. All below-grade and near-grade portions of the structure had severely cracked core samples. Columns and girders had more cracking near the edges of the bridge, and bent caps below leaking joints were most severely cracked. Since there is such abundant material for further reactions, the most effective solution for slowing the ASR is to keep the concrete dry.

Multiple stages of ongoing alkali-silica reactions, dating back to at least the early 1950s, are causing the cracking. As previously stated, the alkali-silica reactions are still active. The petrographic evaluation noted abundant unreacted aggregates which will fuel ongoing reactions. New cracks forming in heavily epoxy-injected locations present evidence that ASR cracking continues to occur, even in severely cracked members. The potential for future ASR reactions and deterioration appears to be high.

A relative humidity of about 60 to 80% is needed within the member to sustain ASR. Some portions of the structure, such as bent caps at joints and below-grade elements, have readily available moisture and surface cracks. These cracks allow moisture penetration well into the concrete member. Other members, such as interior columns, interior girders, and bent caps away from joints are protected from rain and do not have severe surface cracking. These members will most likely continue to perform well, provided the moisture conditions do not change. The relative humidity of the members can be measured to evaluate if sufficient moisture exists for ASR to occur.

The Effect of ASR on Concrete Strength

Studies[3] have shown that the cracking caused by ASR will lower the flexural and compressive strength of unreinforced concrete specimens. The strength reduction is only partly related to the magnitude of expansion; it is also related to mix variables such as cement content, aggregate size and type. Core testing of samples from the Sixth Street Bridge has shown that the compressive strength of the sound portions of severely cracked cores is approximately 3,000 psi (which does not account for the core samples that were too cracked to test) and the strength of uncracked cores is roughly 4,000 psi. Since the sound portions of the severely cracked cores were used for testing, the results over-estimate the compressive strength of severely cracked members. Figures 29 through 39 show the variability of concrete cracking within the cores, and the portion of the cores tested and respective compressive strength results.

Core compressive strength test values are probably somewhat less than the in-situ compressive strength of the concrete due to the absence of core restraint during testing, and the likelihood that additional ASR microcracking has occurred in the cores after removal. The difference in strength test values between cracked and uncracked cores is not necessarily indicative of comparable reductions in tensile or shear strength of the concrete due to cracking. The predominant cracking is transverse to the core axis so the compression force can be transmitted across the cracks. Tensile splitting tests or flexural tests would likely produce a larger difference between cracked and uncracked specimens.

[3] Rigden, S., Majlesi, Y. and Burley, E. (1995). "Investigation of factors influencing the expansive behavior, compressive strength and modulus of rupture of alkali-silica reactive concrete using laboratory mixes," Magazine of Concrete Research, 47 (170), 11-21.

Studies have also shown that reinforced concrete beams with ASR cracking demonstrate similar performance to uncracked beams; however, the ASR cracking was limited to the surface region of the concrete. Also, the nature of the reinforcement is likely to have a significant effect on the relative performance of cracked and uncracked specimens.

Where cracking occurs on the Sixth Street Bridge, it extends to the full depth of the member and creates a dense network of failure planes in the concrete. This will have some detrimental effect on the overall member stiffness, the shear strength of the concrete, and the toughness of the member. The shear strength provided by the steel reinforcement may also be affected due to the effect of cracking on the bond strength of the reinforcement. These factors should be considered when performing a structural evaluation or developing repair or retrofit designs for the viaduct.

Variability of Cracking and Compressive Strength along Core Length

For cores with overall distress ratings of moderate to severe, the degree of cracking varied along the core length. To quantify the variability of cracking along the length of the core samples, cores were visually graded for the degree of cracking along their length. The length of each core was divided into 6-inch segments and the cracking in each segment was given a rating from 1 to 5, with 1 indicating no cracking.

The rating of cracking in segments of the cores was based on observations from our field photographs. A rating of 1 was given to segments that appeared to have no visible cracking. A rating of 5 was given to segments that were rubble or had full depth cracks, making those segments non-testable for compressive strength. A rating of 4 was given to segments with a network of closely spaced cracks or a few wide cracks, but no full depth cracks. A rating of 3 was given to segments with moderate cracking. A rating of 2 was given to segments with light cracking or segments with no cracking that were adjacent to segments with a rating of 3 or more.

The last table in Appendix E shows the cracking ratings for segments along the core length for all of the cores. The table also shows the location along the core where the compressive specimen was taken and the cracking rating was given to the specimen. Since severely cracked cores had many segments that were too cracked to test and compressive specimens were taken from segments with less cracking (ratings of 3 or 4), the compressive strength results for severely cracked cores do not account for heavily cracked areas of the cores that were not testable. The compressive strengths for the different cracking ratings (from 1 to 4) for the specimens were grouped and various statistical values calculated. For cores with an overall distress rating of severe, the average strength of the specimens with a rating of 4 could be taken as the upper bound of strength, since many other segments of these cores were too cracked to test. For cores with an overall distress rating of severe, approximately 60% had less cracking (a rating of 4 or less) in the first 6 inches of the core and 40% had severe cracking (rating of 5) in the first 6 inches.

To determine the degree of cracking and compressive strength values for a column cross-section, the cracking ratings were reviewed for the long core samples (36 inches in length or greater). There were only six long column core samples. The degree of cracking along these cores varied but could be grouped into four types of patterns. Core C40 had mostly severe cracking for the entire length. Core C47 had severe cracking for approximately the first 18 inches and less severe cracking beyond 18 inches. Cores C11 and C50 each had a random pattern of severe and moderate cracking along their length. Cores C33 and C43 had moderate-severe cracking in the first 6 inches and severe cracking further along the core. Figures 40 to 43 show conceptual column cross-sections based on the cracking in these cores. The compressive strength values from these cores were taken from portions of the cores with crack ratings of 3 or 4; therefore, the strength values do not account for the areas of the core that were too cracked to test (crack rating of 5).

Possible Repair Methods

Since ASR appears to be active in the structure, keeping the concrete dry is the most effective means to address further ASR related deterioration. Several methods exist to protect the concrete from moisture, but none are likely to be reliable. Surface coatings have been tried in the past and were proven unsuccessful because it is difficult to adequately coat surfaces such as the top surfaces of bent caps and below-grade members. Also, surface coatings must be vigorously maintained or they can entrap water within the member, worsening conditions. Penetrating sealers are another possibility, but these too are difficult to effectively apply to all surfaces; additionally, they may not bridge new cracks that will form due to ASR from moisture already within the concrete. Penetrating (silane-based) sealers may be effective in reducing the rate of deterioration and could be used for interim service life extension. However, sealers will not stop the reaction and deterioration completely.

Providing a drainage system and improving the waterproofing of the deck, as well as waterproofing the deck joints, would reduce moisture exposure of the substructure and likely reduce ASR caused by deck runoff or water leaking through deck joints. These measures will not address ASR caused by soil moisture or rain exposure.

A possible repair for moderately cracked members involves removal of a minimum of 8 to 12 inches of the exterior concrete, and installation of new reinforcement and new high-quality concrete. This would improve water resistance and strength. Some members would need to be completely replaced. The expansion forces exerted by ASR can vary from 15,000 to 30,000 psi depending on the type and amount of reactive components, which would normally cause significant cracking along the direction of the stress, i.e., predominant longitudinal cracks on girders and columns. The repair design would need to account for allowance or restraint of future ASR expansion of the remaining concrete caused by moisture exposure associated with the repair work, as well as continued ASR due to existing moisture deep within the massive members. Additional investigations will be needed to quantify the potential for future ASR expansion and assess the structural requirements and costs involved for this repair.

Severely cracked members should be monitored and considered for replacement in the next five to ten years since the ASR is still active and continuing to deteriorate already heavily cracked members. Continued cracking will lead to failure of the members. Repair instead of replacement of the severely cracked members may be possible, but there are significant unknowns on how the severely cracked members would perform under seismic loading.

EFFECT OF ASR ON SEISMIC RETROFIT STRATEGY BASED ON IN-FILL WALLS

As mentioned herein, a seismic retrofit strategy alternative (Alternative 1 in the 2000 HBRR Study Report) was previously developed by the City of Los Angeles based on placing in-fill concrete shear walls between the three adjacent columns of a bent. Frequency of the in-fill panels vary, with an average of retrofitting every other bent on the structure east of the Los Angeles River, and retrofitting four bents in the twelve-span section west of the river. No retrofit was proposed for the Los Angeles River main span. The objective of the retrofit strategy was to provide stiffened elements in this continuous structure to reduce its seismic displacement, thus lowering the force demands on the existing columns and bent caps, and assuring overall stability of the structure. The discovery of moderate to severe structural cracks beneath the concrete surface for many of the core samples collected in the substructure bents (columns and bent caps) puts serious doubt on the functionality and structural viability of the proposed retrofit.

Of the five core samples collected from columns with severe surface distress west of the river (from abutment 1 to the West River Pier), all five cores showed severe cracking in the columns. Extensive cracking was observed throughout the length of these cores, which were typically 48 inches long. Some of the cracks were noted 40 inches below the concrete surface. These cracks, if projected vertically throughout the exterior columns, would likely significantly reduce the column ductility and flexural capacity. Considering that most of these bents are relatively short (less than 25 feet in height), the high shear stresses that are expected in the short columns would also render the deteriorated columns vulnerable to shear failure. It is clear that more in-fill concrete walls may be required on the west approach section of the Sixth Street Viaduct to reduce the column vulnerability.

The east river approach presents similar vulnerability. The structure in this section is significantly higher than the west approach. Average height of the bent columns in this stretch of the viaduct is over 55 feet. The surfaces between the in-fill walls and the columns are bonded with pairs of #6 dowels spaced at 6" on center. The dowels will be placed in drilled holes 12" into the existing concrete surface. The analysis of the in-fill wall can be simplified with a strut & tie model as illustrated in Figure 44. Based on an assumed value of lateral earthquake forces of 6,000 kips at the top of retrofitted bents, the tensile, or shear demand across the interface between the in-fill walls and existing columns will be in excess of 6,000 kips, or 100 kips of shear forces per foot of column height; this is equivalent to 175 psi shear stress across the plane that may contain severe cracks. Thus, it is our opinion that Alternatives 1 and 2 of constructing the in-fill walls alongside the damaged columns could extend and widen the cracks on these columns, seriously affecting the structure's ability to withstand lateral loads, or even gravity loads after an earthquake.

SUMMARY OF FINDINGS AND RECOMMENDATIONS

The results of the material testing program indicate that there is significant cracking due to alkali-silica reaction (ASR) throughout the Sixth Street Viaduct. Though the severity of cracking varies depending on location, cracking occurs along the entire length of the bridge. Cracking was observed in all bridge elements including the railings, the deck, the girders, the bent caps, the columns, and the foundations. Severe cracking due to ASR was observed in numerous bridge elements. ASR appears to be active and will likely continue to deteriorate the bridge. Severely cracked members should be considered for replacement in the next five to ten years.

Currently, there is no reliable method to arrest ASR deterioration. Protecting the structural members from further moisture infiltration would only slow the reactive process. A structure that suffers the level of damage from severe ASR as found on the Sixth Street Bridge should be considered for partial or total replacement. Seismic Retrofit Alternative 3[4] required partial replacement of the primary load-carrying members that contain the reactive aggregates, such as exterior bridge columns, bent caps, girders, and foundations. Of the secondary members that may have suffered light to moderate damage due to ASR, rehabilitation and strengthening may be possible. Alternatives of rehabilitation include improving the drainage condition on the deck to prevent water infiltration into the substructure, replacing the deck, and adding jackets to mildly damaged columns and bent caps to improve ductility.

Tests have shown that columns and beams with ASR distress generally exhibit an acceptable level of performance under static load. Based on our preliminary field investigation and visual inspection of the structure, the bridge appears structurally adequate to remain in service. There is no obvious visual evidence of significant stress cracks in girders and cap beams, which is expected if there is serious loss of gravity load-carrying capacity of the bridge girders and beams. However, it is recommended

[4] City of Los Angeles Bureau of Engineering, "HBRR Study Report For Sixth Street Bridge over Los Angeles River," April 2001.

that a load rating analysis be conducted to ensure the structure's ability to withstand the service load. Additionally, a structure-monitoring program should be considered on the girders that have experienced ASR-related cracks. The monitoring program would consist of strain and stress monitoring to provide in-situ proof of the structure's load-carrying capacity, and UPV testing to obtain quantitative data on the severity of internal cracking.

As found in many of the cored samples, large portions of the viaduct have extensive and damaging cracks. The cracking allows water penetration into the interior of concrete members. This moisture will continue to cause ASR reactions and cracking. This condition of cracking will lead to further deterioration of the structure, which will ultimately affect its serviceability. Due to the extensiveness of cracking found in some of the primary column members and bent caps, the structure's ability to withstand even a moderate level earthquake event is in doubt. A complete seismic risk study based on the condition of the structure as it currently exists should be undertaken immediately to assess the structure's vulnerability in a seismic event. It is also recommended that a probabilistic analysis be considered as part of the seismic risk study in order to assess seismic hazard, if the structure is to maintain its full service prior to the completion of the seismic retrofit program.

Based on the ASR damage to the columns and bent caps, the previously accepted retrofit strategy may not provide the intended purpose of safeguarding this structure from catastrophic failure in an earthquake. It is recommended that the City begin developing new seismic retrofit strategy alternatives that will consider the effect of material degradation and weakening of the structural components due to ASR. The development of a revised retrofit strategy may include replacing critical elements of the structure, as well as other possible strengthening alternatives that may include partial or complete replacement of the structure.