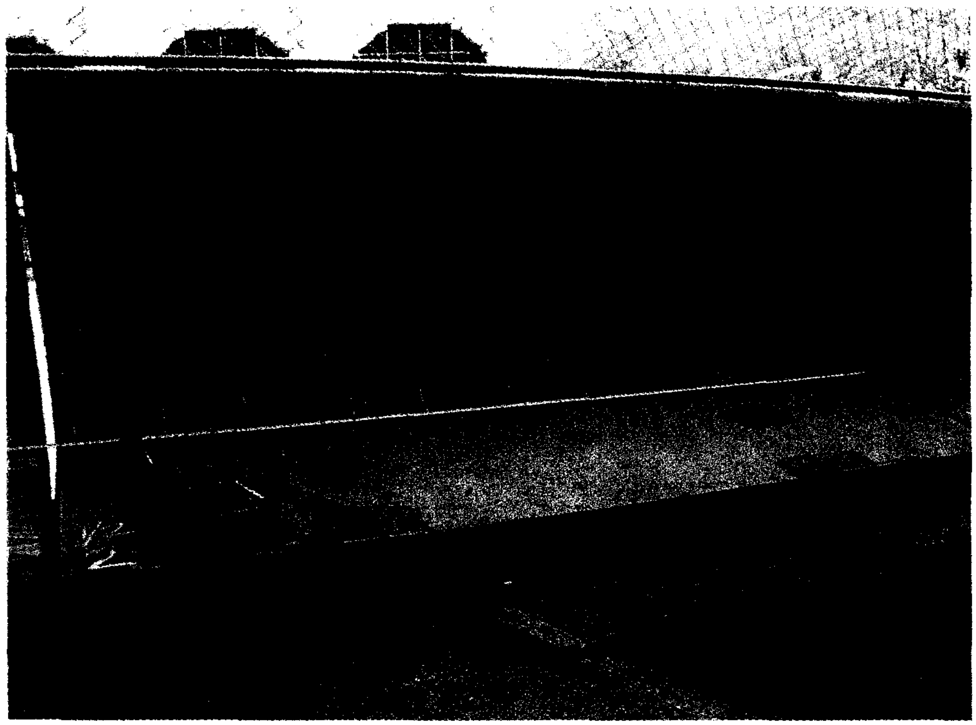
WJE

RIVER PARK SQUARE PARKING GARAGE
Structural Evaluation of Concrete Vehicle Barriers

Spokane, Washington



Final Report
18 July 2006
WJE No. 2006.2097

Prepared for:
Witherspoon, Kelley, Davenport & Toole

Prepared by:
Wiss, Janney, Elstner Associates, Inc.

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EXECUTIVE SUMMARY

At the request of Witherspoon, Kelley, Davenport & Toole, the firm of Wiss, Janney, Elstner Associates, Inc. (WJE) performed a condition assessment and structural evaluation of the precast concrete vehicle barriers at the River Park Square Parking Garage in Spokane, Washington. On 8 April 2006, a patron of the garage drove her vehicle into one of the barriers on the 5th floor of the garage. The barrier was unable to prevent the vehicle from falling off the north side of the garage resulting in the death of the driver. As a result of the accident, the Spokane Department of Building and Code Enforcement (City) issued a letter dated 3 May 2006 indicating that the owner of the garage must demonstrate to the City that "all relevant Code requirements are met."

To address this concern by the City, and to address safety concerns expressed by the building owner, WJE was asked to evaluate the condition and structural capacity of the remaining vehicle barriers in the garage. WJE performed a detailed condition survey of the vehicle barriers in the building including visual inspections, panel measurements, measurements of reinforcing steel, and concrete compressive strength testing. It was found that the concrete compressive strength of the panels generally exceeded the design requirements; however, the placement of the reinforcing steel within many of the panels varied significantly from the design.

The panels were analyzed for strength and compliance with applicable code ductility provisions, both of which are highly dependent upon the location of the reinforcing steel within the panels. Panel capacities were calculated assuming the entire panel width was effective in developing flexural strength. This assumption only appears to be valid for panels subjected to uniform loads or panels that were found to meet the American Concrete Institute (ACI) ductility provisions.

Our analyses found only six panels in the building that do not appear to have met the intent of the code provisions in effect at the time that they were constructed. These panels were constructed in 1999, when a 6,000 lb vehicle barrier load was part of the code. These panels should be upgraded.

All panels built in 1974 satisfied the intent of the applicable building code. Approximately half of the panels built in 1974 do not meet the ACI ductility provisions; however, our analysis found the panels have adequate strength to resist the factored code design loads in effect when the panels were constructed and therefore do not require repairs. The panel that failed in the recent accident was one that did not meet the ACI ductility provisions yet met the strength requirements in effect at the time it was built. Based on recent events, the owner may wish to consider upgrading those panels that do not meet the ACI ductility provisions.

Misplacement of reinforcing steel in reinforced concrete elements is a hidden defect that, barring notice from an inspector or other credible party who has knowledge of the problem, would not be known by the building owner. Prior reports in 1990 and 1993 did not identify the misplacement of the reinforcing steel in the panels.

INTRODUCTION

At the request of Witherspoon, Kelley, Davenport & Toole, the firm of Wiss, Janney, Elstner Associates, Inc. (WJE) performed a condition assessment and structural evaluation of the precast concrete vehicle barriers at the River Park Square Parking Garage in Spokane, Washington. On 8 April 2006, a patron of the garage drove her vehicle into one of the barriers on the 5th floor of the garage. The barrier was unable to prevent the vehicle from falling off the north side of the garage resulting in the death of the driver. As a result of the accident, the Spokane Department of Building and Code Enforcement (City) issued a letter dated 3 May 2006 indicating that the owner of the garage must demonstrate to the City that "all relevant Code requirements are met."

To address this concern by the City, and to address safety concerns expressed by the building owner, WJE has been asked to evaluate the condition and structural capacity of the remaining vehicle barriers in the garage. This report presents the findings of our investigation and provides recommendations for addressing structural concerns.

BACKGROUND

The River Park Square Parking Garage is a 10-story, reinforced concrete structure (Figure 1). The original portion of the garage was constructed in 1974 as an elevated 7-story structure with reinforced concrete columns, beams, and decks. Three additional stories were reportedly added to the structure around 1999 and a single below-grade level was added beneath the existing grade level retail space. Portions of the new elevated levels are of post-tensioned concrete construction. The building originally had vehicle barriers on both the north and south sides of the structure; however, the south barriers were replaced with walls in later remodels to the building.

The vehicle barriers are constructed of pre-cast reinforced concrete (Figures 2 and 3). They are L-shaped with 4 foot long horizontal and vertical legs curved about a 1 foot 5 inch radius. In discussions with Mr. Craig Lee, of Coffman Engineers, who designed the vehicle barriers for the 1999 addition, the barriers designed in 1999 were nearly the same as those originally installed as part of the 1974 construction. The only difference, according to Mr. Lee, was the manner in which the barriers were anchored to the deck.

Reportedly, in 1990 a vehicle impacted a barrier on the south side of the building. As a result of that incident, a structural investigation of the barriers was performed by Mr. Richard Atwood, P.E. of Atwood-Hinzman, Inc. in 1993.

In 2003, WJE performed a limited condition survey of the building at the request of Davis, Wright, Tremaine. This work was not specifically related to the condition of the vehicle barriers.

On 12 May 2006, WJE issued a letter titled *River Park Square Parking Garage, Evaluation of Vehicle Barriers - Preliminary Report of Findings*. In the letter, WJE provided preliminary findings of vehicle barrier capacities based on an analysis that relied upon limited available information and a number of assumptions. The findings in this report supersede those of our 12 May 2006 letter.

INVESTIGATION

Our evaluation of the existing vehicle barriers included a review of documents provided by your office, a field investigation, and structural analyses.

Document Review

The documents reviewed that are relative to our investigation included the following:

- Design drawings by John Graham and Company, 1973
- Spandrel panel detail drawings by Central Pre-Mix Prestress Co. from 1988, 1990, 1993, 1994, and 1999
- Report by Atwood-Hinzman, Inc. titled *Structural Inspection of River Park Square Parking Structure* dated June 1990
- Structural investigation report by Atwood-Hinzman, Inc. from 1993
- Windsor Probe test results of pre-cast concrete wall panels by Budinger & Associates, Inc. dated 21 July 1993
- Letter from City dated 3 May 2006

Field Investigation

Mr. Richard A. Dethlefs, P.E., S.E of WJE visited the site on 8 May 2006. During the visit, detailed measurements were taken of the remaining portion of the failed panel on the north side of the 5th floor. Other vehicle barriers on multiple levels of the building were visually inspected. We were informed that the vertical leg of the panel folded outward as a result of the vehicle impact and was left hanging on the side of the building by the reinforcing steel. The steel bars were later cut and the vertical leg of the panel was removed and is now stored at GeoEngineers, Inc. in Spokane, Washington. Mr. Dethlefs visited GeoEngineers, Inc. and took detailed measurements of this specimen.

Ms. Amy Woods and Mr. Michael Greenwood of WJE were on site from 30 May through 2 June 2006 and 6 June through 7 June 2006 to perform nondestructive testing and condition surveys of the vehicle barriers. Our field investigation of the panels included photo documentation, visual observations, measurements of reinforcing steel depth and spacing in the horizontal panel legs, and measurements of thickness of the horizontal panel legs when accessible. The firm of Budinger Inc. (Budinger) was hired to remove concrete cores from selected panels, and to perform concrete compression strength testing on selected cores.

Analysis

Structural analyses of the vehicle barrier panels were performed to check for structural capacity and conformance with various code provisions, including: the provisions in effect when the original 1974 portions were designed (i.e. the 1971 ACI-318), the provisions in effect when the 1999 addition was designed (i.e. the 1995 ACI-318) and the 2002 ACI-318 which represents the applicable structural provisions currently in force in Spokane. Our analyses included an assessment of both the 'as-designed' panels and the 'as-built' panels.

FINDINGS

Document Review

Following are descriptions of the information provided in the relevant documents that were reviewed as part of our investigation.

Design drawings by John Graham and Company, 1973

Architectural and structural design drawings from the original 1974 construction of the garage were reviewed. There were no details in the structural drawings for the pre-cast vehicle barriers. A detail on Sheet A7 of the drawings showed the dimensions of the vehicle barriers; however, details of the concrete strength, and size and placement of the reinforcing steel were not provided. Separately, we were provided with an 11x17 copy of a detail from Sheet A13 of the drawings that shows the reinforcing details of the panels. Sheet A13 is missing from the drawings provided. The detail calls for the panels to be reinforced with #6 primary bars (i.e. bars oriented perpendicular to the long dimension of the panel and curved to match the panel centerline radius) at 12 inches on center and #4 horizontal bars at 12 inches on center.

Detail drawings from Central Pre-Mix Prestress Co.

The drawings from Central Pre-Mix Prestress Co. are shop drawings for new and replacement vehicle barrier panels installed at the River Park Square garage throughout the years. It appears that in all instances, the panel orders were made by Robert B. Goebel - General Contractor Inc. (Goebel).

According to a drawing titled *Spandrel Panel* by Central Pre-Mix Prestress Co. dated 8 April 1999, the panels installed in 1999 are L-shaped with 4 foot long horizontal and vertical legs curved about a 1 foot 5 inch radius. The panels are 6 inches thick on the vertical leg and around the curve. The horizontal legs of the barriers are 5 inches thick. The panels are generally approximately 9 feet 9 inches long and are reinforced with #6 primary reinforcing steel bars at approximately 12 inches on center (11 bars total in 9 foot 9 inch panel length). Horizontal reinforcement is provided by #4 reinforcing bars spaced at 12 inches on center max. The top edge and toe edge of the panels are reinforced with two #6 horizontal reinforcing bars. The concrete is specified to have a 28-day compressive strength of f'c = 5,000 psi and reinforcing steel is to be ASTM A615 Grade 60. A copy of the panel detail is provided in Drawing 1 in Appendix A.

Atwood-Hinzman, Inc. report, June 1990

It appears that Atwood-Hinzman, Inc. (AHI) was asked to inspect the vehicle barrier panels in 1990 to address concerns of "longitudinal" cracking and panel displacement reported by Goebel. In a table titled "Riverpark Panel Problems," AHI identified 18 panels that were cracked and/or displaced. The displacements appeared to range from 1/4 inch to 1-1/2 inches. The table does not describe where the panel displacements were measured and does not provide a description of the cracking that was observed.

AHI concluded that the affected panels "have failed and should be replaced as we cannot determine the strength of the failed panels. At this time, the strength of the concrete in these panels has not been determined." AHI stated that test cylinders were being removed to determine the concrete compressive strength; however no test results were included in the report. AHI recommended removal and replacement of the affected panels, which were replaced later that year.

AHI also identified spalling on the top surface of the panels and deteriorated concrete at the roof due to weathering. AHI recommended repairing the spalled areas and sealing the concrete panels.

Budinger & Associates, Inc Windsor Probe test results, 21 July 1993

At the request of AHI, Budinger performed Windsor Probe tests of the concrete vehicle barrier panels. The Windsor Probe test uses a powder charge to force a pointed steel stud into the concrete. The test provides a rough indication of concrete compressive strength. Budinger performed the test at 15 panel locations. The results varied from 6,600 psi to 8,000 psi with an average of 7,326 psi. Budinger concluded that the concrete strengths indicated by the Windsor Probe were "well within the generally accepted range for the panels in question."

Atwood-Hinzman, Inc. report, 1993

As a result of a failure of a vehicle barrier due to auto impact in 1990, AHI performed an analysis of the vehicle barriers at the River Park Square garage. It appears that AHI analyzed the "as-designed" vehicle barrier and used the concrete strengths from Budinger's Windsor Probe tests in the analysis. AHI did not provide a description of the panel that failed in 1990 and it is unclear whether or not AHI ever inspected the failed panel. AHI assumed that the force of the code-required 6,000 lb impact load was spread over a 3 foot wide area. AHI concluded that the panels were "capable of resisting the force required by the Uniform Building Code."

AHI stated that their analysis assumed the primary reinforcing steel was in the center of the panel. They theorized that variances in the placement of the reinforcing steel may have accounted for the discrepancy between their analysis and the actual breakage.

In the report, AHI stated that a 3,000 lb auto traveling 30 mph would exert a force of 3,000 lbs on the panel which is one-half the load required by the U.B.C. AHI concluded that this is a good indication that the "loading is correct."

AHI concluded that the panels "are not resisting the required lateral loading of 6,000# although the engineering analysis indicates that they should."

AHI stated that when the affected panel was removed, the welded connection holding the panel in place "had deteriorated which also reduced the panel's ability to resist the impact of vehicles." AHI did not include photos or a description of the "deterioration" that was referred to. AHI did not appear to have analyzed the connection of the panel to the garage deck.

AHI concluded that "we have contradictory information, and an informed decision with respect to load capacity can't be made." AHI proposed two "solutions to the problem." They recommended removing and testing a panel to failure to establish load capacity or assume that the panels will fail and add steel cables to stop vehicles from impacting the panels.

Letter from City dated 3 May 2006

As a result of the accident, the City issued a letter dated 3 May 2006 indicating that the owner of the garage must demonstrate to the City that "all relevant Code requirements are met" in order to maintain the certificate of occupancy for the building.

Field Investigation

There are 268 vehicle barriers on the building. To facilitate our evaluation, locations and numerical designations were assigned to each panel as shown in Drawing 2 in Appendix A. Two hundred fifty-six of the panels are located on the north elevation of the building. Twelve barriers, six each on the 6th and 7th floors, are located on the south elevation. These panels are designated by hatched lines in Drawing 2. Panel designations include both the floor number and panel number as counted from the east end.

Panel Condition Survey

The panel condition survey included a visual inspection and photo documentation of each panel. Also included in the survey were thickness measurements of the horizontal panel legs (when accessible) and measurement of the depth to the primary reinforcing steel in the horizontal leg of all panels. Panel thickness measurements were taken when the front or rear side of a horizontal leg was exposed and at core locations. Attempts to measure panel thickness using a roto-hammer were unsuccessful. Reinforcing steel depth measurements were made using a Hilti Ferroscan meter.

Conditions such as cracking, spalling, scaling, out-of-plumb, and corrosion of embedded reinforcing steel were documented.

The findings of the panel condition survey are listed in a spreadsheet in Appendix B - Barrier Panel Field Data. The panels are identified by floor number and panel number and correspond to the panel designation numbers shown in Drawing 2, Appendix A. The spreadsheet includes visual observations, panel thickness measurements, depth of concrete cover of reinforcing steel (measured from the top surface of the horizontal legs of the panels), and spacing measurements of the primary reinforcing steel.

Photographic documentation of all panels is provided in Appendix C - Barrier Panel Photographs.

The following general conditions were observed:

Visual Observations.

a. Cracking was observed in some of the panels. Cracking typically was hairline in thickness and ran along the length of the barrier through the horizontal leg (Figure 4).
b. Spalling has occurred in several of the panels, typically at the toe edge of the horizontal leg (Figure 5). Patching was noted at some locations.
c. Surface scaling has occurred on the top surface of the horizontal legs of many of the barriers, typically on the 1974 barriers (Figure 6).
d. At several locations, the vertical legs of adjacent barriers were visibly out of alignment. This condition appeared to be the result of installation problems rather than post-installation deterioration or distress.

Concrete cover of reinforcing steel in the horizontal leg.

a. In most instances the depth of concrete cover over the primary reinforcing steel in the 1974 barriers exceeded 3 inches.
b. In general, the primary reinforcing steel in the 1999 barriers was located much closer to the position specified in the Central Pre-Mix shop drawing (i.e. centered 3 inches below the surface) than the 1974 barriers.

Barrier thickness in the horizontal leg.

a. In the 1974 panels, horizontal leg thickness varied from 4.84 inches to 5.25 inches with an average value of 5.08 inches.

b. Horizontal leg thickness in the replacement and 1999 panels varied from 5.0 to 5.3 inches with an average value of 5.13 inches.

Observations of failed panel at Floor 5/6.

The panel that failed in the recent accident was panel number 15 at Floor 5/6 (Figures 7 through 9). The failed panel was reinforced with #6 primary reinforcing steel bars at approximately 12 inches on center across the length of the panel. There were 11 bars total in the 9 foot 9 inch panel. The panel thickness of the vertical leg was 6 inches. The panel thickness of the horizontal leg was measured by Budinger at the three cores taken for compressive testing. The average thickness of the three cores was 5 inches. A horizontal #4 reinforcing steel bar was observed along the break line of the vertical leg of the panel. There was no evidence of significant corrosion of the reinforcing steel observed within the panel.

During our first inspection of this panel, the depth of concrete cover was measured to the top of each of the primary reinforcing bars using a tape measure. It was found that the average distance from the top (inside) face of the panels to the reinforcing steel was 3.28 inches. During our more recent survey of the panels, the depth of concrete cover to the vertical reinforcing steel was measured using the Ferroscan meter, which indicated the overall average depth of cover is 3.45 inches. The Ferroscan depth measurements were used in our analyses. Given the 3/4 inch diameter of the primary reinforcing steel, the average distance of the center of the reinforcing steel to the bottom (compression) face of the 5 inch thick panel was 1.18 inches.

No evidence of vertical cracking or other distress was observed in the vertical leg of the failed panel stored at GeoEngineers. In fact, short of rub marks and some apparent tire marks near the center of the failed panel, there was almost no indication at all in the upper 2-1/2 feet of the panel that it had been impacted.

The panel was removed using a roto-hammer, crow bars, and torches (Figure 10). Anchors of the horizontal leg of the precast barrier panel were visible after the mortar fill surrounding the anchors was chipped out. The two front angles and top two plates had light surface rusting visible on the metal. No cracking or displacement of the anchors or concrete was observed (Figures 11 and 12). Torches were used to remove the bolts and to cut the anchorage from the panel. A ridged mortar joint at the left edge of the panel base was still intact and showed no signs of cracking, indicating that the horizontal leg of the panel did not uplift as a result of the impact (Figures 13 and 14). The sealant joint at the right edge of the panel was cut out.

Concrete Compression Testing

Core locations were selected by WJE. Removal of concrete core samples and compression strength testing was performed by Budinger of Spokane, Washington. Representative core samples were removed for testing from each of the elevated decks, including portions of the garage constructed in 1974 and 1999. Seventeen cores (Sample Nos. 1 to 17) were removed and tested from random panels throughout the garage. An additional six cores were removed from the horizontal leg of the failed panel on Floor 5/6. Three of the cores from the failed panel (Sample Nos. 18a to 18c) were tested for compression strength by Budinger. The remaining three cores from this panel are in storage. Each core was 2-1/2 inches in diameter.

The compression strength results ranged from 5,720 psi to 8,370 psi. The average of the 20 cores tested was 6,658 psi. A summary of the compression results is provided below in Table 1. The full report from

Budinger on the compressive strength testing including details, documentation, photographs, and results is included in Appendix D - Concrete Compression Testing Results.

Table 1. Compressive Strength Results Summary.

Core ID	Floor Nos.	Panel No.	Compressive Strength (psi)
1	10/11	5	6760
2	9/10	2	7110
3	8/9	3	5980
4	8/9	16	6840
5	7/8	13	5880
6	7/8	30	7690
7	6/7	6	7230
8	6/7	18	7970
9	5/6	5	6660
10	5/6	29	6260
11	4/5	23	6580
12	4/5	11	6420
13	3/4	2	**8370**
14	3/4	23	6420
15	2/3	8	5890
16	2/3	19	6740
17	1/2	3	6040
18a	5/6	15	**5720**
18c	5/6	15	6490
18e	5/6	15	6110
Average			6658

Structural Analyses

Structural analyses of the vehicle barrier panels were performed to check for structural capacity and conformance with both current code requirements and the codes in effect at the time of construction. Our analyses included assessment of both 'as-designed' and 'as-built' conditions. The panels were assessed for both strength and ductility considerations. Details of this effort are provided in Appendix E - Barrier Panel Analysis.

Code Review

The 1997 Uniform Building Code (1997 UBC) was the governing model building code in effect at the time that the 1999 addition to the structure was constructed. The 2003 International Building Code (2003 IBC) is the current model building code adopted by the City of Spokane. In both of these codes, the structural requirements for the design of a vehicle barrier are covered in Chapter 16 of the code. The provision in the 2003 IBC reads as follows:

> ***Section 1607.7.3 Vehicle Barriers****. Vehicle barrier systems for passenger cars shall be designed to resist a single load of 6,000 pounds (26.70 kN) applied horizontally in any direction to the barrier system and shall have anchorage or attachment capable of*

> *transmitting this load to the structure. For design of the system, the load shall be assumed to act at a minimum height of 1 foot, 6 inches (457 mm) above the floor or ramp surface on an area not to exceed 1 square foot (305 mm2), and is not required to be assumed to act concurrently with any handrail or guard loadings specified in the preceding paragraphs of Section 1607.7.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.*

When the 1974 portion of the building was built, the 1970 Uniform Building Code (1970 UBC) was in effect. The 1970 UBC did not require consideration of any loading unique to vehicle barriers. Therefore, only the loads associated with actual exposure conditions (i.e. gravity and wind loads) needed to be accommodated. Except in rare instances, codes in effect at any particular time do not require structures designed, constructed and maintained in accordance with previous codes to be modified to address code changes. To our knowledge, the subject parking structure does not constitute an exception.

Analysis - General

The failed panel sustained a flexural failure at the base of the curved portion of the panel, where panel thickness decreases from 6 inches to 5 inches. This represents the critical location for combined gravity and vehicle impact loads.

The design of reinforced concrete is governed by the American Concrete Institute (ACI) 318 Building Code Requirements for Structural Concrete (ACI 318). Each of the editions of this code in effect at the time all of the various panels were designed included nearly identical provisions related to both strength and ductility. Spokane currently enforces an edition of ACI 318 that contains similar provisions.

The failed panel appeared to have been impacted near its center. No evidence of vertical cracking or other distress was observed in the vertical leg of the damaged panel. Given this impact location and the size and stiffness of the panel's vertical leg, the failed panel most likely mobilized the flexural strength of the entire critical cross section.

Our analyses assumed that in properly constructed panels, the entire cross section would be effective in resisting the applied bending moments. For panels in which the reinforcing steel was misplaced so as to reduce flexural ductility below the minimum value required by code, (i.e. where the reinforcement ratio exceeds 75 percent of the "balanced" value), the ability to mobilize available capacity to sustain the effects of concentrated loads is compromised, especially when the concentrated loads are applied near a panel's end.

Demand Calculations

The 2003 IBC and the code in effect at the time of the 1999 construction require a 6,000 pound vehicle impact live load to be applied at a distance 18 inches above the surface of the deck. Since there is a 6 inch curb at the toe edge of the panels, we assumed the load would act at 18 inches above the top surface of the horizontal leg of the panels. This resulted in the load being applied at a point 20.5 inches above the center of the critical section of the panel. The panels constructed in 1974 were not required to be designed to resist a concentrated vehicle impact load. The panels would have been required to resist gravity effects in combination with code specified wind loads.

As Designed Panel Analysis

The drawings that we have reviewed and our field examination efforts indicate that the 1974 and 1999 designs are essentially the same. According to the design documents, the primary reinforcing steel was to be centered 3 inches from the inside face. If the steel is so located, the distance from the centroid of the reinforcing steel to the outside face would be 2 inches. The concrete in the panels was specified to have a 28-day compression strength of 5,000 psi.

Assuming the entire panel width (9 feet 9 inches) acts to resist an applied impact force at 18 inches above the horizontal surface of the panel, the as-designed panel has the capacity to resist a *design* equivalent impact load of 11,800 pounds. The design impact load is the force calculated using the appropriate load and resistance factors for strength design of concrete per ACI-318. The panels at column locations, which have an effective panel width of approximately one foot less than the full length panels, have less design capacity than the full length panels but still satisfy the relevant code requirements.

The as-designed panel meets the applicable code ductility requirements.

As Built Panel Analysis

Twenty concrete cores were removed and tested for compression strength (f'c). Fifteen cores were removed from panels believed to have been built in 1974 and five cores were removed from panels believed to have been built in 1999. Using a 90 percent exclusion limit of the concrete strength data resulted in an f'c of 5,600 psi for the 1974 panels and 6,000 psi for the 1999 panels. These were the values used in the panel capacity analysis.

Panel thicknesses were measured when panel edges were accessible or when cores were removed. The panel thicknesses ranged from 5-1/4 inches to 4-7/8 inches. For panels where the thickness was specifically measured, the measured thickness was used in the panel analysis. Since applicable strength reduction factors were used in the evaluation and measurements indicated actual thicknesses were within construction tolerances, panels where thickness was not measured were analyzed using the nominal specified thickness of 5 inches.

For each panel, the area of primary reinforcing steel (per foot), the 'd' distance (distance from the centroid of the reinforcing steel to the extreme concrete compression fiber), reinforcing ratio, phi factor, nominal and factored moment capacities, dead and live load moment demands, and demand-to-capacity ratios were calculated. A check of each panel was performed to determine if the as-built panel satisfied the reinforcing ratio limit of 75 percent of balanced reinforcing ratio.

The demands were calculated for all panels, whether built in 1974 or 1999, assuming the current code-required impact load of 6,000 lbs acting 18 inches above the horizontal leg of the panel. Phi (strength reduction) factors, or resistance factors, which are used in strength design, were linearly interpolated between 0.65 and 0.90 in accordance with ACI 318 – 2002, Section 9.3.2 for calculation of the panel capacities. In calculating panel capacities, it was assumed that the entire panel width was effective in resisting the applied loads. This assumption appears reasonable for panels that meet the ACI reinforcing ratio limits.

The ability of critical as-built conditions in the population of 1974 panels to sustain applicable gravity and wind loads was also assessed. This evaluation included an appropriate reduction in the phi factor to account for the fact that these as-built conditions did not provide the minimum specified degree of ductility.

The results of the analyses of the as-built panels are summarized in Appendix E - Barrier Panel Analysis.

CONCLUSIONS AND DISCUSSION

Prior Reports

The AHI reports of 1990 and 1993 did not identify the misplacement of the reinforcing steel in the panels. Their only mention of this issue comprised speculation about issues they did not pursue.

The AHI reports in fact did not make any conclusions regarding why the panel in 1990 failed. Instead of focusing their analysis on the actual panel that failed, AHI analyzed the panel design, which based on our recent findings, obviously varies significantly from the as-built panel conditions. AHI also erroneously concluded that a 3,000 lb vehicle traveling at 30 mph will impart an impact load of 3,000 lbs, thereby concluding that the code-required vehicle impact load of 6,000 lbs is a reasonable load. In fact, a 3,000 lb car traveling at 30 mph will impart significantly more force than 3,000 lbs on an at-rest object, especially if that object is rigid and brings the vehicle to a stop in a short distance, which is the intent of a vehicle barrier. AHI did not consider that the load imparted by a moving vehicle may actually be in excess of 6,000 lbs. In short, the AHI report did not provide the building owner with a clear direction for future assessment of the remaining vehicle barriers.

General Observations

In general, the panels throughout the garage appeared to be in good condition. The panels did not appear to be suffering from lack of maintenance or significant deterioration or corrosion. We did not observe any evidence of deterioration of the connection points of the panels to the concrete decks. In addition, there was no evidence of any type of failure related to the connection points in the recent accident.

At selected locations throughout the building, there was evidence of corroding horizontal reinforcing steel in the horizontal leg of the panels near the toe. This condition was evident in the failed panel as well. It is likely that this condition is caused by vehicles transporting de-icing salts into the garage during the winter and dripping the road salts onto the toe edge of the panels while parked. Moderate corrosion of reinforcing steel at this location has no impact on the capacity of the panels.

Light scaling was observed on the top surface of the horizontal leg of many of the panels. This condition is likely the result of moderate freeze-thaw exposure and to date has had no impact on the capacity of affected panels.

As-Designed Panel Analysis

As designed, both the 1974 and 1999 panels appear to meet all of the relative code requirements of the current code and the codes that were in effect at the time the panels were built.

As-Built Panel Analysis

1974 Panels

The 1974 panel design was code-compliant. However, construction of these panels often deviated significantly from design requirements. As a result, many of the 1974 panels do not satisfy associated ACI 318 requirements for ductility. The panels that do not satisfy the ACI ductility provisions are identified by the notation "X" in the final column of our analysis spreadsheet in Appendix E. In order to assess the significance of these nominal code deficiencies, reduced phi factors consistent with low ductility were implemented in our evaluation. In all cases, the reduced as-built strength remained sufficient to sustain the factored 1974 design loads.

In order to provide a certain degree of ductility, the applicable ACI code did not permit flexural member reinforcement ratios to exceed 75 percent of the balanced condition. Therefore, where misplacement of rebar violates this provision, an argument can be made that the minimum level of ductility was not provided. However, to our knowledge, the applicable 1974 Spokane building code provisions did not prohibit the use of low ductility elements (e.g. unreinforced masonry walls) as garage parapets. Therefore, evaluation of the as-built barrier elements using phi factors consistent with low ductility performance is valid, and in this case indicated that all as-built 1974 panels satisfied the intent of the applicable building code

1999 Panels

In 1999, a 6000 lb vehicle barrier load was part of the applicable code. Therefore, where panel strength is insufficient to handle gravity and vehicle loads, retrofit is required regardless of whether the deficiency is due to construction or design error. As with the code in effect in 1974, minimum ductility was required in the 1999 panels.

None of the ACI codes relevant to this project allow reinforcement ratios in excess of the limiting values for flexural members. Therefore, where the limiting reinforcement ratio is exceeded in a flexural member, ACI considers that member insufficiently ductile. However, as discussed above for the 1974 panels, a reduction in ductility does not necessarily comprise a true deficiency. If the affected panel has sufficient strength to accommodate factored loads after application of a low-ductility phi factor, it may satisfy code intent.

In the case of the 1999 panels, which are required to resist concentrated vehicle impact loads, it is unlikely that a panel failing to meet the minimum ACI ductility requirements would be able to mobilize the entire cross section of the panel to resist concentrated impact loads, especially when such loads are applied near a panel's end. In other words, if the vehicle impact load is applied near the end of a panel that does not meet the ACI ductility requirements, its ability to mobilize available flexural strength is severely limited and highly variable.

The capacities calculated in our analysis spreadsheet in Appendix E assume that the entire width of the panel is effective in resisting the applied loads. It is our opinion that this assumption is only valid for those panels that meet the ACI ductility provisions.

The six 1999 panels that do not meet the ductility provisions of the code are identified in the analysis spreadsheet in Appendix E. They are located on floors 7/8 and 8/9.

Upgrade Considerations

In our opinion, the 1974 panels are not required to be repaired or upgraded. This is because they remain capable of sustaining applicable design loads. However, since construction errors in many of the 1974 panels left them incapable of sustaining vehicle impact loads that are now considered realistic, ownership may want to consider upgrading these elements. The panels that fall into this category are those that were constructed in 1974 and do not meet the ductility provisions of the ACI code which represents approximately half of the two hundred-two 1974 panels.

All 6 of the 1999 panels that do not meet the ductility requirements should be upgraded. These elements are not capable of reliably sustaining the combined gravity and impact loading specified in the applicable code. The remaining 1999 panels are adequate by both contemporary and current standards.

At your request, we are currently working on a retrofit design to bring all of the vehicle barrier panels in the building into conformance with currently applicable structural standards.

Summary

Panel capacities were calculated assuming the entire panel width was effective in developing flexural strength. This assumption only appears to be valid for panels subjected to uniform loads or panels that were found to meet the ACI ductility provisions. The ability of panels not meeting the ACI ductility provisions to mobilize available flexural strength is severely limited and highly variable.

Our analyses found only six panels in the building that do not appear to have met the intent of the code provisions in effect at the time that they were constructed. These panels were constructed in 1999, when a 6,000 lb vehicle barrier load was part of the code. These panels should be upgraded.

Approximately half of the panels built in 1974 do not meet the ACI ductility provisions; however, the panels were calculated to have adequate strength to resist the factored design loads in effect when the panels were constructed and therefore do not require repairs. The panel that failed in the recent accident was one that did not meet the ACI ductility provisions. Based on recent events, the owner may wish to consider upgrading those panels that do not meet the ACI ductility provisions.

Misplacement of reinforcing steel in reinforced concrete elements is a hidden defect that, barring notice from an inspector or other credible party who has knowledge of the problem, would not be known by the building owner. The AHI reports in 1990 and 1993 did not identify the misplacement of the reinforcing steel in the panels. Their only mention of this issue comprised speculation about issues they did not pursue.