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May 12, 2006

Milt Rowland
Assistant City Attorney
808 West Spokane Falls Boulevard
5th Floor
Spokane, WA 99201

Re: RPS Parking Garage

Dear Milt:

We have been asked to respond to a letter dated May 3, 2005 (2006) directed to Mr. Bob Smith at River Park Square by Joseph L. Wizner, Director of Building Services, with respect to the April 8, 2006 accident at the RPS Parking Garage.

We assume, from your letter, that the meeting, which you set for Wednesday, May 17, 2006 at 1:30 p.m., is not a formal hearing required under SMC 17F.070.440 in that Mr. Wizner, as building official, has not made any findings of any violations of Chapter SMC 17F-070, nor has he prepared a written complaint, all of which are required by SMC 17F.070.420. Rather, we assume that the meeting scheduled for May 17 is to provide the City with the information requested by Mr. Wizner. Accordingly, we reserve all applicable rights should, subsequent to the meeting of May 17, the City wish to commence a formal procedure pursuant to SMC 17F.070.420.

With that in mind, we want to assure you that we do intend to cooperate in all respects, and look forward to providing the available relevant engineering information at the May 17 meeting. We also expect our engineers will discuss with Mr. Wizner at that time the investigation conducted to date and the initial conclusions reached. Please be advised that we intend to work with the City to ensure that the RPS Garage meets or exceeds all applicable building codes. Careful inspection and analysis of the parking garage is as important to RPS as it is to the City.

As I mentioned, River Park Square, its engineers and contractor have worked quickly and diligently to investigate and study the garage to be able to assure the City and the public of the garage's safety and compliance with all applicable codes. It started 45 minutes after the accident when Goebel's superintendent arrived on the scene to secure the site. On Monday, April 10th at 8:00 a.m., Terry Goebel arrived at the garage and began his inspection.

He inspected the garage, contacted the structural engineer, and began pulling engineering drawings, architectural plans, files and reports. Thereafter, on April 12th, Mr. Goebel and Craig Lee, a structural engineer of Coffman Engineering, met to further study the matter. Later that day, Mr. Lee inspected the concrete barrier (which was stored off site) and the damaged automobile. The very next day, Mr. Lee spent 3.5 hours inspecting the garage and specifically the concrete panels or barriers. On April 20th, Mr. Lee spent another 4 hours measuring the concrete panels at the garage and did an engineering analysis of the panels. This is in addition to the work performed by other structural engineers retained by Safeco, the garage's insurance carrier.

Also, River Park Square hired the Seattle office of a national consulting engineering firm, Wiss, Janney, Elstner Associates of Northbrook, Illinois to conduct an independent structural inspection of the garage to determine its compliance with all applicable codes. Mr. Richard Dethlefs of that firm has inspected the garage and the concrete panel in question, as well as reviewed engineering and shop drawings of the panels, and documented his initial findings with regard to the condition of the vehicle barriers. Enclosed is a copy of Wiss, Janney's preliminary report. As you will note, Wiss, Janney has found that the garage panels meet or exceed applicable building codes.

As stated in its preliminary report, Wiss, Janney has been asked by RPS, in order to ensure the continuing safety and viability of the garage, to perform additional studies. When this work is done and the engineering firm completes its comprehensive report, together with recommendations for voluntary upgrades, naturally we will timely share them and our implementation plans with the City.

Again, we look forward to working with the City in this matter.

Very truly yours,

WITHERSPOON, KELLEY, DAVENPORT
& TOOLE

William D. Symmes

WDS:jlf
Enclosure

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Via Fax and U.S. Mail

12 May 2006

Mr. William D. Symmes
Witherspoon, Kelley, Davenport & Toole
1100 U.S. Bank Building
422 West Riverside Avenue
Spokane, WA 99201-0300

Re: River Park Square Parking Garage
Evaluation of Vehicle Barriers - Preliminary Report of Findings
WJE No. 2006.2097

Dear Mr. Symmes:

At your request, Wiss, Janney, Elstner Associates, Inc. (WJE) performed an evaluation of the vehicle barriers at the River Park Square Parking Garage in Spokane, Washington. On 8 April 2006, a patron of the garage drove their vehicle into one of the barriers on the 5th floor of the garage. The barrier failed allowing the vehicle to fall 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" in order to maintain the certificate of occupancy for the building. To address this concern by the City, WJE has been asked to evaluate the condition of the vehicle barriers in the garage. The purpose of this letter is to document our initial findings regarding the condition and capacity of the vehicle barriers at the building. Currently, we have been asked to plan on performing additional investigation into the condition of the vehicle barriers and will issue a final report with recommendations when we are completed.

Background

The River Park Square Parking Garage is a 10-story, reinforced concrete structure. 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 precast reinforced concrete. They 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 9 feet 9 inches long. According to a drawing titled *Spandrel Panel* by Central Pre-Mix Prestress Co. dated 8 April 1999, the panels installed in 1999 are reinforced with #6 longitudinal 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. 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.

Investigation

As part of our evaluation of the existing vehicle barriers, we have visited the garage and reviewed the documentation provided by your office. The documents reviewed included the following:

- Four spandrel panel detail drawings by Central Pre-Mix Prestress Co. dated 8 April 1999
- Structural investigation report by Atwood-Hinzman, Inc. from 1993
- Letter from City dated 3 May 2006
- Complaint to Abate Public Nuisance by Spokane Research & Defense Fund, a Washington non-profit corporation.

During our site visit on 8 May 2006, we visually inspected the vehicle barriers on multiple levels of the building. Detailed measurements were taken of the failed panel on the north side of the 5th floor. It is our understanding 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. We visited GeoEngineers, Inc. and took detailed measurements of the vertical leg of the panel.

Findings

It was found that the failed panel was reinforced with vertical #6 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 legs was 6 inches. The panel thickness of the horizontal leg of the panel adjacent to the failed panel was 5-1/4 inches. We were unable to get a thickness measurement on the horizontal leg of the failed panel due to limited access. A horizontal #4 reinforcing steel bar was observed along the break line of the vertical leg of the panel. There was no evidence of corrosion of the reinforcing steel observed within the panel.

The depth of concrete cover was measured to the top of each of the vertical reinforcing steel bars. It was found that the average distance from the top inside (tension) face of the panels to the reinforcing steel was

3.28 inches. Accounting for the 3/4 inch diameter of the reinforcing steel, this resulted in the average distance of the center of the reinforcing steel to the outer (compression) face of the 5-1/4 inch thick panel of 1.60 inches. If other panels are only 5 inches thick, as shown in the design drawings, the distance from the compression face of the concrete to the center of the reinforcing would be 1.35 inches.

While at the site, we used a Proceq Profometer rebar locator to measure the depth of the reinforcing steel in the panels immediately adjacent to the failed panel. It was found that in two of the three panels, the vertical reinforcing steel appeared to be located at approximately the neutral axis. We were unable to determine the depth of the reinforcing steel in the third panel.

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, there did not appear to be provisions in the code that governed the design of vehicle barriers.

Analysis

We have analyzed the vehicle barrier panels to check their conformance with the 2003 IBC requirements. Our analysis includes an assessment of both the 'as-designed' panel and the 'as-built' panel. For the as-built condition, our analysis is reliant upon our measurements taken of the failed panel where we were able to take accurate measurements of the steel locations in the panel.

As Designed. The as-designed panel has the vertical reinforcing steel centered in the precast panels at the neutral axis of the panel. If the steel is placed at the neutral axis, the 'd,' or distance from the centroid of the reinforcing steel to the extreme compression surface of the panel is 2.5 inches. The concrete in the panels (1999 version) was specified to have a 28-day compression strength (f'c) of 5,000 psi.

The code requires the 6,000 pound load be applied at a distance 18 inches above the surface of the deck; however, 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 neutral axis of the critical section of the panel.

Assuming the entire panel width (9 feet 9 inches) acts to resist the applied force, the as-designed panel has a *design* capacity of 17,600 pounds. The design capacity is the capacity calculated using the appropriate load and resistance factors for strength design of concrete per the American Concrete Institute (ACI - 318), the governing code for design of structural concrete.

As Built. As described, the longitudinal reinforcing steel within the panel that failed was not placed at the neutral axis. The reinforcing was actually located more toward that compression surface of the panel. In our analysis of the as-built panel, we used an average distance from the center of the reinforcing to the extreme compression surface of the panel of 1.35 inches. We did not utilize the actual panel thickness measured at the adjacent panel of 5.25 inches because there may be instances in the garage where panels are not that thick. In addition, our limited measurements of rebar depth in the panels adjacent to the failed panel indicate that at least some of the panels have better placement of the reinforcing steel and will have capacities in excess of that which we have calculated.

Assuming the entire panel width (9 feet 9 inches) acts to resist the applied force, the as-built panel has a *design* capacity of 8,400 pounds. Again, the design capacity is the capacity calculated using the appropriate load and resistance factors for strength design of concrete per the American Concrete Institute (ACI - 318), the governing code for design of structural concrete.

Conclusions and Discussion

There may be some discussion as to how much of the width of the panels are actually appropriate to use in the capacity analysis of the panels. For instance, if the design load is applied to the panel in a 1 foot square area as required by the code, and the forces are transmitted downward into the panel at approximately 45 degrees, that would result in 4 feet (12 + 18 + 18 inches = 4 feet) of the panel section acting to resist the applied force. However, we believe that this assumption would dramatically underestimate the actual capacity of the panels. In fact, concrete is very good at distributing shear; and since the panels are reinforced with horizontal reinforcement, there should be almost no shear lag in distributing the applied force into the entire panel width. We cannot foresee any benefit to a yield-line theory analysis of the panel, since we can see by the failed panel that the method of failure for the panel was bending at the 5 inch cross section at the base of the panel curve. In addition, no evidence of vertical cracking or other distress was observed in the vertical leg of the 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. As a result, it appears that the panel was more than able to distribute the applied load across the entire width of the panel so that the entire cross section could act to resist the applied loads.

Our analysis of the as-designed panels found that they have a demand-to-capacity ratio of 0.34. Our analysis of the as-built panel (based on measurements of the failed panel) found that it had a demand-to-capacity ratio of 0.71. The placement of the reinforcing steel away from the neutral axis of the panel resulted in a loss in capacity of 52 percent; however, the as-built panel used in our analysis should still have adequate capacity to resist the design live load of 6,000 pounds applied 18 inches above the top surface of the horizontal leg. As a result, the as-designed and as-built panels appear to meet the requirements of the current 2003 IBC code.

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According to our analysis, the as-built panel would have to be able to mobilize 7 feet of panel width to meet the code required live load of 6,000 pounds at 18 inches. The panel will mobilize 7 feet of panel width if the load is applied at least 3 feet from a panel edge and the load is able to distribute downward into the panel at an angle of 63 degrees from vertical. The angle of 63 degrees appears to be reasonable and we believe that a load applied near the center of the panel would actually mobilize the entire panel width.

The panels were checked for punching shear and were found to have more than adequate capacity for this failure mode.

There is a possibility that a panel impacted near one end would have a triangular failure at the corner of the panel. This type of failure would likely not be catastrophic as it would not allow a car to pass over the panel.

In general, the panels throughout the garage appeared to be in good condition. We did not observe any evidence of deterioration of the connection points of the panels to the concrete decks. 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 will have no impact on the capacity of the panels but would be a maintenance and aesthetic concern.

We have assumed a concrete compressive strength of 5,000 psi in all of our analyses and a steel yield stress of 60,000 psi. It is likely that the actual steel yield stress will be in excess of 60,000 psi. It is also possible that the concrete strength may be in excess of the 5,000 psi specified strength; however, to confirm this, we recommend removal of concrete core samples from representative panels in the garage for compressive strength testing.

Summary and Recommendations

The as-designed panels meet the current code requirements for strength, even though the panels constructed in 1974 do not appear to have been governed by a known code requirement at the time of construction. The placement of the reinforcing steel within the failed panel has resulted in that panel having less capacity than was intended in the original design; however, based on our analysis, the panel still appears to meet the current code requirements for strength. For the purposes of this evaluation, we have taken the construction of the failed panel to be representative of the remaining panels within the building. Based on the rebar detector measurements of the panels adjacent to the failed panel, it appears that the placement of the reinforcing steel is better in some panels than the failed panel.

Although the panels appear to meet the strength requirements of the governing codes, it appears that vehicles are able to impart more than 6,000 pounds of force on the barriers.

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It is our opinion that the vehicle barriers in the River Park Square Parking Garage are in general conformance with the current code requirements for vehicle barriers. The panels do not appear to be suffering from lack of maintenance or significant deterioration or corrosion. The vehicle barriers likely provide a similar level of safety as many of the other parking structures in and around Spokane and throughout the state of Washington.

Very truly yours,

WISS, JANNEY, ELSTNER ASSOCIATES, INC.

Richard A. Dethlefs, P.E., S.E.
Project Manager

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