

Starbucks Center Renovation

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ABSTRACT

When Starbucks made a commitment to lease up to four large floors of warehouse space in Seattle's SODO Center and convert it to the company's corporate headquarters, the Seattle Building Code mandated a life-safety upgrade of the massive central building. Additional seismic upgrades were completed on the adjacent north and south buildings. This paper will discuss the resulting structural evaluations and seismic upgrades of the facility, which was renamed the Starbucks Center.

BUILDING HISTORY AND CURRENT USE

Starbucks Center is a multi-building complex constructed in six phases—1912, 1914, 1945, 1954, 1965, and 1974—and served as the Sears catalog distribution center for the western United States. Upon completion of the second phase in 1915, the building was the largest structure west of the Mississippi River. At nearly 2,000,000 SF, it is today the largest building in the City of Seattle. Sears continues to operate a retail store in the complex, but the distribution facility ceased operations in the late 1980's.



Figure 1: Starbucks Center

The present owners purchased the property from Sears in 1990, saving it from demolition, a fate suffered by two other Sears distribution facilities in Kansas City and Philadelphia. Current uses include corporate office, retail, light manufacturing, and storage in the main building; retail and storage in the south building; and parking in the north building. Starbucks has become the largest tenant, moving its corporate headquarters into the facility over the past few years. Accordingly, the building has been given a facelift and renamed Starbucks Center (see Figure 1).

The main, central building is a nine-story cast-in-place concrete structure, built in several phases since 1914. While the construction varies slightly from phase to phase, it generally consists of circular columns, capitals, drop panels, and flat slabs. The structure has a partial basement and is founded on more than 10,000 timber piles. The original south building is a six-story heavy timber framed structure with perimeter concrete frames and walls. Finally, the newer north building is a modern seven-story cast-in-place concrete structure with two-way flat slab floors, circular columns, and perimeter bearing walls.

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SEISMIC UPGRADE TRIGGERS

Starbucks began leasing and converting a large portion of the main building in the early 1990's. However, before Starbucks could move into the building, the owner had to be mindful of the substantial alteration provisions of the Seattle Building Code. The code considers a project to be a substantial alteration if any of the following occurs:

- Extensive structural repair.
- Remodeling which substantially extends the useful physical or economic life of the building.
- A change of a significant portion of a building to an occupancy that is more hazardous.
- Re-occupancy of a building that has been vacant for over 12 months.
- A significant increase in the occupant load of an unreinforced masonry building.

The change of occupancy from warehouse to corporate office space during the Starbucks Center renovation caused the main building to qualify as a substantial alteration.

The use of the original building has essentially not changed and therefore the City of Seattle has not required any life-safety improvements. A seismic upgrade of this building, however, occurred in association with lending requirements for the overall complex. This building was found to have some potential for partial collapse and therefore mitigation was required to satisfy the lender's criteria.

The north building was converted from warehouse into a parking garage. As part of this conversion, two stories were added. The gravity system of the building was adequate due to the reduction in live loads, however, the seismic weight increased substantially. The City of Seattle then mandated fairly stringent seismic requirements.

SEISMIC EVALUATION AND UPGRADE OF THE MAIN BUILDING

If a renovation is deemed a substantial alteration, the seismic provisions of the current building code, or an approved standard, are enforced. Currently, the City of Seattle accepts FEMA-178 (BSSC 1992) as an approved standard. The seismic performance objective of FEMA-178 is life safety.

Due to the complexity of the structure, the FEMA-178 evaluation became an involved process. A three-dimensional, linear dynamic analysis of the entire structure was performed. The computer model included all of the building columns and floor slabs. The slabs were modeled as frame members using the effective beam width approach. The typical reinforcing layout leads to premature hinging within the bay, thereby softening the frame. The effects of hinging were studied in a simplified model, in which it was determined that an effective width factor of 1/6 adequately approximated the ultimate condition.

Additional refinements in the basic elastic model included a 25% reduction of the beam/column joint's rigidity and a drop panel cracking factor of 70%. Due to their size and amount of reinforcement, the columns were modeled with gross section properties. These assumptions were validated in the analysis.

From preliminary analysis it was fairly clear that the columns and slabs were not shear critical. Therefore, the response modification factor, R , was taken as 4 (intermediate concrete moment frame) for evaluation of the unretrofitted structure. The deflection amplification factor, C_d , was assumed to be 3.5. Simplified soil-structure interaction effects were included by modeling the lateral stiffness of the piles.

The results of the evaluation indicated a lack of strength in the lateral system primarily due to the slab's low bending capacity. The analysis indicated that hinges would form at the slab/drop panel interface at fairly low levels of lateral load. This hinging limited the lateral strength of the structure to less than that recommended by FEMA-178. Furthermore, it softened the structure to a point where excessive drift, P-delta effects, and ultimately slab punching shear were concerns for the given displacement demands. The columns, however, had adequate shear and bending capacity.

Upgrade Options

Due to the size and complexity of the renovation project, several seismic upgrade schemes had been studied for several years prior to the development of the final concept. Most relied on the addition of significant strength to the building by way of a new, rigid lateral system. The final concept, however, was based on supplementing the existing structure with a flexible lateral system.

The existing structure had a relatively high building period with deficiencies in strength, ductility, and stiffness. The previous approaches had corrected these deficiencies by adding very strong lateral elements, such as shear walls. These systems were so stiff, however, that the building period lowered, greatly increasing earthquake demand. This high demand translated directly into high retrofit costs and difficult construction.

In contrast, the final concept attempted to keep the structure relatively flexible, thus limiting the increase in earthquake force demand. While both approaches significantly reduced drift from the existing condition, the compliant system actually produced lower inelastic deflections than the stiff system due to lower spectral demand.

Once the feasibility of the flexible concept was confirmed, the detailed design of the upgrade was completed. The specific objectives of the upgrade were as follows:

- Provide the lateral strength specified by FEMA-178.
- Provide reliable ductility.
- Limit inelastic drift to 2% to preclude punching shear failures.
- Configure the new system to be compatible with the existing structure at elastic and inelastic drift levels.
- Mitigate the possibility of uplift on the timber piles due to overturning from new elements.
- Do not induce torsion in the structure.

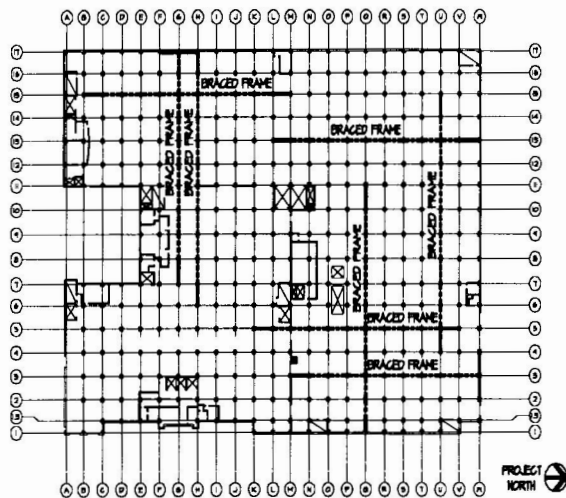


Figure 2: Plan Layout

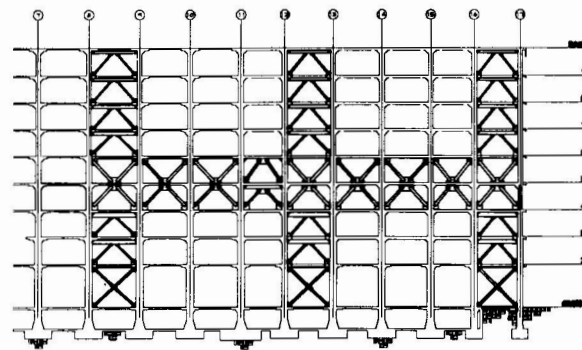


Figure 3: Frame Elevation

Given these objectives, a new system was developed. The basic system consists of twelve steel, eccentrically braced frame (EBF) towers. These towers are interconnected with two-story trusses to form frames. The trusses were provided to limit overturning forces on the individual towers. A plan view showing frame locations is provided in Figure 2; an elevation of a typical frame is shown in Figure 3.

The new system takes advantage of the existing structure's most positive components—the spirally reinforced, concrete columns. The change of use from heavy warehouse to corporate office space over most floors in the building increased the column's reserve capacity. The reserve capacity in these relatively ductile elements was utilized in the retrofit design.

The link beams in the EBFs were sized for stiffness. The braces were also sized for stiffness and for a minimum ultimate strength of 125% of the force that can be delivered by the EBF links. The truss members were sized to remain elastic and provide overturning stability to the system.

Results of Elastic Analysis

The new braced frames were added to the original building model and an elastic response spectrum analysis was completed using the ground motions described earlier. The analysis was based on an R factor of 6 for the combined system and a C_d factor of 3.5. The results of the analysis were as follows.

- The fundamental modes were translational with periods of approximately 1.7 sec in both primary directions. The ultimate lateral deflection at the roof level was 4 inches and the ultimate story drifts were all less than 0.6%.
- The slabs have limited flexural capacity. The various slab types were analyzed and the flexural capacities were determined along the length of the slab. Existing dead loads serve to preload the slab, and the flexural capacity

curves were modified accordingly. The modified flexural capacities at the positive and negative weak points were then summed and divided by the distance between them to obtain the shear that would cause the slabs to develop a mechanism. This shear was then taken as the maximum allowable shear for the member. It was confirmed that the slab/beams had capacities greater than the elastic demands.

- The strength of the existing columns in the braced frames was checked for axial tension and compression. This check in the response spectrum analysis was somewhat academic because the ultimate capacity was later verified in the nonlinear, pushover analysis. The remaining columns were checked for flexure, and elastic demands were found to be 15% to 25% of capacity. Finally, ultimate shear demand was found to be less than 50% of the column's capacity in all cases. Shear stress in a typical column is shown in Figure 4.
- The slabs and drop panels were evaluated by taking their dead load plus 50% of their live load to obtain a gravity-load shear stress. The largest shear stress was approximately 2/3 of the maximum recommended for flat slab/frame structures subject to 1.5% ultimate drift.
- As mentioned above, the EBF beams were sized for stiffness. The shear links had ultimate rotations of less than 0.06 radians. When subjected to the design response spectrum forces, the braces and truss members remained within their elastic limits. All new members and connections were designed and detailed according to the provisions of the current Uniform Building Code.

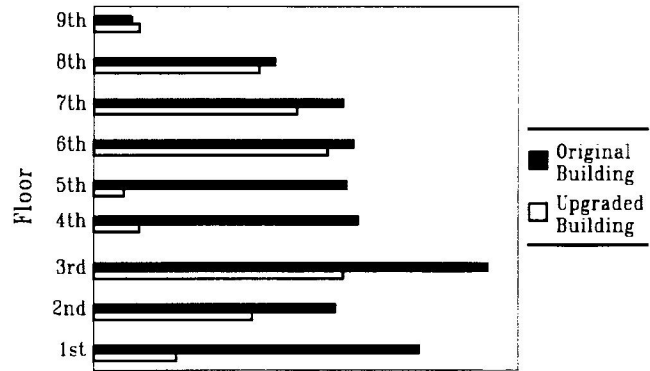


Figure 4: Column Shear

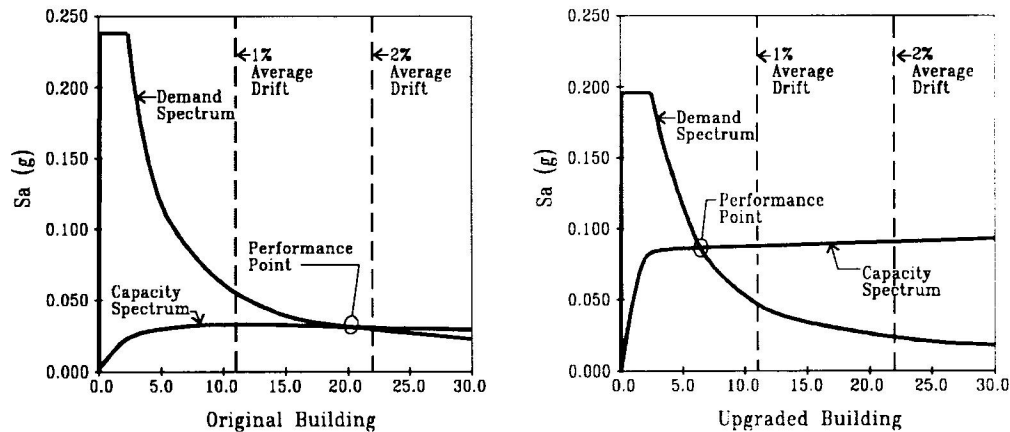


Figure 5: Pushover Results

Nonlinear Pushover Analysis

Due to the size and complexity of the seismic upgrade, a nonlinear static pushover analysis was performed. The objectives of the analysis were as follows:

- Confirm the compatibility of the new and existing systems at various levels of drift.
- Confirm that the EBF link beams yield before the slab ductility demands become excessive.
- Confirm that net tension does not develop in the piles at the ultimate base shear.
- Evaluate nonlinear, P-delta effects and the structure's ultimate drift.

The analysis was performed using two-dimensional frames. First, a model was developed of the original structure and a load-deflection curve was generated. Next, a series of models with the new frames were created. These models were linked with the original building model to evaluate combined behavior. Finally, the effects of each of the new frames were summed to generate an overall load-deflection curve for the upgraded structure.

The results of the analysis indicated that all of the objectives had been satisfied. The demand-capacity curves for the original and upgraded structures are provided in Figure 5. As can be seen, the upgrade increased the structure's strength by a factor of approximately 3.

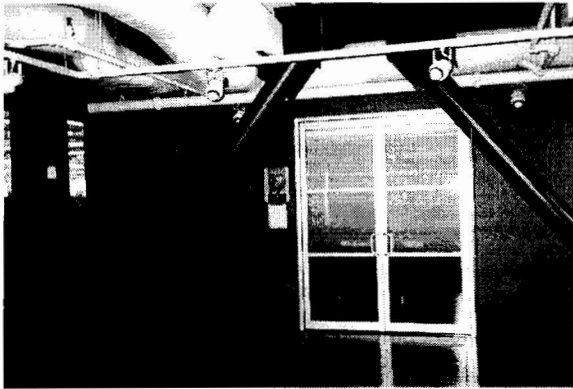


Figure 6: Upper Floor Bracing

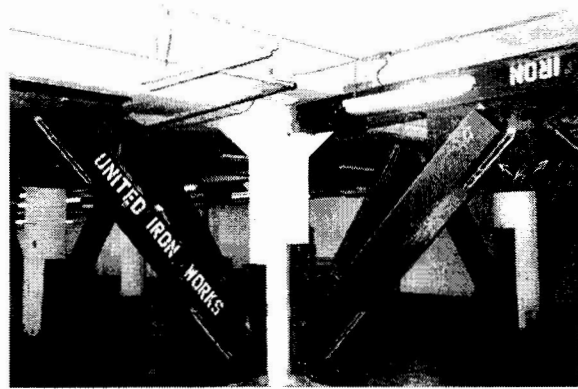


Figure 7: Mid-Floor Bracing

Construction Phase

Construction of the upgrade was one of the most challenging aspects of the project. The building contained numerous tenants, all of whom needed to remain fully operational during construction. For this reason, all work in the year-long construction phase was completed at night.

The work began in the upper floors (Starbucks' space), much of which was completed concurrently with tenant improvements. A second general contractor was completing these improvements and all of the work required extensive coordination with the facilities and space planning departments at Starbucks to minimize disruption to occupants. A photograph of a typical brace installation in an office location is shown in Figure 6.

The bulk of the steel erection occurred in warehouse space on the middle floors. Individual steel pieces up to 17 feet long and weighing up to 2400 lbs. were transported into the building using dollies and the large freight elevators that Sears originally installed. A photograph of one of the more congested areas on the 5th floor is shown in Figure 7. The remaining work on the lower floors was also quite challenging, with most of the construction occurring in congested, fully operational light manufacturing and retail space.

Given the unique nature of this project, a great deal of effort was expended up-front with the contractor to develop constructible details. A number of options were considered for each of the typical conditions, with the fabrication shop and the ironworkers having significant input in the process. The result was an overall system that went together on time and on budget.

SEISMIC EVALUATION AND UPGRADE OF THE ORIGINAL BUILDING

A seismic evaluation of the overall complex was performed as part of due diligence in a loan refinancing. The lender's criteria included site hazards (faults, liquefaction), probable maximum loss (PML) estimates, and life-safety concerns. The site hazards were acceptable per these criteria and although the PML for this building was relatively high, the criteria allowed an aggregate PML for the entire complex. This aggregate PML was at an acceptable level. The final criteria, which stated that the structure should maintain its vertical load carrying capacity in a 500-year earthquake, was a concern.

The lateral force resisting system of this building consists of perimeter reinforced concrete wall frames and laminated timber floor diaphragms. Using a FEMA-178 approach with 500 year seismic ground motions, the main system (wall frames and floor diaphragms) had adequate capacity. The interconnection of the elements, however, was deficient both for in-plane shear transfer and for out-of-plane wall anchorage. Finally, there were life-safety concerns with interior clay tile partitions.

The seismic upgrade consisted of the addition of out-of-plane wall anchors from the floor and roof diaphragms to the exterior walls. Options were given for adding the wall anchors from either above or below the diaphragm level to the joists or to blocking perpendicular to the joists. Additionally, in-plane shear transfers consisting of 3x3x18ga angles were added (see Figure 8). The contractor chose to alternate the installations from above and below each floor as to minimize the number of floors and spaces in which work needed to be completed. Additionally, all interior clay tile partitions were removed.

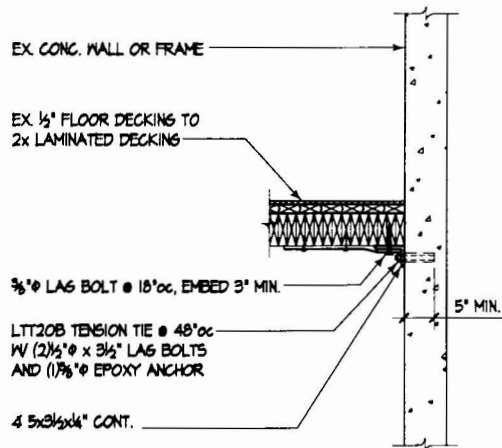


Figure 8: Floor-to-Wall Tie

SEISMIC EVALUATION AND UPGRADE OF THE NORTH BUILDING

As mentioned earlier, the north building was converted from a warehouse to a parking garage. As part of this conversion an exterior parking ramp structure and two new floors were added. Because of the addition of a significant amount of dead load to the structure, the City of Seattle treated the renovation somewhat like a new building. The agreed seismic approach was to use current code forces and FEMA-178 acceptance criteria for the evaluation of the existing structure.

The lateral system of the original warehouse consisted of virtually continuous shear walls along all four sides of the perimeter. The walls were constructed of cast-in-place concrete on the lower floors and reinforced CMU on the upper floors. There were a few minor openings in the walls for on a few of the floors and a series of large openings on one wall at the first floor for truck loading docks.

For the garage conversion the loading dock openings were infilled, new openings were cut in one side of the building for access to the ramps, and a significant portion of one wall was demolished for construction of elevator lobbies. In general, the demand/capacity ratios for the existing shear walls were well within acceptable levels. The only exception was the wall at the elevator lobbies. The seismic upgrade then consisted of construction of concrete shearwalls and pile foundations at the new elevator core. These walls were designed and detailed according to the provisions of the current Uniform Building Code.

CONCLUSION

This project, the largest seismic upgrade in Seattle, required a team effort by the engineers, architects, owners, contractors, tenants, and building officials. Various criteria were developed for each of the three buildings. An extensive evaluation, analysis, and design process was completed for the main building. More simple, straightforward evaluations were performed for the remaining buildings. The result in each case was a well-coordinated construction effort and a successfully completed project.