1994 NORTHRIDGE EARTHQUAKE: DAMAGE TO A FOUR-STORY STEEL BRACED FRAME BUILDING AND ITS SUBSEQUENT UPGRADE

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SUMMARY

In 1988, the Hewlett-Packard Company (HP) initiated a seismic program consisting of a preliminary seismic evaluation plan and a post-earthquake response plan. The initial evaluation of an owned or leased HP building includes the assessment of the structure's expected performance and the creation of a seismic folder for post-earthquake investigations. The post-earthquake investigations of HP buildings within Los Angeles following the 1994 Northridge Earthquake demonstrated the importance of a corporate seismic program in terms of locating damage and hastening repairs

One HP building in particular sustained substantial damage to its 4-story braced frame system that might have been overlooked without the pertinent information in the seismic folder. The extent of damage in the second level from thin-wall local buckling of the steel tube braces warranted a seismic repair to address the downtime concerns of HP and the improved seismic performance of the structure. In order to prevent similar damage from occurring in a future earthquake, the braced frames were upgraded using braces without thin walls and a configuration that will distribute brace buckling over multiple stories. In addition to avoiding localised damage, the upgrade was designed to avoid global structural performance deficiencies inherent in modern concentric steel braced frames. The damage to this building helped substantiate the need for tighter limits on wall thicknesses for tube steel brace members that have since been incorporated into seismic design provisions.

INTRODUCTION

Following the 1994 Northridge Earthquake, Degenkolb Engineers performed post-earthquake investigations of the Hewlett-Packard Company's (HP) owned and leased buildings in the earthquake-damaged regions. The rapid response and effectiveness of the building investigations were possible due in part to HP's well-established corporate seismic program. Had HP's program not been so thorough, critical damage to one of its four-story steel braced-frame buildings may have been overlooked, which could have prompted a longer repair schedule. Once the structural damage was identified, HP had to make a crucial decision as to whether the building be repaired or seismically improved. HP opted to enhance the building's seismic performance with an improved braced frame system. This paper describes aspects of the seismic program that facilitated finding structural damage, the decisions that had to be made in regards to repairing the building, and the scheme used to improve the building's seismic performance.

DESCRIPTION OF BUILDING

The subject building is a four-story office building located in the outskirts of Los Angeles, California near North Hollywood, approximately 17 kilometres east-southeast from the epicentre of the Northridge Earthquake. Its structure consists of a steel frame for gravity loads and steel braced frames for wind and seismic forces. The structure was designed in accordance with the 1980 Los Angeles Building Code, and constructed in 1986. A photograph of the building is shown in Figure 1.

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In plan, the building has several reentrant corners and one of the elevations is skewed relative to the framing layout as shown in Figure 2. The building originally had minor load transfer discontinuities; in particular, one bay of bracing was offset in-line, shifted one bay in the first level relative to the upper levels. Approximately 50% of the building layout has a basement story for underground parking and storage.

The floor framing consists of concrete fill on metal deck that is supported by composite steel beams and girders. The steel structure is supported on precast concrete pile foundations. Seismic forces are resisted in each direction by six bays of Chevron-configured tube steel braces for a total of 12 braces in each direction in each story. The locations of braced frames are indicated in Figure 1 and a typical braced frame elevation is shown in Figure 3. The braces were slotted at their ends and welded to the gusset plates, and the gusset plates were welded to the beams and bolted to the column flanges through double angles. Beams within a bay of bracing generally had welded beam flange-to-column connections, and the web was bolted with double angles.

The exterior of the building is clad with precast concrete spandrels and windows between spandrels. Tall parapets extending 5.8 meters above the roof serve as a screen wall for mechanical equipment in one area of the building. In general, building columns cantilever above the roof to support the precast elements.

CORPORATE SEISMIC PROGRAM

In 1988, one year before Loma Prieta and six years prior to the Northridge Earthquake, Hewlett-Packard initiated a seismic program to facilitate post-earthquake investigation and repairs. The following are the principal aspects of HP's seismic program:

(1) create a database of the building inventory,

(2) perform preliminary evaluations of owned and leased buildings,

(3) prepare seismic folders for each building, and

(4) have a structural engineering firm on retainer for post-earthquake investigations.

The database is used to accurately identify buildings located in an earthquake-damaged region. The pertinent structural information within the database includes the structure type, year constructed, and seismic performance rating for each building. Based on the susceptibility of certain structure types and the proximity of the buildings to a known epicentre, the database can be used to quickly establish the order of post-earthquake investigations.

Preliminary seismic evaluations are performed to determine the expected seismic performance. The results of a performance evaluation are included in a seismic folder for each building, a copy of which is kept at the site. A plan showing the location of lateral force resisting elements is included in the folder to help post-earthquake investigators locate probable structural damage within the building.

Equally important to the seismic evaluation and folder is the retainer between the company and the structural engineer. Immediately following an earthquake, engineers are typically inundated with work. Having an engineer under contract ahead of time to perform evaluations and repairs allows for a faster recovery from an earthquake. Also of importance is have a strong existing relationship with a general contractor that is willing to put the company's repair needs above other companies' needs as HP has. General contractors are also inundated with work after an earthquake. Therefore, a contractor needs incentive, such as strong consideration for future work, if he/she is going to make a company's repair needs a higher priority than other companies' needs.

POST-EARTHQUAKE INVESTIGATION OF BUILDING

The seismic folder prepared for one particular building proved to be invaluable for promptly locating building damage in what initially appeared to be a routine inspection. The damage to the exterior consisted of two broken windows with little misalignment of the precast panels. Based on the small amount of damage to the exterior, one would not expect the degree of structural damage this building sustained. A walk through the building helped investigators obtain an initial sense for the degree of overall non-structural and structural damage. The investigation team observed a disproportionate amount of damage to the ceilings in the second floor level indicating a potential problem. The structural plan in the seismic folder helped the team quickly locate the braced frames. Because the braces were concealed behind interior partition walls or covered with metal furring and gypsum wallboard, it was not possible to directly observe the braces without removing architectural finishes.

In the first few locations of braced bays, cracks were observed in the partition walls and gypsum wallboard covering, but the level of damage did not warrant removing the architectural finishes to view the braces.

Eventually, the team came to a location in a stairwell with a large bulge in the wall at the second level. Because a bay of bracing was located behind this wall, it was decided that the braces be exposed to view the cause. The braced frame damage concealed by the walls consisted of the following: the bottom flange of the supporting overhead beam twisted out-of-plane, the end connection of one of the braces failed and the other brace fractured. The intense level of structural damage in this particular frame warranted further investigation of the other braced frames. The additional investigation into the remaining braced frames concluded that every second level brace oriented to resist north-south forces lost all or most of its capacity.

The loss of capacity was primarily due to premature fracture of the tube steel braces initiated by local buckling of the thin walls. The width to thickness (b/t) ratio of these braces was 29. Fractures were located at the middle of the brace length and immediately adjacent to the gusset plate connections at the ends, locations most affected by overall brace buckling (Figure 5). Tang and Goel predicted such fractures in 1989 based on laboratory tests, but up until Northridge, post-earthquake investigations of buildings with tube steel braces did not indicate a problem with thin-wall buckling.

Structural damage in the first, third, and fourth levels was relatively light with only a few conditions where local buckling had begun. In these cases, cracks had not yet developed at the corners of the braces or the cracks had not yet propagated to cause complete fracture of a brace as observed in the second floor. The damage in the third and fourth levels was probably light because the second floor braces buckled, limiting the force to the upper stories. No brace damage was observed for braces oriented in the east-west direction. This was consistent with damage observed in other buildings throughout the affected areas of the San Fernando Valley, and is a result of the stronger ground motion in the north-south direction.

Other structural damage included cracking of the floor diaphragms at reentrant corners and at a location where a bay of bracing had an in-plane offset of one bay in the first level. In both cases, the beam to column connections had failed, requiring that seismic forces flow through the slabs and thus causing them to crack.

The most hazardous condition observed was the potential for a large precast panel for the screen wall above the roof to fall. The top of this panel was leaning out only about an inch after the earthquake making the damage difficult to find. The cause of the panel leaning was that one of two supporting cantilever columns had fractured at its roof level splice. The splice consisted of a cover plate that the column segments above and below were welded to. Also of importance was the fact that the strong axis orientation of the wide-flange columns were rotated 90 degrees at this splice. The column to plate connection failed completely allowing the column to lean outward. This is particularly significant since the sidewalk was near the main entrance to the building.

REPAIR VERSUS UPGRADE AND OTHER OWNER'S DECISIONS

Once the damage was discovered, HP had to make a decision on how to proceed with the building. Would it be acceptable just to replace the broken braces and connections? Doing so would have resulted in a structure that would not have met the requirements of the Los Angeles Building Code, 1992 edition. What would the City of Los Angeles require? Would only repairing the damage leave HP with a building that was still a potential life-safety hazard in a future earthquake? Would the necessary repairs and possible upgrades be made within the 90-day period allowed for in the lease agreement with HP’s tenants? How much would the repairs and possible upgrades cost? These questions had to be answered quickly.

HP chose to upgrade the lateral force resisting system to the provisions for a special concentric braced frame in the Uniform Building Code, 1994 edition, and the requirements of the Los Angeles Building Code, 1992 Edition. A principal reason for this decision was that there were indications that the City of Los Angeles was going to require a building with more than 10 percent loss of strength in its lateral force resisting system to be upgraded to the current building code. There was confusion as to whether the City would require all levels of the building to be upgraded or just the second level since substantial damage only occureed in the second level. HP, with the guidance of its structural engineer, chose to address the whole building.

An additional deciding factor was that replacement of the broken braces and connections, or upgrading only the brace system in the second story, could not guarantee life safety. The building nearly developed a single story mechanism during the earthquake, and repairing the building without properly addressing the weakness at all stories could result in similar or worse performance in a future earthquake.

Estimates of construction costs and schedule were needed to aid HP's decision making process. The post-earthquake investigation team worked with a general contractor familiar with HP's construction needs to provide these estimates. It quickly became evident that the necessary upgrades could not be completed in 90 days as HP allowed for in its leases with the tenants. In order to reduce the schedule to the minimum length of time, the contractor removed the existing system, while the structural engineers from the post-earthquake investigation team designed the upgrade scheme. The final cost for the upgrade turned out to be $6,000,000 (U.S. dollars). The building re-opened for general occupancy 120 days after the earthquake.

DEFICIENCIES OF MODERN BRACED FRAMES TO BE AVOIDED IN UPGRADE

Once HP decided to upgrade the lateral force resisting system, the design team had to develop a strengthening scheme to improve seismic performance. The building would remain a steel braced frame, but with the new structural elements exceeding the minimum braced frame requirements in the 1992 Los Angeles Building Code. In addition to the code requirements, the design team tried to avoid the following deficiencies common in modern braced frames:

- connections that do not develop the tensile capacity of a brace
- a configuration, such as a chevron or inverted chevron, that concentrates deformations in a single story
- thin walled tubes that could fracture prematurely
- stout braces
- weak columns that buckle before a braced frame reaches its ultimate capacity

Most of these deficiencies are now addressed in *AISC's Seismic Provisions for Structural Steel Buildings* for a special concentric braced frame, but are not addressed for ordinary braced frames. Because of the severity of these deficiences, it is the authors' opinion that ordinary concentric braced frames should generally not be used in regions of high seismicity.

IMPROVED BRACED FRAME SYSTEM

At the time of the upgrade design, provisions for a special concentric braced frame had been proposed but not yet adopted for inclusion into the 1994 Uniform Building Code.

Therefore, proposed provisions and recommendations made by Goel, et al (1989, 1991) were used for the upgrade design. The design team considered the advantages and disadvantages of using a two-story X configuration, a modified chevron configuration that had a zipper column in the centre of the bay of bracing in all levels above the first floor, and a tied column in the centre of the chevron at all levels. These configurations are described in greater detail in a report by Khatib, Mahin, and Pister. The modified chevron with a zipper column was chosen for the upgrade, because it distributed buckling over the height of the building better than the other configurations considered. An example of this configuration is shown in Figure 4. Instead of using tube steel braces, wide flanged members were used. Connections were designed for the member tension capacities rather than the force requirements of the Los Angeles and Uniform Building Codes. The lowest levels of several of the columns were strengthened with steel side plates in order to avoid bucking of the columns. In accordance with what Goel (1991) espouses, relatively slender braces were used. The expected behaviour of the upgraded system is further described in Bonneville and Bartoletti (1995) and in Bonneville, et al (1995).

Although not considered at the time, the authors' recommend that the capacity of a brace in tension plus its residual compressive strength once it has buckled should equal or exceed two times its compression capacity. For example, using a residual compressive capacity of 30 percent for a tube steel brace as allowed for in FEMA 273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* the tensile capacity required would then be at least 1.7 times the buckling capacity. This will allow for a frame to continue gaining strength once a brace buckles. If this is not provided, the shape of the non-linear static pushover curve is likely to be sloped downward after the braces buckle in one level.

CONCLUSIONS

Based on events following the 1994 Northridge Earthquake, HP's seismic program proved to be a working success. One HP building in particular demonstrated the importance in the development of the seismic folder in the preliminary seismic evaluation and the critical decision making in the post-earthquake response. The packet of information included in the seismic folder was particularly useful for locating damage to the steel braced frames.

The extent of damage in the second level from thin-wall local buckling of the steel tube braces warranted a seismic repair to address the downtime concerns of HP and the improved seismic performance of the structure. The upgrade intended to prevent local buckling, promote a well-distributed brace buckling over multiple stories, and avoid the structural performance deficiencies inherent in modern concentric steel braced frames. More important than the success of the corporate seismic program, the damage to this building helped substantiate the need for stricter limits on wall thicknesses for tube steel brace members that have since been incorporated into seismic design provisions.

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Figure 1: Building Elevation

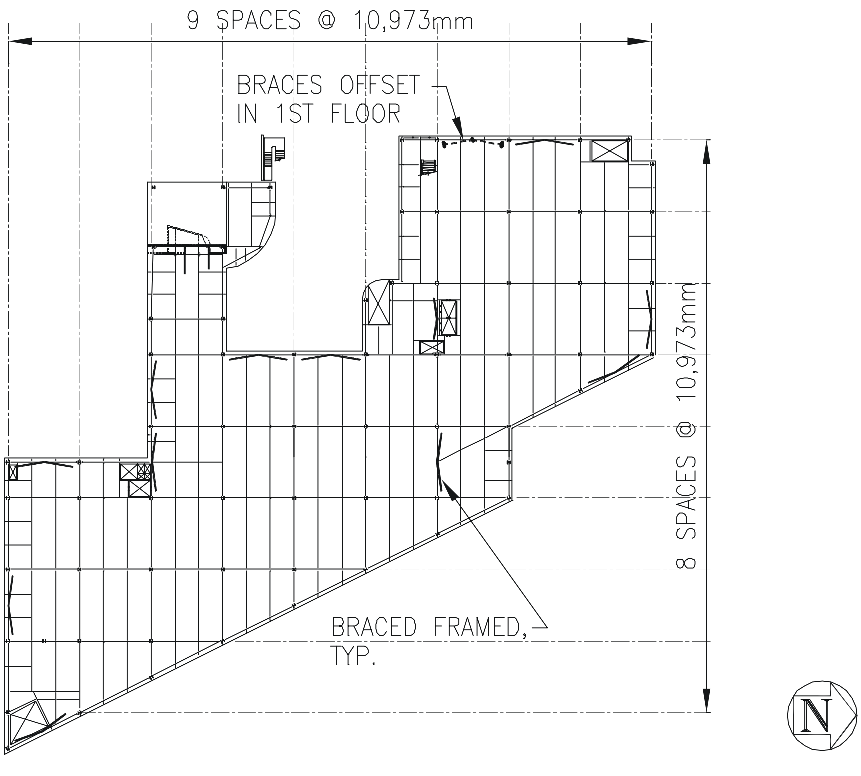

Figure 2: 4-Story Steel Braced Frame; 2nd Floor Framing Plan (3rd, 4th and Roof Similar)

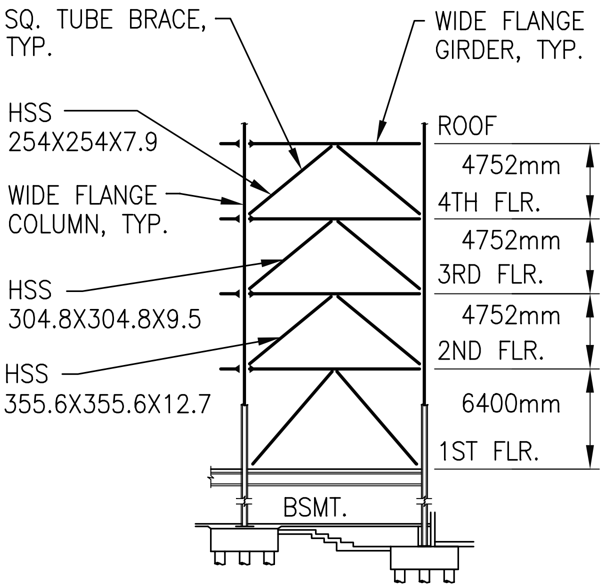

Figure 3: Typical Chevron Braced Frame

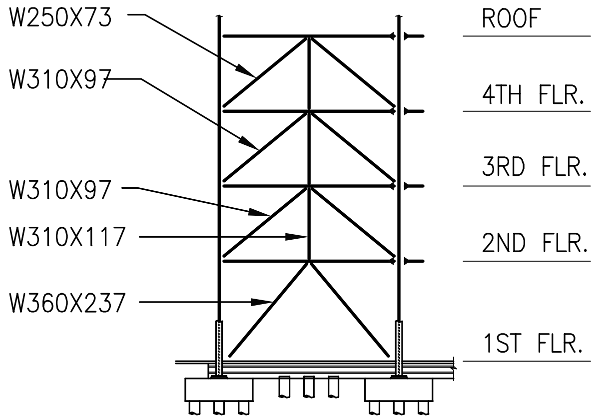

Figure 4: Upgraded Braced Frame with Zipper Column

Figure 5: Brace Fracture