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SEISMIC UPGRADE OF LIONS GATE HOSPITAL'S ACUTE TOWER SOUTH

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SUMMARY

The proposed new external steel braced frames and new exterior reinforced concrete shear walls for the seismic retrofit of the Lions Gate Hospital are designed and detailed in accordance with specific requirements of ductile braced frames and ductile shear walls, respectively. The intent is to utilize the ductility associated with the braces and walls to absorb the seismic input energy while protecting the floors and gravity load carrying system. Pushover analyses of the frames/walls in the E-W and N-S directions confirmed the formation of axial hinges in the braces and flexural hinges at the base of the walls.

The proposed upgrade scheme for the steel floor decking and the connection plate details of the floor to the external steel braced frame system is crucial for establishing a proper load path to transfer the storey shear forces to the perimeter external framing. The use of epoxy/fibre composite reinforcing steel plates bonded to the existing metal decking to create a continuous metal deck diaphragm independent of the thin concrete topping is a unique solution to address the inadequate shear capacity of an effectively unreinforced and discontinuous concrete topping in transferring the seismic loads to the external lateral load-resisting elements.

INTRODUCTION

Seismic risk assessment of critical structures and buildings in acute care hospitals is an important topic in the earthquake engineering community. Hospitals must remain operational after an earthquake to render emergency care. This may require retrofit of the building, its foundation, and its contents. While other buildings house important emergency services (i.e., fire stations, police stations, etc.), hospitals pose unique challenges as they integrate complex structural and non-structural systems.

Design of seismic retrofitting of hospitals is a challenge that must balance cost and minimizing disruption to the critical ongoing services of the facility. This paper presents the retrofit design of a 15,000 square

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meter, 7-storey Acute Patient Care Tower that houses hundreds of wards and is essential to the post-earthquake medical response in the North Shore Region of the Greater Vancouver area.

BUILDING DESCRIPTION

The original structure was constructed in two phases between 1958 and 1967, and is a riveted steel moment frame structure with a thin unreinforced concrete slab on metal deck supported by open web steel joists. Furthermore, the building is made up of 3 structurally independent sections, with inadequate expansion joint allowance.

The Acute Tower South (ATS) is a seven-storey building with a basement. The footprint of the building is about 21 m in the N-S and 81 m in the E-W directions. The existing gravity and lateral load carrying system comprises of steel beams and columns with semi-rigid connection details. The floor system consists of 44 mm (1¾ in.) thick concrete topping placed over 38 mm (1½ in.) deep metal deck. A 20 mm (¾ in.) thick terrazzo was placed on top of the concrete to provide a finish. The steel deck is supported on top of open web steel joists about 1.1 m apart. Stair and elevator shafts have steel framing all around with infill clay tiles. A plan and an elevation view of the building are shown in Figure 1.

ANALYTICAL MODELLING OF THE BUILDING

A detailed SAP2000 (CSI, 2000) computer model of the building was prepared. The model includes all storey beams and columns simulated as FRAME elements. The open web steel floor joists were modeled as pin-ended FRAME elements. The storey floors were modeled as 44 mm thick concrete SHELL elements. A three-dimensional view of the SAP2000 model of the building together with a typical interior frame is shown in Figure 2.

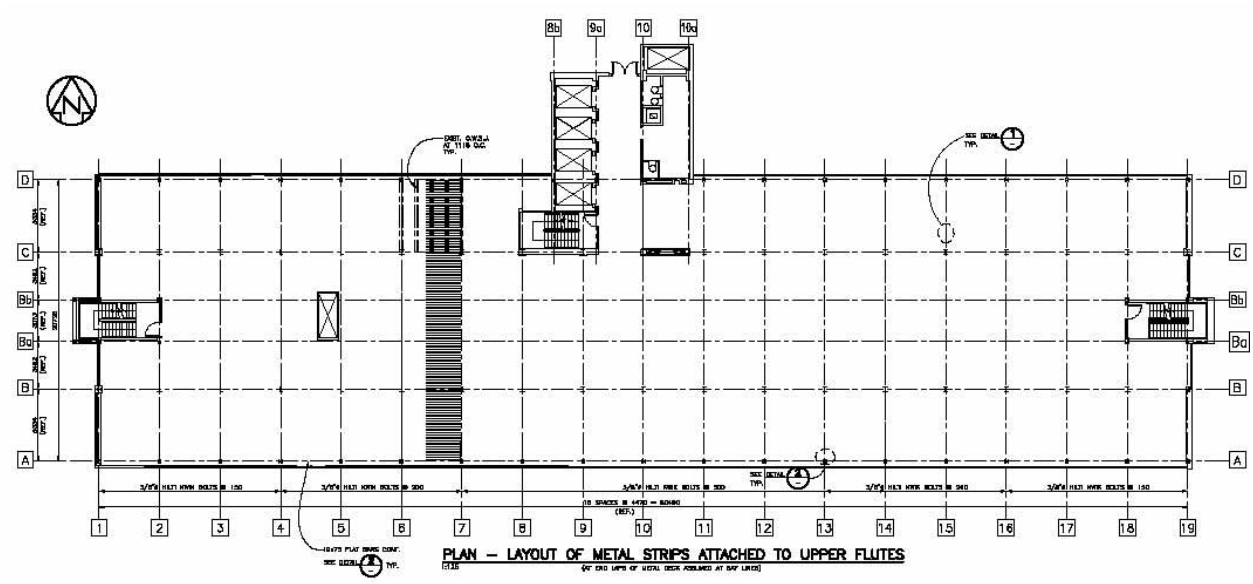
The computed mass of each floor including partitions, mechanical piping and electrical equipment was estimated at about 600 kg/m^2. This resulted in an overall weight of the building at the first floor of 82,400 kN. This weight is used to compute the code prescribed seismic storey shear demand on the building.

All exterior and N-S interior beam-to-column connections were modeled as pin joints. All interior beam-to-column connections in the EW direction were modeled as fixed joints. The pin connection detail assumption minimizes the lateral seismic load absorbed by the steel framing system.

The building was analyzed using the response spectrum analysis procedure prescribed in the National Building Code of Canada (NBCC, 1995). The seismic design spectrum curve recommended for the City of Vancouver was used. Furthermore, a pushover analysis was carried out to determine the location of plastic hinges within the braces and at the base of the shear wall.

Pushover Analysis

Chapter 3 of the Prestandard and Commentary for the Seismic Rehabilitation of Existing Buildings (FEMA 356) explains the nonlinear static analysis procedure, commonly known as pushover analysis. It makes a reference to several standard methodologies including NEHRP guidelines for the seismic rehabilitation of buildings (FEMA 273) and post-earthquake evaluation and repair of concrete and masonry buildings (ATC 40). SAP2000 static pushover analysis capabilities, which are fully integrated into the program, allow quick and easy implementation of the pushover procedures prescribed in the ATC-40 and FEMA-273 documents.



Plan View

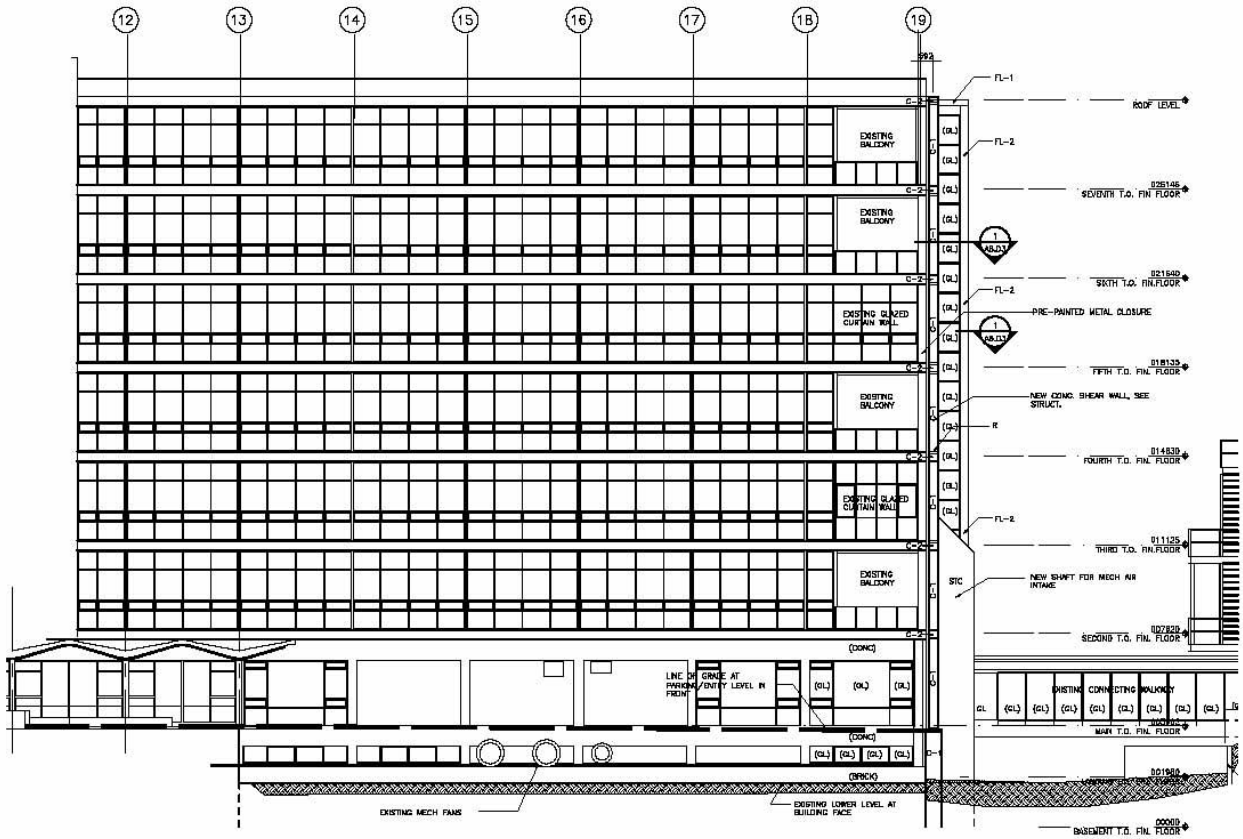


Figure 1: Plan and elevation views of the Lions Gate Hospital

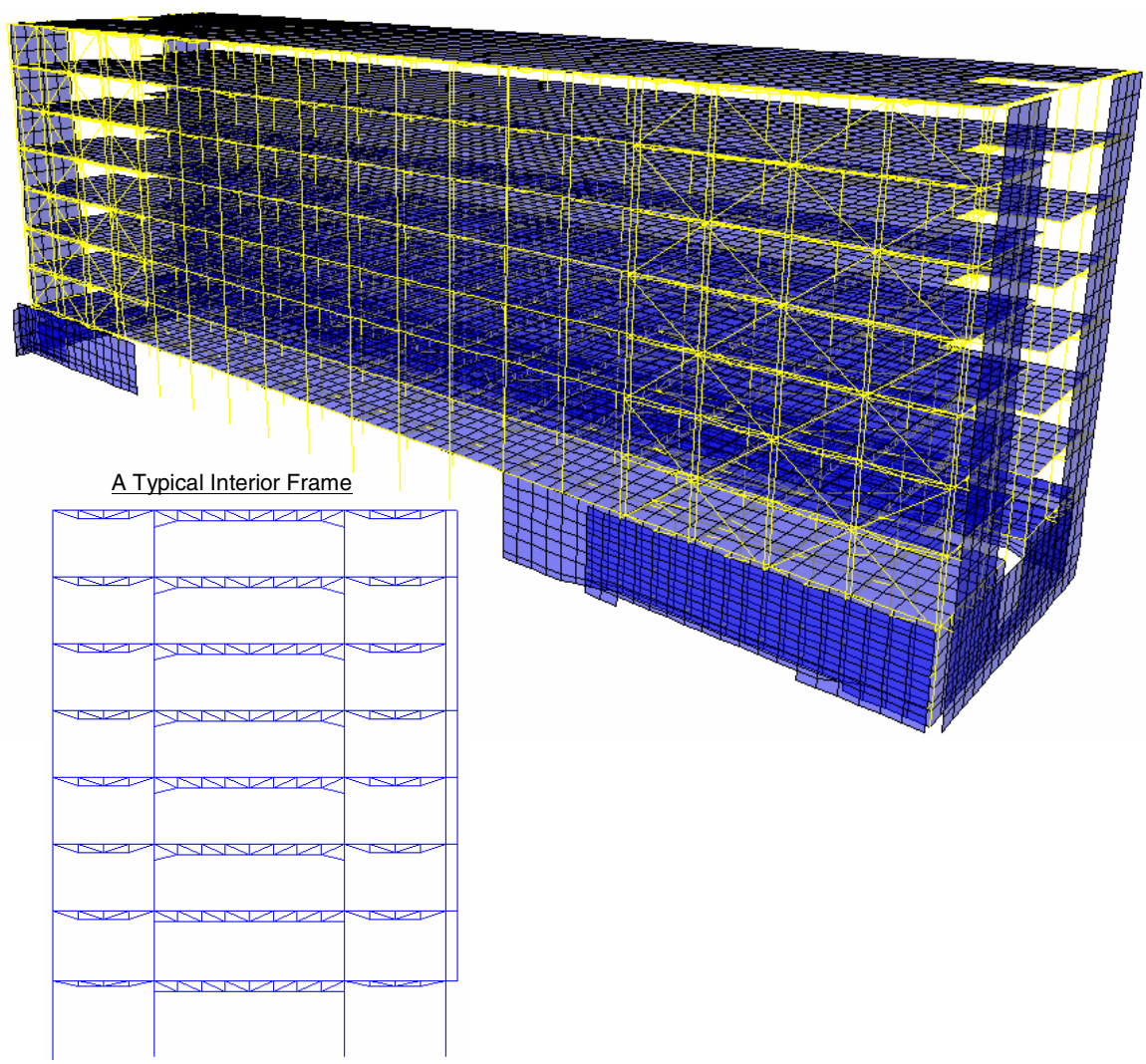


Figure 2: 3D and typical interior frame elevation views of the SAP2000 model of the Lions Gate Hospital; New shear walls shown at ends, new external bracing near side

The results of pushover analysis are usually plotted in terms of lateral force and lateral roof displacement. It can also be plotted in terms of spectral acceleration and spectral displacement for use in capacity spectrum method of evaluating structures (ATC 40).

Pushover analysis was conducted according to the criteria outlined by FEMA 273 Section 3.3.3. This document defines the force-deformation criteria for hinges used in a pushover analysis. As shown in Figure 3, five points labeled A, B, C, D and E are used to define the force deflection behavior of the hinge and three points labeled IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention) are used to define the acceptance criteria for the hinge. The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in FEMA 273.

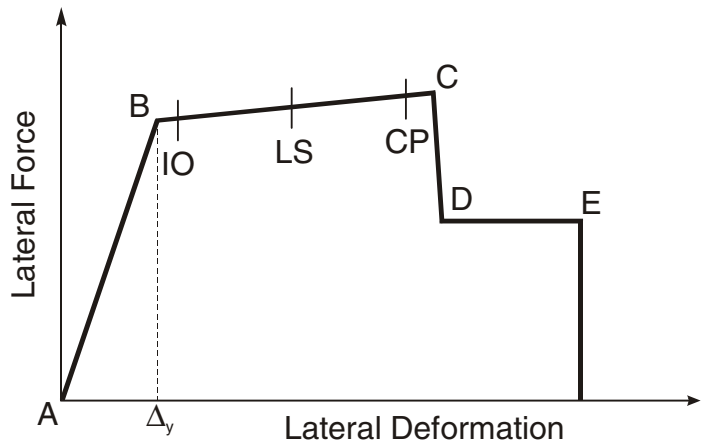


Figure 3: Force-deformation relationship for a pushover hinge (FEMA 273)

For braced frame structures the Δ/Δy (Post-yield displacement/yield displacement) acceptance criteria for nonlinear procedures is defined as: 1.25, 6 and 8 (in between points B and C in Figure 1) for hollow tube tension braces for IO, LS and CP, respectively. For hollow tube compression braces the acceptance criteria is defined as: 1.25, 5 and 7 for IO, LS and CP, respectively.

The nonlinear axial hinge properties of the braces and axial-moment interaction of the columns were computed taking into account the expected yield strength of the steel material as 350 MPa with an overstrength factor, Ry, of 1.1. For the compression capacity the unsupported length of each brace was considered as 90% of its node-to-node length.

Input parameters for the load-deformation response of the structural elements were adopted from FEMA 273 using Table 5-7. A strain-hardening slope of 2% of the elastic slope was considered for the post-yield load-deformation response of the braces. Figure 4 illustrates the load-deformation behaviour of the module braces and beams and columns, as modeled in SAP2000.

The interface of the concrete strip footing and soil was modeled as a series of discrete compression only spring elements. The axial spring of each element was assigned in such a way to result in 45 MN/m^3 modulus of subgrade reaction at the bottom of the footings. It is, however, recognized that because of the difficulties in determining soil properties and the likely variability of soil supporting foundations, an upper and lower bound approach to defining stiffness and capacity of the interface spring elements is required to evaluate the sensitivity of the structural response to these parameters. No sensitivity study, however, was carried out during this project. A 45 MN/m^3 soil modulus of subgrade was deemed appropriate for this assignment by the Geotechnical Engineer.

Loading Pattern

The pushover analysis was carried out for a loading pattern similar to the profile of the story shear inertia forces consistent with the story shear distribution calculated according to the response spectral analysis. Other loading patterns such as constant acceleration method are also recommended in FEMA 273. The load distribution per response spectral analysis was deemed adequate for this project.

Displacement Target

One of the main discussions surrounding the pushover analysis methodology is the determination of a target displacement. Several methodologies for estimating the amount of deformation induced in a structure by the design earthquake have been proposed and are included in various references.

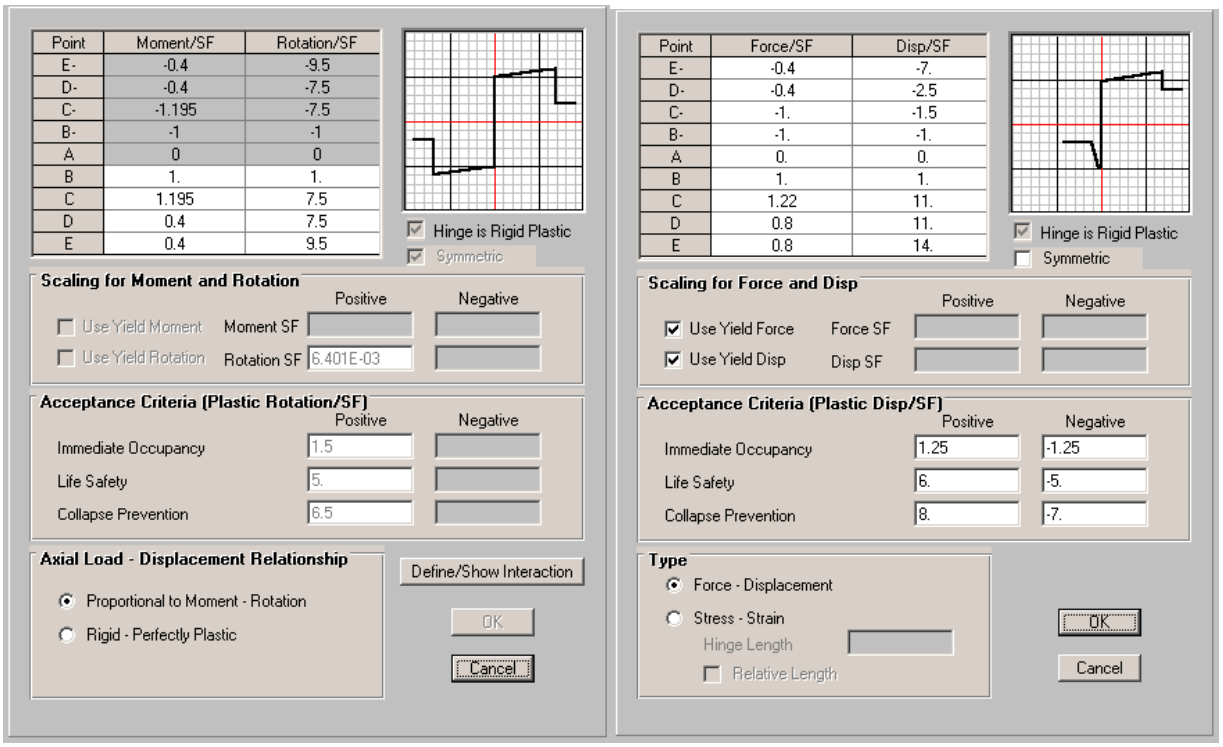


Figure 4: Load-deformation behavior of the braces (right) and beams and columns (left), as modeled in SAP2000

The NEHRP 2000 (FEMA 368) proposes the continuation of nonlinear analysis by increasing the loading pattern until the deflection at the selected control point exceeds 150% of the expected inelastic deflection. For a structure that is mainly dominated by the first fundamental mode of response the expected inelastic deflection at each level shall be taken as the deflection predicted from a modal response spectrum analysis factored by the coefficient C_1:

$$C_1 = \frac{(1 - T_s / T_1)}{R_d} + (T_s / T_1)$$

where T_s is the characteristics period of the response spectrum, defined as the period associated to the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum and T_1 is the first fundamental mode of structure in the direction under consideration. R_d is given by the following equation:

$$R_d = \frac{1.5R}{\Omega_o}$$

where R and Ω_o are, respectively, the response modification and overstrength coefficients from Table 5.2.2 of the NEHRP 2000. When the ratio of T_s/ T_1 is less than or equal to a value of 1.0, the coefficient C_1 shall be taken as having a value of 1.0.

PROPOSED RETROFIT OPTIONS

The preliminary design considered two basic retrofit options: the first, conventional structural elements all external to the building plus connecting together and reinforcing of all diaphragms; and second, friction dampers all located internal to the building. The costs for the two options were nearly identical, however ultimately the external conventional option was selected as the option deemed least disruptive to patient care.

The retrofit solution consists of the addition of external ductile shear walls in the N-S direction that effectively replaced a leaking building envelope at each end of the building, external ductile braced steel frames in the orthogonal direction, and a unique upgrade of the floor diaphragms utilizing steel plates bonded to the existing metal decking to create a continuous metal deck diaphragm independent of the thin concrete topping. Vertical soil anchors were used at the foundation level to help resist the overturning moment demand at the base of the frames and walls. The following will summarize the proposed retrofit elements.

External Braced Frame System

The external braced frame beams and columns were consisted of I-sections while the braces were Hollow Structural Shape (HSS) boxes. The seismic design of all framing members ensured proper *b*/*t* ratios to avoid local buckling and adequate slenderness ratio for all braces to avoid tension-only braces. All braces were connected to the surrounding beams and columns through a proper gusset plate connection detail. All gusset plates would meet the AISC special concentric braced frame detailing requirements. This includes ensuring buckling of the braces in the out-of-plane direction by providing adequate unrestrained space at each end of the brace to gusset connection to allow formation of hinges in the gussets as braces undergo cycles of tension-compression.

The results of response spectral analysis indicated that the braces in the 4th floor and above could be further optimized than what was developed in the initial design phase. The optimization of the braces ensures that they would undergo inelastic action under prescribed storey shears and thus reduce the overall seismic demand to the building. The optimized bracing members together with the storey shear demand and nominal shear capacity of one of the braced frames at each floor level are shown in Figure 5.

Pushover Analysis Results

The target displacement of the building in the E-W direction was computed as 200 mm at the roof level. This displacement was assumed to be the ultimate target displacement of the building in line with the external braced frame direction.

As the building was pushed to the selected target displacement, compression buckling occurred in the brace elements at the 4th level. As the demand reached the target displacement, the compression buckling extended to almost most braces above the 4th level together with some tension yielding and some uplift for the foundation across the strip footings. The foundation uplift at the edge of the footing was about 12 mm. This is an axial extension in the soil anchors resisting uplift forces at each end of the footing. An elastic axial extension of about 16 mm is expected for the soil anchors at full capacity.

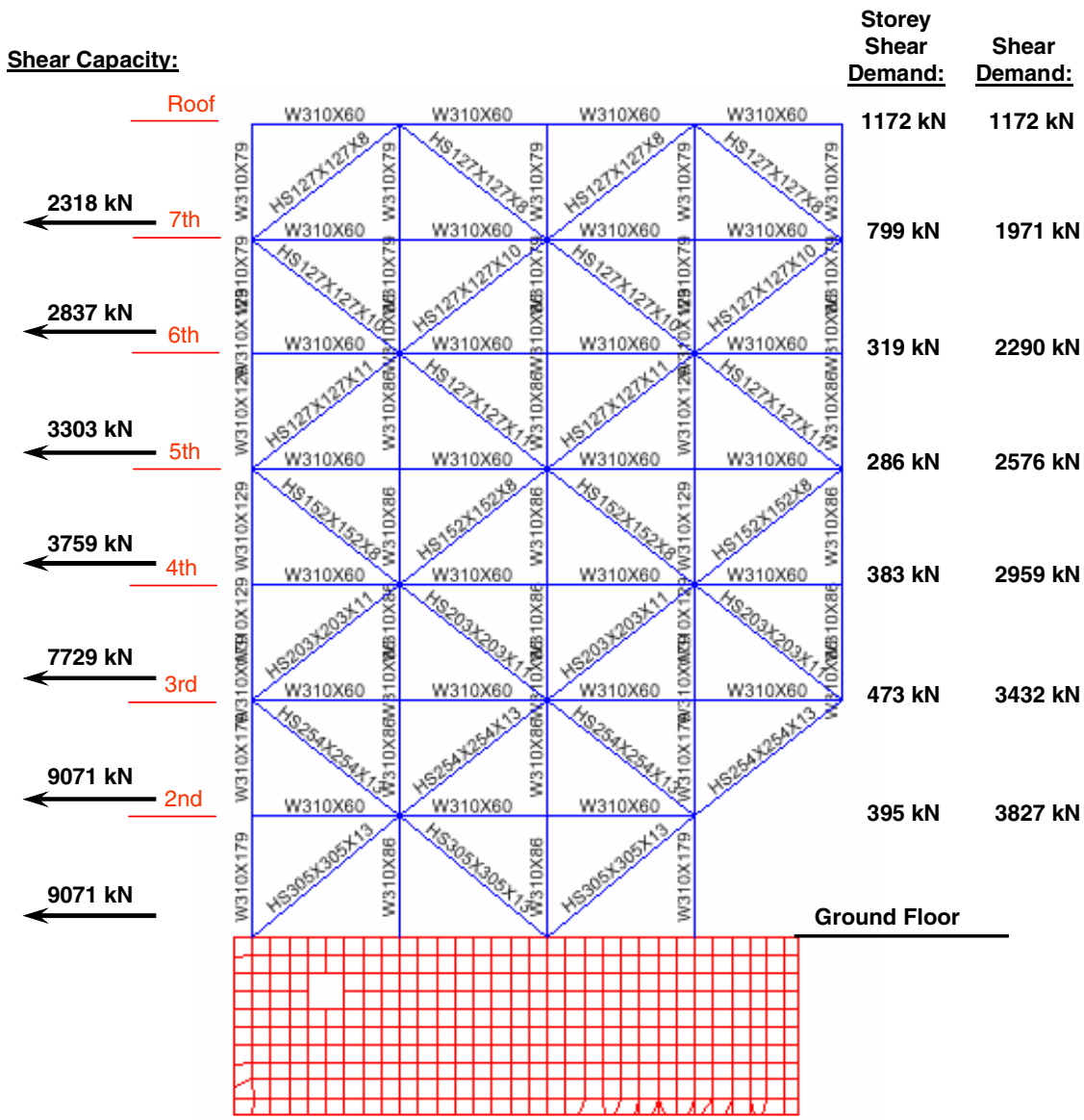


Figure 5: Lions Gate Hospital – longitudinal braced frame sizes & storey force demand and capacities for braced frame near the N-E corner

This indicates that the energy absorption in the system is limited but well distributed across most braces above the 4th level. The foundation rocking/stamping would also contribute to the energy absorption mechanism in the system. Figure 6 illustrates the extent of brace nonlinearity and foundation uplift for the full length of the strip footings at the end of pushover analysis for the earthquake loading in the E-W (displacement target of 180 mm at the roof level) direction.

The color spectrum at the bottom of Figure 6 indicates the extent of nonlinear action. The purple color (far left) indicates start of nonlinear behaviour, the dark blue color indicates limited yielding in the region for immediate occupancy, and the light blue is the life safety zone while the green color is the collapse prevention zone. The yellow color is the start of strength degradation. The orange color indicates that the member has reached its reserved capacity and the red color is where the member strength is essentially

zero. The axial load in the compression braces drop out to their assumed 40% post-compression reserve capacity shortly after the braces reach their buckling strength.

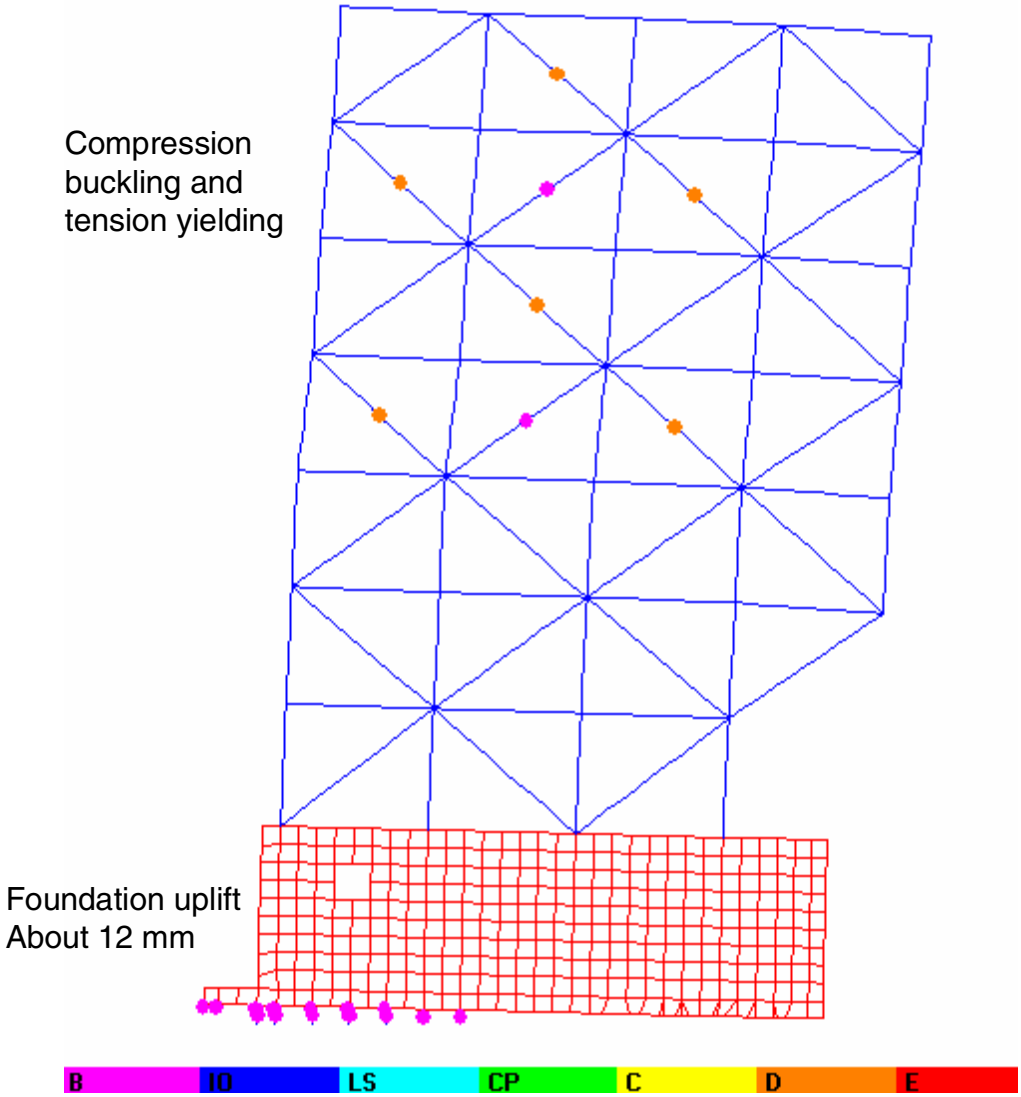


Figure 6: Extent of nonlinearity (compression buckling/yielding & foundation uplift) in the braced frame located near the N-E corner of the building at the end of pushover analysis for earthquake loading in the E-W direction

External Reinforced Concrete Wall

The external reinforced concrete shear wall in the N-S direction was designed and detailed as ductile wall. The design concept ensured the formation of plastic hinges at the base of the wall just above the basement floor.

Soil anchors were provided at ends of the wall to provide the fixity and rigidity desired with adequate capacity to resist the overturning moment demand at the base. The soil anchors were 45M DYWIDAG bars with axial tension capacity of about 1200 kN.

Pushover Analysis Results

The target displacement of the building in the N-S direction was computed as 280 mm at the roof level. This displacement was assumed to be the ultimate target displacement of the building in line with the external concrete shear wall direction.

An elevation view of the concrete shear wall in the N-S direction at the target displacement of 280 mm at the roof level is illustrated in Figure 7. At this target displacement the moment at the base of the wall above the ground level has reached its probable moment capacity undergoing plastic deformations. The soil anchors at the base of the wall are at about 80% of their capacities with an axial elongation of about 10 mm.

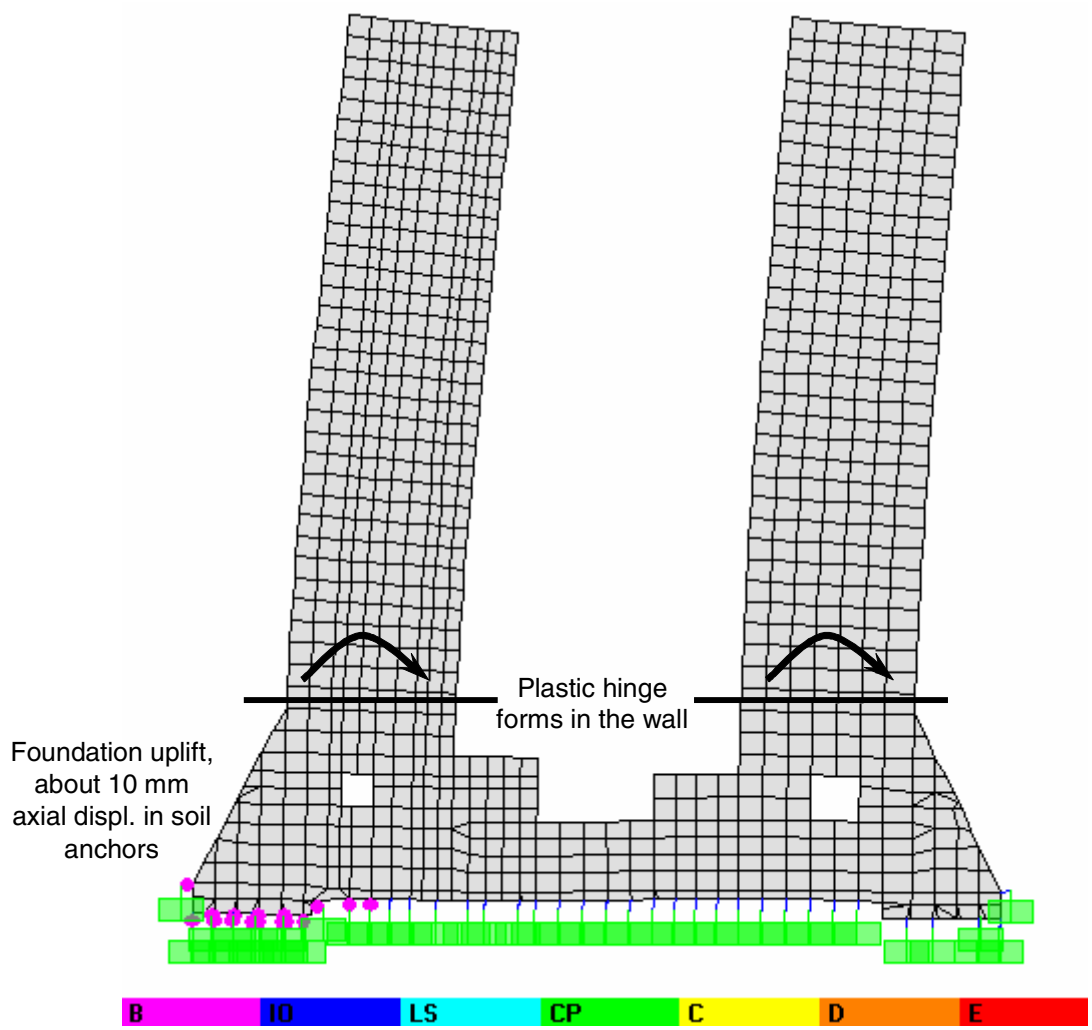


Figure 7: Extent of nonlinearity (plastic hinge formation in the wall & foundation uplift) at the end of pushover analysis for earthquake loading in the N-S direction

Floor Diaphragm

An important aspect of the design is to ensure that the floor diaphragms are properly attached to the external framing system. This requires an upper bound estimate of the shear capacity of the braces to make sure that the floor connection plate details have sufficient capacity to transfer the storey shear forces to the

boundary frames. Based on the results of dynamic analysis and calculated nominal capacity of the braced frames at each floor level, an upper bound storey shear force of 900 kN was assumed to be transferred to each of the steel braced frames for the floors 2nd to 6th. For the 7th floor and roof, the connection plate details were to be designed for a shear transfer load of 1200 kN.

The shear capacity of the 44 mm thick concrete floor was computed as about 400 kN for the last four bays adjacent to the external braced frame system. This is considerably smaller than the required demands of 1200 and 900 kN. Furthermore, the capacity of the concrete slab was suspect due to the assumed cold joints due to the original construction sequence, and effective lack of reinforcing steel (minor wire reinforcing noted in some locations).

To augment the shear capacity of the concrete floor a scheme was proposed to utilize the shear capacity of the metal deck diaphragm. It was, however, apparent that the existing 600 mm wide by 4500 mm long steel deck segments did not have a proper side lap connection details to provide adequate shear resistance against storey force demands. To utilize the shear capacity of the steel deck floor system continuity plates are to be provided to properly tie the steel deck segments together. This could be achieved through the use of epoxy/fibre composite reinforcing or Hilti nails and steel plate straps at discrete locations. Both Hilti nails and epoxy/fibre composite reinforcing options were deemed feasible for providing adequate shear capacity. The economical and practical constructability aspect of each system was to be investigated for final adoption. Either proposed scheme resulted in an additional shear capacity of about 1000 kN for the composite steel deck floor system

Floor Connection Plates

The floor diaphragms were connected to the external braced frames using discrete steel plate elements. The plate elements were 650 mm wide and 2500 mm long placed in the space between the open web floor joists. A total of 20 plates (10 plates for each of the braced frames) with shear transfer capacity of 90 kN for the floors 2nd to 6th and 120 kN for the 7th floor and roof were utilized. The plates will be connected to the steel deck from underneath using Hilti adhesive anchors.

CONCLUSIONS

External retrofit schemes are very appropriate for upgrading hospital facilities, and can be designed in a manner to protect the original structure while behaving in a ductile manner. The challenge is to develop details to upgrade the floor diaphragms and their connections to the external elements, which can be constructed in a practical and cost effective manner in the ceiling space of a working hospital.

The proposed upgrade solution for the Lions Gate Hospital met the requirements of the NBCC (1995) for essential facilities. It provides a practical solution for the seismic upgrade of the hospital with minimum disruption to its operation. The retrofit scheme consists of the addition of external ductile shear walls in the N-S direction that effectively replaced a leaking building envelope at each end of the building, external ductile braced steel frames in the orthogonal direction, and a unique upgrade of the floor diaphragms utilizing steel plates bonded to the existing metal decking to create a continuous metal deck diaphragm independent of the thin concrete topping.

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