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Performance-based Seismic Analysis for the Retrofit of Pre-code Hospital Building in a Moderate Seismic Region

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ABSTRACT

Existing critical facilities that are seismically deficient can often benefit substantially from the use of performance-based analysis as it helps designer navigate around rigid code constraints meant for new buildings, and at the same time, better capture the actual behaviour of the structure for designing against more stringent operational requirements beyond life-safety. For critical facilities located in low and moderate seismicity regions, the value of performance-based analysis may be more apparent as the performance of the building depends more on the delicate balance of dynamic response quantities rather than the overall building strength and ductility emphasized in prescriptive code design. This study describes the seismic rehabilitation study of a pre-code steel hospital building in southern Ontario used for decision-support for a multi-billion dollar site redevelopment. Part of the target building is slated for demolition and change in the redeveloped site. However, the building itself needs to remain and continue to provide critical healthcare services during and after the redevelopment. This stringent functional requirement demands the use of performance-based design and analysis to support the decision-making.

This study follows the ASCE-41 tier 3 procedure for the rehabilitation study while guided by the FEMA P-58 analysis throughout the process to ensure that functional continuity requirements can be achieved. Field investigation and component level finite element analyses were performed to develop a structural model for carrying out nonlinear dynamic analysis for the assessment of structural demand and functional performance. Based on the deficiencies identified in the structural and non-structural systems, a minimal intervention retrofit option using passive supplemental damping was proposed and assessed against the criteria for operational performance. The study illustrates the use of fully integrated performance-based seismic analyses to understand the behaviour and develop rational measures for enhancing seismic performance of existing critical facilities.

Keywords: Performance-based design, critical facility, FEMA P-58, supplemental damping, functional recovery

INTRODUCTION

Southern Ontario is a region of low to moderate seismicity. While large and damaging earthquakes can occur in the region, they are very infrequent and most of the existing building stock has never experienced such an event. The buildings constructed prior to the introduction of modern seismic provisions in the building code likely do not meet the current code requirements, or in some cases, not designed for seismic loads at all. Hence, the seismic performance for these buildings is largely unknown, and this is often an issue when modification and alteration need to be done. In the case of critical buildings, like hospitals, the required amount of structural work to bring the building to conform with modern prescriptive building codes is often prohibitive in terms of both cost and operational disruption. As a result, seismic consideration becomes a great deterrent for modernizing these buildings despite being only exposed to minor or moderate seismic hazard.

As an alternative to satisfying the prescriptive requirements in the local building code, performance-based seismic design (PBSD) is seen as a possible solution to these situations as it allows the use of rational analysis methods to demonstrate the buildings conformance with the intended performance level stipulated in the code. This allows designers to make better use of the existing structural system and more flexibility in developing upgrade options that will ultimately meet budget and project constraints. Furthermore, developments in performance-based assessment of building structures in the last two decades [1-6] has made it possible to directly evaluate the overall building performance from an operational perspective, which allows the owners to develop a better understanding of the expected impacts of earthquakes to ensure that the desired operational performance goals are met. An application of the performance-based seismic assessment following the ASCE-41 (ASCE 2017) tier 3 comprehensive procedure for structural performance, and the FEMA P-58 methodology for overall building performance is presented in this paper for a hospital building located in Southern Ontario as part of a large campus modernization project.

DESCRIPTION OF THE PROJECT

Figure 1 shows a picture of the target building, which is one of the main wings of the regional hospital that contains the emergency unit, in patient unit amongst other critical medical services. The building was designed in 1958 with 8 storeys above grade and a penthouse. There is also a 3-storey protrusion in the original building, indicated in Figure 1, which is the existing pharmacy that is structurally connected to the main building. At the time of design, two storeys were anticipated to be added, and these were added in 1968, with the first of these storeys sitting directly on top of the existing penthouse level, which was converted into a mechanical floor. The column splices at the bottom of the new storeys appear to be designed for gravity only. Finally, an elevator frame that is connected to the main structure was added in 1970, where seismic load for the added structure alone was considered. Although seismic provision was introduced in the NBC 1953, it is unclear if seismic design had been enforced at the time of design and there was no evidence from existing drawings that seismic loads were considered in the original building and in the addition.

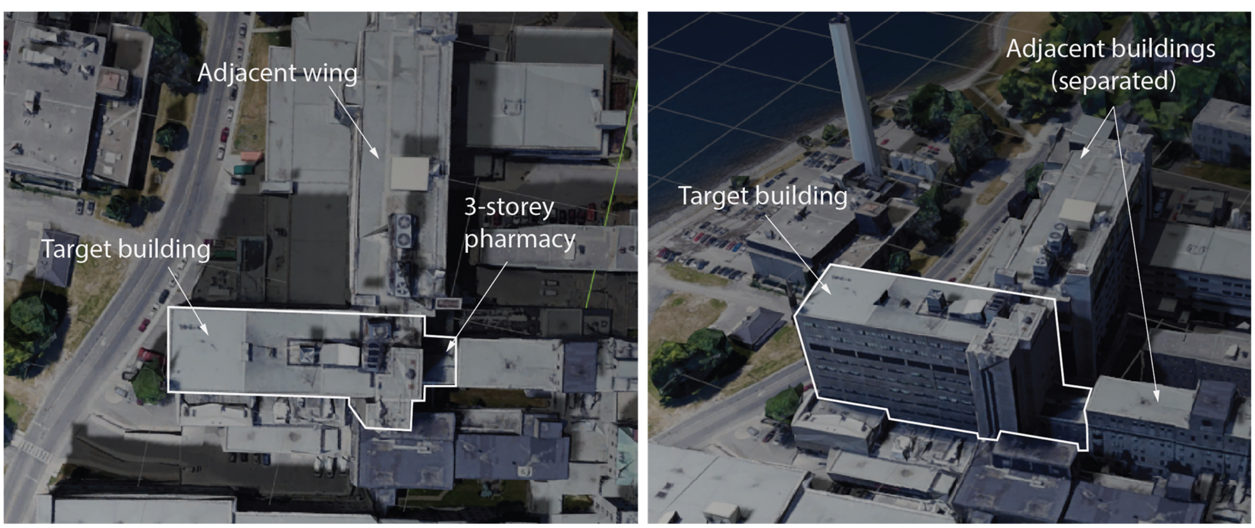

Figure 1: Photo of the hospital building

Figure 2 shows a typical framing plan of the building. The structural system consists of a steel frame with a mixture of bolted and welded partially restrained (PR) moment connections and fully restrained connections. This is not surprising given the year of construction coincides with the period of steel building boom in Canada, where systems utilizing PR connections, such as "Type 2 construction" [7] were popularized. The frame members are covered in terra cotta tiles for fire proofing. However, it was confirmed by site investigation that a gap is present between the steel frame and the terra cotta so the latter does not participate in the lateral load resistance.

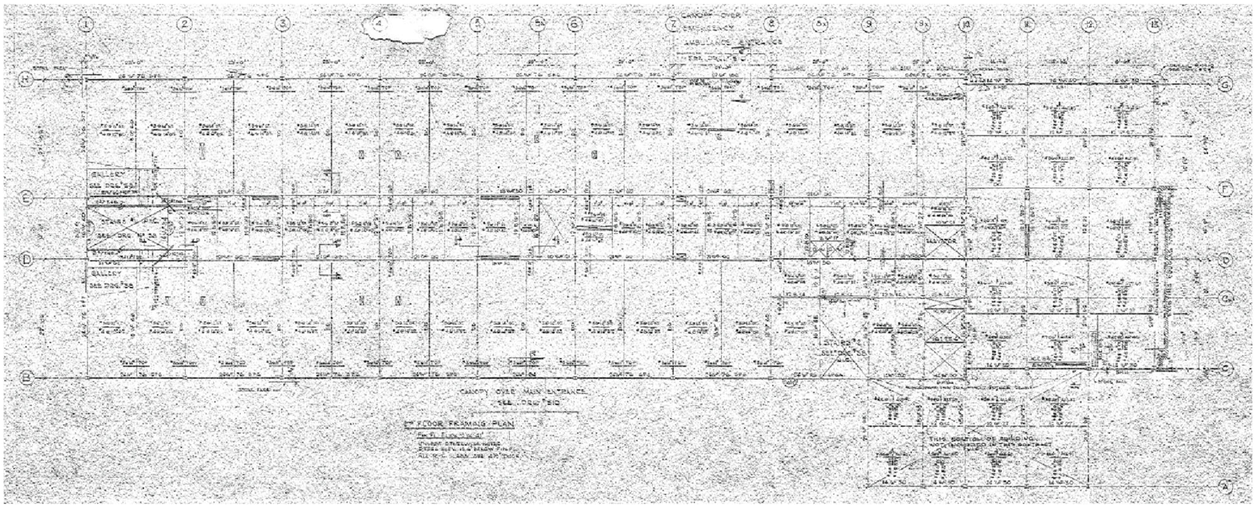

Figure 2: Typical framing plan

To accommodate the proposed modernization and expansion of the hospital campus, the 3-storey part of the building needs to be demolished and the existing pharmacy needs to relocate. This constitute a building alteration that would trigger a full retrofit of the building to bring it up to current seismic standard. However, since the building does not have a qualified lateral force resisting system recognized by the code, the retrofit will require the construction of a new lateral force resisting system, which is cost prohibitive and would require prolonged disruption of the critical functions currently facilitated by the building. Unless the proposed demolition can be done with only minor intervention, alternative options, such as rebuilding the hospital in a new site, may need to be considered. To assist decision-making of the owner in the modernization planning, the project structural team, which consists of Kinetica Risk and EXP, undertook a performance-based assessment with the following goals:

1) Assess the seismic performance of the building after the proposed partial demolition to demonstrate that the demolition does not worsen its seismic performance.
2) Propose a minimal intervention seismic retrofit that would meet the intended performance level of the current building code.

Due to the use of non-conforming connection types by modern code standard, field investigation was required to develop a good understanding of the lateral load resisting system, in particular the beam column connections. Figure 3 shows selected schematic sketches and photos of several variations of the existing beam column connections uncovered during the site visit. Based on the field investigation, most of the connection along the short building axis consists of welded flange cover plate moment connections with a bolted shear tab. The cover plate of these connections is hidden under the concrete slab above, so its dimensions were confirmed using an x-ray scan. Along the long building axis, there are joints where the beam flanges are connected to the web of the columns in the weak column axis. However, most of the connections along this direction are bolted shear tab connections with either a bolted or welded plate to the column, or a welded angle to the web of the column, which does not have much capacity in resisting moment. In some locations, the web is connected to the flange of the column in order to accommodate electrical conduits. Also, a seat angle bolted to the bottom flange can be observed in some connections. However, these are assumed to be angles used for fixing the beams during erection and are not part of the main structural system. According to ASCE-41, the connections along the short building axis is classified as a fully restrained moment connection whereas the shear tab connections are considered partially restrained connections. Note that in all cases, shear stubs that couple the beam to the concrete slab above is not confirmed, and hence no composite action is considered. Furthermore, samples of steel coupon from beams, columns and bolts were removed and tested in the lab to establish the material properties. In order to establish the performance of the building under seismic loads, it is of utmost importance to characterize the joint behaviour.

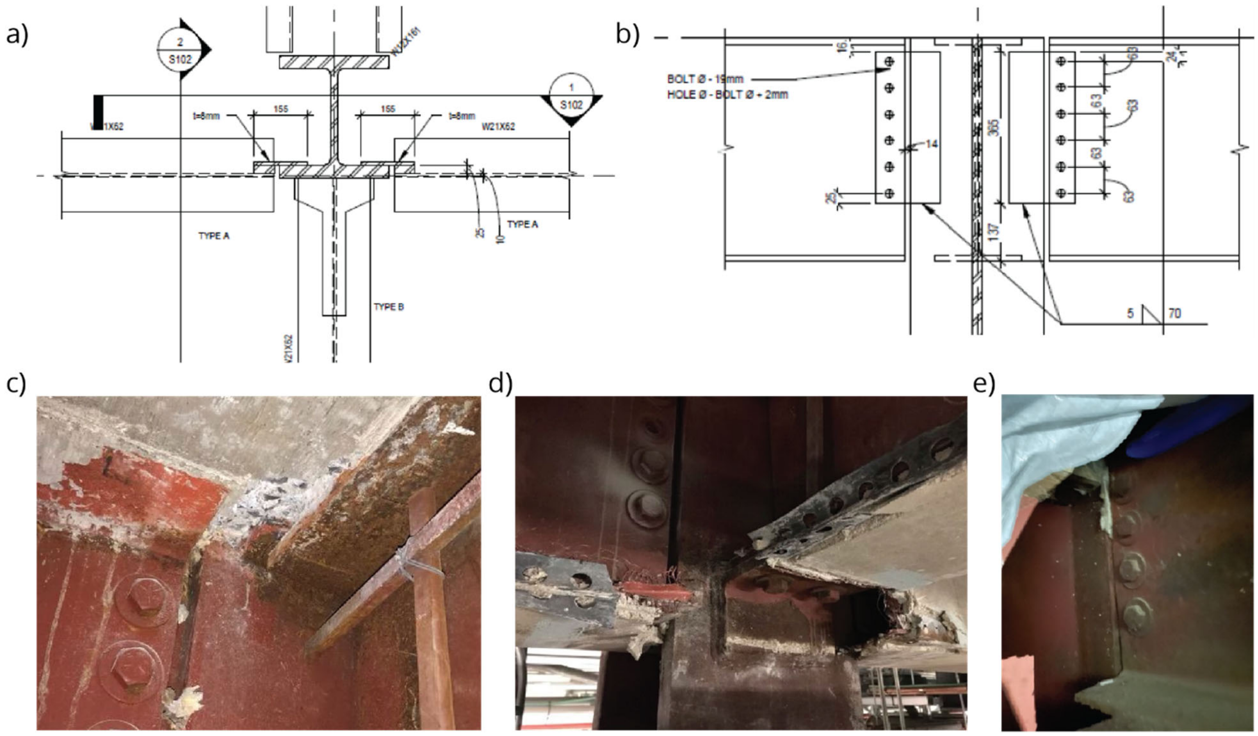

Figure 3: *Beam column joint sketches and photos a) cover plate flange connection sketch b) typical shear tab connection sketch c) photo of cover plate connection d) photo of shear tab connection with seat angle c) photo of shear tab connection to column flange*

DESCRIPTION OF SEISMIC STRUCTURAL PERFORMANCE ASSESSMENT

Material properties of the frame and connection elements were established through testing samples collected from the site visits, which are summarized in Table 1. Prior to the developing a model for the building, solid finite element analyses (FEM) were carried out in Abaqus [8] for each of the observed joint types in order to establish load-deformation relations that can be used to simulate their seismic response. The qualification protocol from the AISC 341-16 standard [9] was used to develop the reverse cyclic response histories for each joint. Slack in the bolted connection were added to the protocol to ensure that the amount of nonlinear deformation in the steel conforms with the AISC protocol.

Table 1 – Summary of steel material properties

Component	F_y (MPa)	F_u (MPa)
Rolled Sections and Plates	246	447
Bolts	634	827
Weld EE60XX	330	430

Figure 4 shows reversed cyclic analyses from two commonly occurring joints in the building. The first is a welded cover plate connection at the top with a CJP weld at the bottom connecting the beam and column flanges. Buckling of the top plate is restrained by an applied compression to simulate the concrete slab from above. It can be seen that aside from very localized weld stresses at the tip of the fillet weld, gross sectional yielding in the stem of the cover plate can be developed resulting in a stable hysteretic behaviour. Note that friction contribution to the moment rotation response between the slab and the cover plate is ignored because it is deemed not reliable. The building relies mainly on this type of joints to resist lateral forces in the short axis direction. The second type of commonly occurring connection in the building consists of shear tab connections to the beam with either a welded or bolted connection to the flange of the column along the weak axis of the column W-section. The response is shown in Figure 4b where the bolts are made invisible for clarity. This type of connection cannot develop the beam capacity, and it relies on the eccentric bolt group to develop a small moment resistance. Initially, a small amount of friction develops between the faying surface of the beam web and the shear tab. At roughly 1% rotation, bearing occurs in the bolt, and the stiffness pick up substantially. While a block tear out eventually occurs in the Abaqus analysis, the capacity of these types of joints are truncated by the theoretical capacities established using the CSA S16 eccentric bolt and eccentric weld analyses. These shear tab joints are found along the along direction of the building oriented in the weak-axis direction of the columns. Another group of joints that is found along the long building axis that resists lateral load are welded angle to beam flange connections. These joints develop asymmetrical moment rotation response as the angle-flange connections are stronger in compression than in tension.

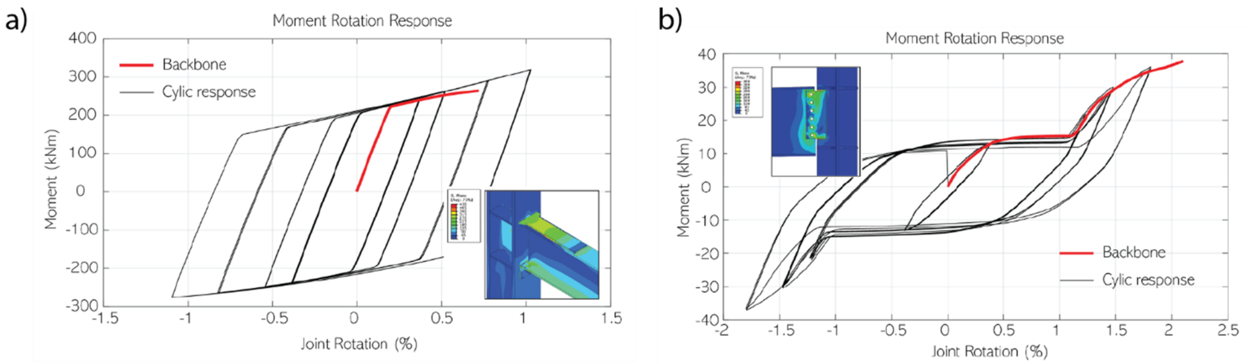

Figure 4: Abaqus cyclic reversal analysis for the characterization of joint behaviour a) typical welded cover plate connection b) typical shear tab connection

A Perform3D [10] steel frame model is developed with joint responses calibrated using the Abaqus reverse cyclic results. Figure 5 shows the Perform3D model and a sample calibration for the joint. The model uses fiber columns and elastic beams with inelastic connections. For joints with the first type of moment connections, a beam with hinge is used where the hinge moment-rotation properties are calibrated to match the Abaqus results. For joints with the second type of moment connections, or any variations mentioned above, the model initially uses the beam with hinge model that captures only the friction effect to reduce runtime. A rotation tracker is added to these joints to detect bearing. If bearing is detected, then the model is updated to include an explicit joint assembly that reproduces the two-phase slip and bearing behaviour shown in Figure 4b. This is done by creating a lever mechanism around the joint using rigid elements connected to the beam-column joint node. The tips of the levers are connected to uniaxial fibre section that models the bearing response, and it is connected in series to a gap element that captures the slack in the bolted connections and an elastoplastic element that captures the bolt friction. The resulting joint hysteretic response is compared against the Abaqus expected behaviour in Figure 5 for the shear tab connection where the shear tab is connected to the column flange. Finally, each joint is given a corresponding rotational capacity based on whether it is fully or partially restrained, and its connection type in the ASCE-41.

The first three fundamental periods of the building prior to the proposed demolition are 4.44s, 3.54s and 3.30s. The modes correspond to lateral translation along the long axis, lateral translation along the short axis, and torsion, respectively. After the proposed demolition, the periods become 4.46s, 3.56s and 3.31s, which are very similar. Figure 6 compares the pushover analysis using the NBC equivalent static force pattern before (base case) and after (demolition case) the proposed demolition for the long axis (N-S direction) and short axis (E-W) directions. The building does not develop a clear yield point in both directions. The point at which the first joint exceeds the immediate occupancy (IO), life-safety (LS) and collapse prevention (CP) joint rotation limits in the ASCE-7 are indicated for both buildings. The primary mechanism for developing lateral resistance is through the connections where the flanges of the beams are connected. However, the large number of shear tab connections also make significant contribution to the initial stiffness and strength. The successive slipping of the shear tab joints is the main reason for the gradual softening seen in the pushover curve in both directions. While these connections can sustain large deformations before failure, the overall structure does not develop a clear yield point and plateau because the slippage of the connections do not occur simultaneously.

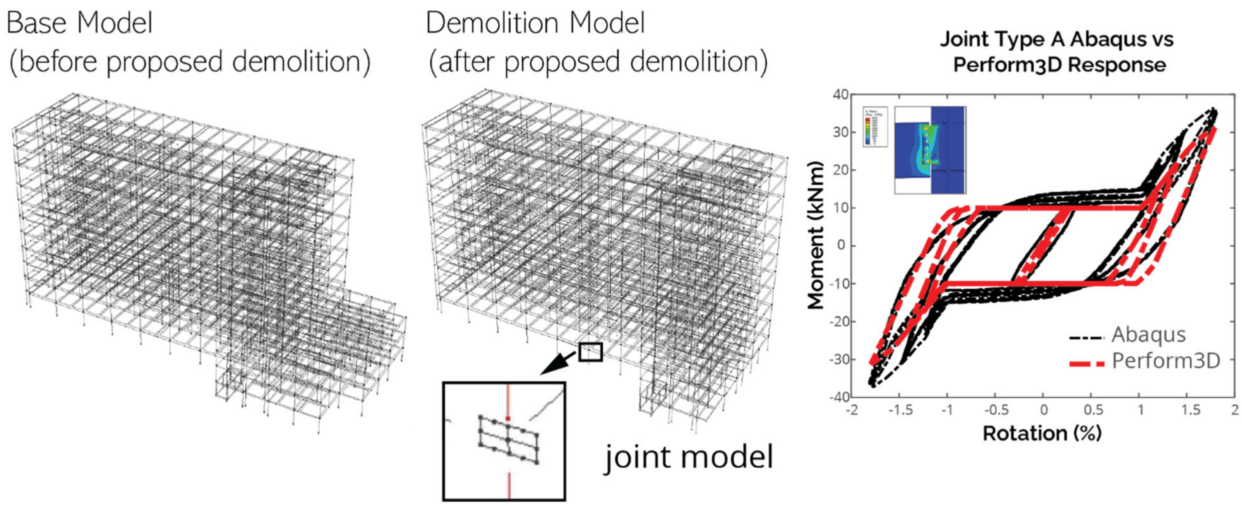

Figure 5: Perform3D frame model and sample joint calibration

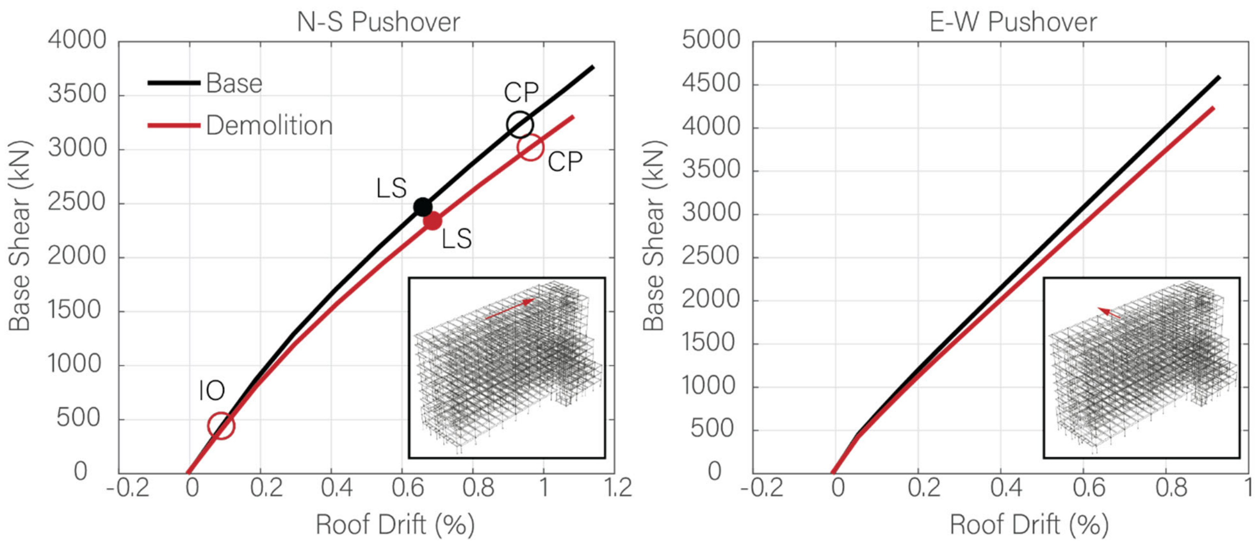

Figure 6: Comparison of the pushover response before and after the proposed demolition

A suite of 11 earthquakes selected and scaled to the NBC2020 (NRC 2020) target spectrum based on the requirement of the NBC structural commentary J is applied to the building to perform nonlinear dynamic analysis for the Base building and the Demolition building. Despite being a moderate seismic region, the soil class at the site of the hospital building is class A, which results in a substantial reduction of the hazard. Due to the rarity of strong earthquakes in the area, the selected suite of ground motions consists of both historical earthquakes and synthetic records [11]. Figure 7 summarizes the response spectra of the scaled earthquakes and the target spectrum. Figure 8 presents the drift and the storey shear response of the two buildings for the two orthogonal axes. The plots show 13 levels of which the first 8 levels are part of the original 1958 building, levels 9, 10 and 11 are the 1968 addition, and a small mechanical penthouse that occupies a corner of the roof makes up level 12 and 13. A clear increase in drift can be seen for the levels that were added in 1968. The main reason is due to the gravity only column splice connections at the base of these floors when they were added. In the E-W direction, the storey drifts at these floors exceed the requirement for post-disaster buildings equal to 1%. While this is an obvious deficiency in the building, based on the drift and storey shear responses there is not much difference before and after the proposed demolition.

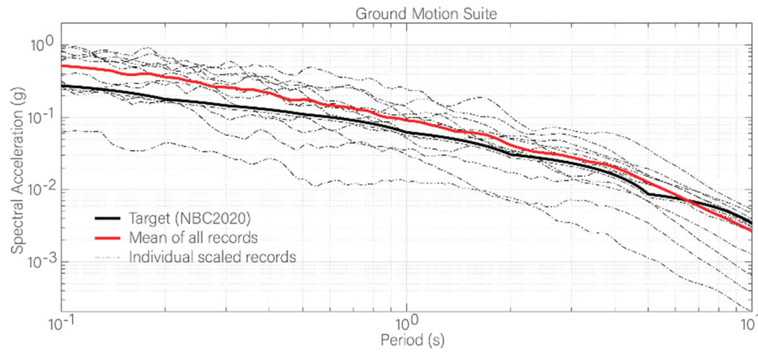

Figure 7: Scaled ground motion suite for the hospital in Southern Ontario (site class A)

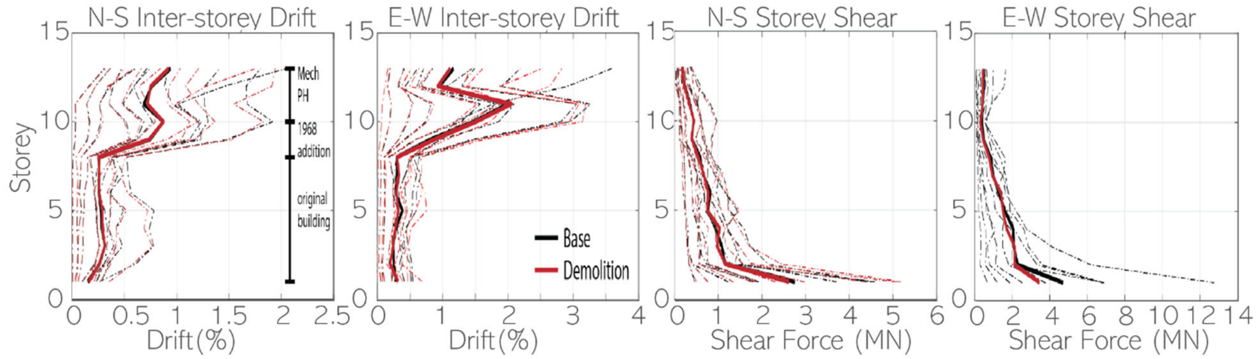

Figure 8: Global drift and storey shear response for Base and Demolition building

Figure 9 provides a visual summary of the post-processed building result in terms of joint rotation demand capacity ratio (DCR) measured against the ASCE-41 limits for full and partially restrained moment connections. The color of the frame represents the largest of the i-th or j-th node connection DCR measured against the ASCE-41 LS plastic rotation capacity.

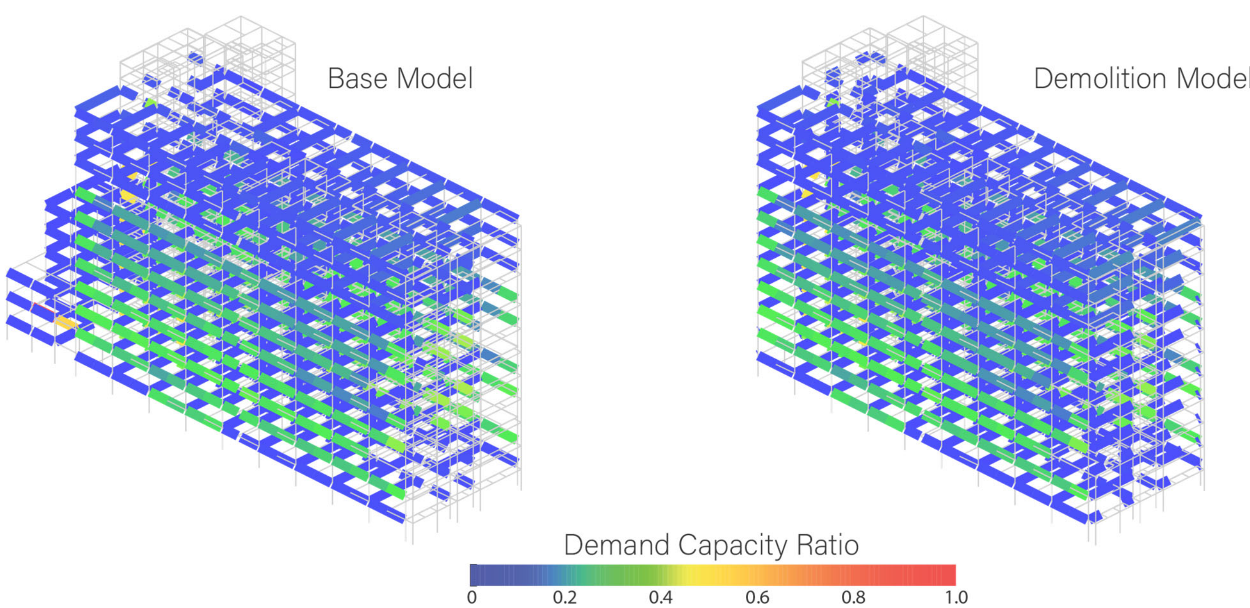

Figure 9: Beam-column joint rotation DCR against LS performance criteria

Similar to the building results presented above, the behaviour of the Base and Demolition buildings are nearly identical, with the vast majority of the element DCR values being within 10%. Furthermore, all joint rotations are less than the ASCE-41 LS limits, with the largest joint rotation reaching about 50% of the limit. Given that many of the joints contribute to the seismic response by allowing small friction slips to dissipate energy, the uncertainty associated with the friction strengths of these joints is a critical aspect that was investigated using a sensitivity analysis where the friction strengths (bolt prestressed force) are varied. For brevity, the results are not presented. While the performance of the building is affected to an extent by the friction in the shear tab joints, these analyses do not change the conclusion above pertaining to the change in behaviour before and after the proposed demolition. Hence, regarding the first goal of the assessment, the nonlinear dynamic analyses did not find evidence that the seismic performance of the building is worsened by the demolition. However, the building is obviously deficient seismically due to its reliance on non-conventionally, and potentially non-reliable mechanisms for lateral resistance. To rectify this, the second part of the assessment is to develop a retrofit scheme that causes minimal disruptions to the hospital operations while meeting stringent operational requirements.

DEVELOPMENT AND VALIDATION OF THE STRUCTURAL RETROFIT WITH PASSIVE DAMPING

The second goal of the study is to develop a seismic retrofit scheme to accommodate the future use of the building after the planned modernization. Since the building does not have a qualified lateral force resisting system per the current building code, a performance-based approach needs to be used in order to avoid adding brand new structural systems, which is prohibitively costly and disruptive. In addition to this consideration, there are several project constraints that were considered by the structural team. Namely, the structural upgrade must minimize the disruption to the existing emergency and in-patient units, it should provide a level of structural performance similar to what is expected of a new hospital building, and it should minimize the post-earthquake functional disruption.

Since the building was not designed for earthquake forces, a significant increase in stiffness would likely require substantial strengthening of the existing cast-in-place concrete diaphragm, and the foundations. Furthermore, increased stiffness will increase floor accelerations causing further damage to the mechanical, HVAC and architectural components to the building, which were not designed and anchored in accordance with modern seismic standards. From a cost and functional disruption perspective, it was desirable to maintain low stiffness and use passive supplemental damping to control drift and acceleration. A flexible structure also has the added benefit of being dynamically insensitive to the earthquakes in central eastern North America where most of the energy contents concentrate in the high-frequency band, and are short in duration [11]. As a result, a retrofit that involves strengthening of selected exterior joints and adding passive supplemental fluid viscous dampers was proposed. A schematic of the proposed retrofit solution is shown in Figure 10 showing two frames along each direction upgraded to develop full moment capacities of the beams with supplemental fluid viscous dampers. The P-Spectra method [12] was employed to design the dampers, targeting 10% added viscous damping in both building directions. The required damper distribution is expressed in terms of viscous constants per storey, which are summarized in Table 2.

Figure 11 compares the global response of the Base building and the Retrofit building, which implements the proposed retrofit with strengthened exterior frames and supplemental fluid viscous dampers. It can be seen the drift is controlled to only a fraction of the original structure, which satisfies the 1% drift limit stipulated by the National Building Code. The exception to these are the penthouse levels (floors 12 and 13), which were not considered as part of the proposed seismic retrofit. As desired, the storey shear forces of the main frame are comparable to, or smaller than the original structure, which will protect the existing diaphragms and foundations. A performance-based check using the ASCE-41 immediate occupancy (IO) deformation limits were performed and the result confirms that all retrofitted beam and column connections, all column and beam hinge rotations meet the IO performance criteria. Hence, structurally, the retrofitted structure meets IO performance as per ASCE-41. Additionally, a performance model of the building is developed and a FEMA P-58 loss assessment as well as a REDi functional recovery assessment were performed to better understand the requirements for bringing the retrofitted building to Operational performance under the 2% in 50 year event in the NBC 2020. These study is described in the next section.

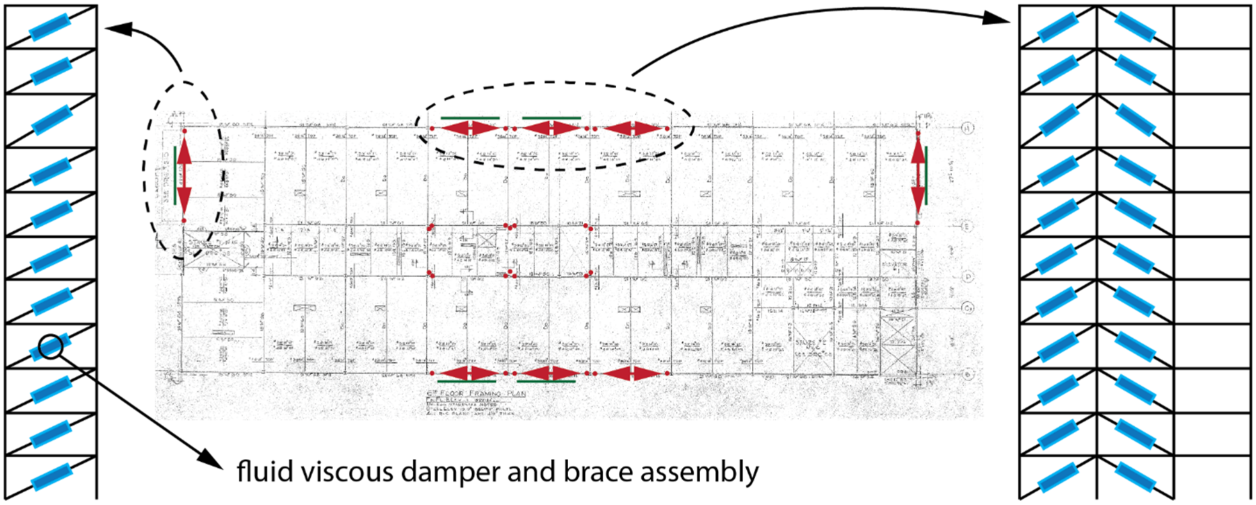

Figure 10: Schematic of proposed seismic retrofit

Table 2 – Summary of calculated required fluid viscous damper properties

Storey	Short Direction Frame Damper (x2)	Long Direction Frame Damper (x4)
10	4.51 KNs/m	0.43 KNs/m
9	4.65 KNs/m	0.50 KNs/m
8	3.94 KNs/m	0.61 KNs/m
7	9.83 KNs/m	2.68 KNs/m
6	11.12 KNs/m	2.77 KNs/m
5	13.77 KNs/m	3.38 KNs/m
4	13.39 KNs/m	2.78 KNs/m
3	16.09 KNs/m	4.61 KNs/m
2	15.22 KNs/m	6.27 KNs/m
1	12.62 KNs/m	8.48 KNs/m
Emergency	9.54 KNs/m	3.31 KNs/m

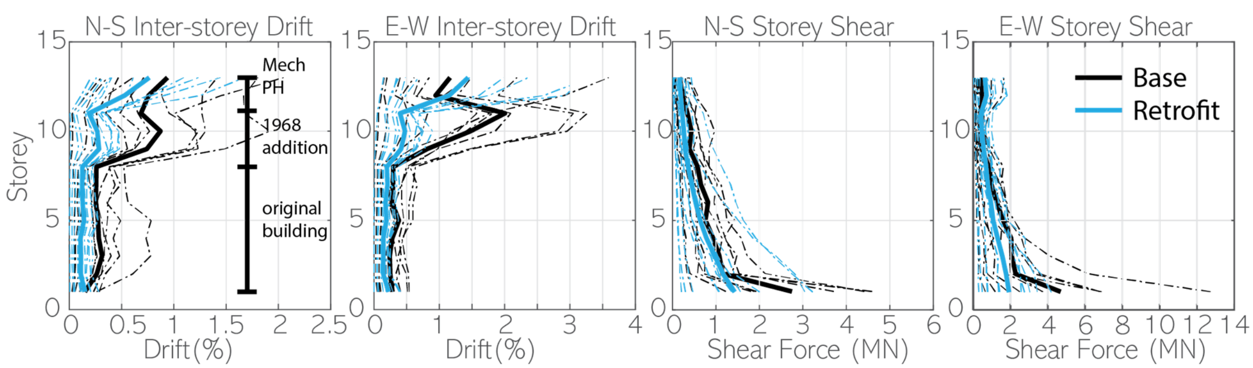

Figure 11: Global drift and storey shear response for Base and Retrofit building

PERFORMANCE ASSESSMENT OF THE RETROFITTED BUILDING

The S-RAMP software [13] was used to construct the performance model and perform the FEMA P-58 and REDi analyses in order to determine the seismic loss and recovery time. The purpose of the assessment is to inform the decision maker on the scale of capital investment required to bring the non-structural components and the rest of the building to operational performance under the current code level earthquake. For the purpose of this study, "Operational" performance is deemed to be satisfied if the median functional recovery time is 0. Since the main intent of the analysis is to demonstrate the feasibility of achieving operational performance, the FEMA P-58 normative quantity database was used to generate the hospital contents by floor in lieu of a comprehensive contents survey. Elements in the mechanical floor (level 8) were adjusted based on field observations conducted during the site investigation. An intensity-based analysis using 5000 simulations was conducted to assess the building performance, and Figure 12 summarizes the structural response parameters (inter-storey drift, residual drift, peak floor velocity and peak floor acceleration) used for the FEMA P-58 assessment. It can be seen that except for the penthouse levels, which were not considered in the retrofit, the retrofitted building maintains generally very small drift, velocity, and most importantly, floor acceleration owing to the flexible structural system and supplemental damping. While damage is still expected to occur to some mechanical elements, the severity will be small to moderate and spot improvements can be used to remedy these.

Figure 13 summarizes the loss assessment results in terms of mean and median loss and downtime. Since the analysis considered only the 2% in 50 years event, the annualized loss was not evaluated. The mean direct loss of $4.47M is less than 3% of the replacement cost of the building, and is considered minor. Most of the losses are attributed to exterior terra cotta partition damage, mechanical damage, and structural damage in the steel joints. The mean and median functional recovery times, on the other hand, are on the order of 5 months, which is very significant. As mentioned above, the functional recovery time is assessed using the REDi methodology, and it is widely recognized that several of the assumptions made by the REDi method may not be realistic. One of these assumptions is that the building is not functional unless 100% of its pre-earthquake function has returned. In reality this is not true as reactionary measures put in by tenants and occupants tend to allow a much quicker partial recovery, which are not accounted for in the REDi procedure. However, the bottlenecks to recovery identified in this process are still very useful in informing decision makers the approximate scale of the problem and potential solutions.

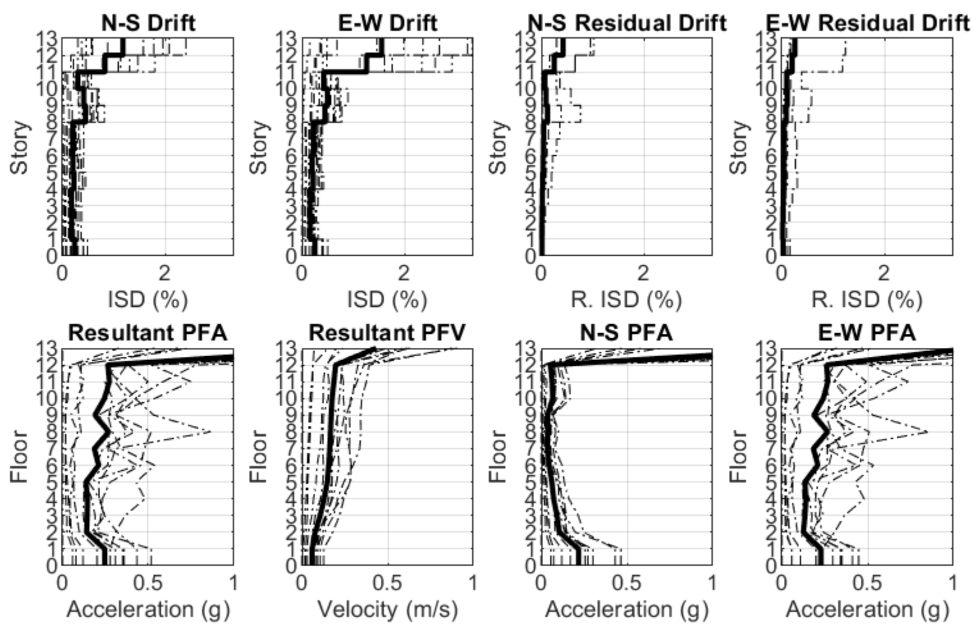

Figure 12: Summary of structural response parameters used for the FEMA P-58 intensity-based analysis

Since the goal is to achieve 0 median recovery time, realizations around the median functional recovery of 158 days were further examined to identify specific bottlenecks. These were found to be unbraced terra cotta partition damage, elevator damage, suspended lighting damage, damage in drywall partition and finish (beyond cosmetic), as well as damage to mounted equipment in the mechanical storey including air handling units (AHU), motor control centre, chiller and diesel generator. Some of these elements can be readily remedied using inexpensive details. On the other hand, bottlenecks like elevator damage is extremely difficult to eliminate. However, it is not necessary to fix everything in order to achieve zero median functional recovery since the elimination of each bottleneck will reduce the probability of failure in all scenarios, not just the median case. A proposed non-structural upgrade scope involving installing out of plane bracing to unbraced terra cotta walls, add independent suspension wires to lighting throughout the building, and isolating the large AHUs, chiller, motor control panel, and the switch gear on the mechanical floor was enough to reduce the median recovery time to zero under the 2% in 50 years event. Note that aside from the terra cotta partition and suspended lighting which are found throughout the building, all other elements are concentrated in the unoccupied mechanical storey. Work here is anticipated to cause minimal disruption to the operations.

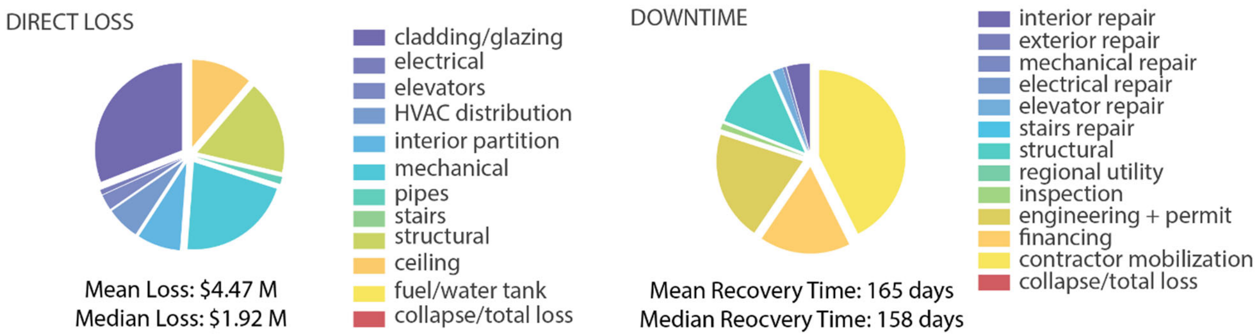

Figure 13: Summary of loss and recovery time, and component-wise contribution to risk

CONCLUSION

Modifications and alterations of existing pre-code hospital buildings can be an extremely challenging task because their seismic performance cannot be assessed by conventional code methods. Even when such method is possible, prescriptive code requirement will likely result in prohibitively expensive upgrade requirements that can deter such efforts. Often, the end result is either nothing gets done, or a brand new development is to occur elsewhere which is not a sustainable model for infrastructure renewal. In these situations, performance-based seismic design offers a way to consider more practical options to deal with these building stocks without sacrificing performance.

The present paper describes a case study of a 1950's pre-code steel hospital building located in a low to moderate seismicity region in Southern Ontario undergoing a major redevelopment to accommodate new healthcare needs in the area. Part of the building requires demolition to accommodate the redevelopment, while the remaining structure needs to continue to facilitate healthcare services in the future. The building does not have a lateral force resisting system recognized by the code, and a performance-based seismic assessment following the tier 3 comprehensive procedure in the ASCE-7 was carried out to develop a good understanding of its seismic performance in order to better inform the owners about the feasibility of the proposed redevelopment. Through on-site inspection and detailed finite element modelling of the different variations of the beam-column joints, the mechanisms through which the existing building resists seismic load was determined. A nonlinear Perform3D model was then developed to assess the impact of the proposed demolition using nonlinear dynamic analysis. It was found that despite not having a robust structural system, the building is able to resist the earthquake loads through limited joint resistance developed by the gravity framing before and after the proposed demolition. Furthermore, by using the same tier 3 procedure, a proposed retrofit involving selective strengthening of the exterior moment frame joints and passive supplemental viscous damping was shown to meet ASCE-41 immediate occupancy structural requirements, which is comparable performance to newly designed hospital buildings conforming to the NBC 2020. Subsequent FEMA P-58 and REDi assessments were also performed to inform the hospital owners of additional non-structural work that needs to be done to achieve an operational building under the 2% in 50 years event, which supports the long-term planning of the development. This study illustrates that the fully integrated performance-based seismic assessment tools can be effectively utilized to guide major capital investments that may otherwise end up being more costly or less sustainable from a long-term perspective.

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