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RETROFITTING OF AN EXISTING BUILDING WITH CONCRETE WALLS AND STEEL FRAMES FOR GRAVITY LOADS AND PROGRESSIVE COLLAPSE RESISTANCE

Florea Dinu[1,2], Calin Neagu[1], Dan Dubina[1,2], and Constantin Stanciu[3]

[1] Politehnica University Timisoara, CMMC Department
Ioan Curea 1, Timisoara, Romania
e-mail: florea.dinu@upt.ro; calin.neagu@up.ro; dan.dubina@upt.ro

[2] Romanian Academy, Timisoara Branch
Mihai Viteazul 24, Timisoara, Romania

[3] Structural ING LTD,
Bd. Tomis, 151, Constanta, Romania
e-mail: stanciu@structuralgroup.ro

Abstract

The building subjected to intervention is composed of concrete walls and steel frames and is located in a moderate seismic area. The steel frames are made with partial strength joints and are designed to carry gravity loads only. The building has 11 above grade stories, with a lateral setback starting from the 8th story. The main structure was completed in 2003, but without installing the enclosure walls and roofing. Since then, most code provisions employed in the initial design were updates several times. Also, due to the aggressive environmental conditions, characteristic to the sea climate (the building is located close to the sea cost), the corrosion affected the steel frame elements (beams, columns, connections) and the profiled steel sheeting of the composite floors. In addition to the retrofitting intervention, the new architecture asked for completing the setback till the last story. The study presented in the paper summarizes the structural assessment and proposed interventions for retrofitting the gravity load system and improving the progressive collapse resistance using alternate path method and nonlinear static procedure. Ongoing studies will also consider more advanced analyzes using non-linear dynamic analyzes.

Keywords: existing building, concrete wall, steel frames, corrosion, seismic design, robustness, progressive collapse.

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1 INTRODUCTION

Many buildings or other types of constructions remain unfinished at various stages, put on hold, or suffer serios time delays from the schedule. One of the best-known examples is the Jeddah Tower, which was originally designed to break the records and rise to more than 1 km in height. With less than half the total height built, the construction works were halted and not yet resumed since 2018 [1]. Before resuming the works, constructions may require extensive structural assessment to determine the condition of the structure, identify the damages due to environmental exposure, and provide recommendations for intervention.

The investigated building is an existing construction, which has one story below grade and 11 story above grade (Figure 1). The main structure has been completed in 2003 but remained unfinished. With no enclosure walls and roofing, the construction remained unprotected against weather conditions till today. The building was initially designed to be used as a hotel. In plane, the structure consists of two rectangular zones, linked by a central area (Figure 2). The smaller rectangular zone is rotated by 20 degrees from the larger one. The building has a lateral setback from the 8th story which provides a transition till the last story. The floor height is 3.45m except the ground floor which has a height of 4.45m. The overall length is 55.30 m, the width is 13.30 m, and the maximum height is 39.10 m. The structural system is composed of reinforced concrete walls and piers, and steel frames. The reinforced concrete walls are 30 cm or 40 cm thick (Figure 3.a) and are designed to resist the lateral loads without any contribution from the steel frames, which are designed to carry gravity loads only. The floors are made from relatively thin slabs supported on roll-formed profiled galvanized steel sheet (Figure 3.b). Note that the floors were designed as composite floors, with the steel deck acting as the tensile reinforcement. The concrete class is C25/30 both in walls and floors.

Figure 1: View of the existing building

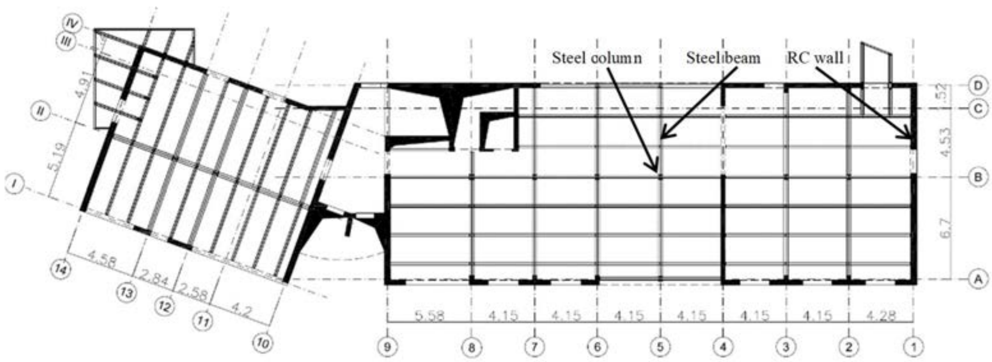
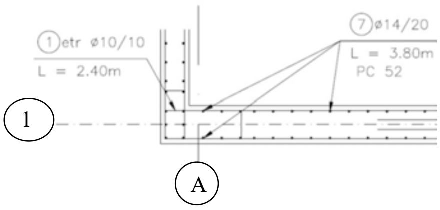
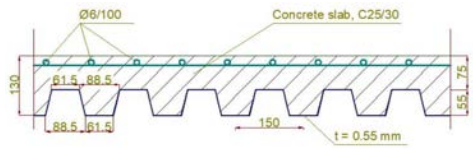


Figure 2: Current floor plan

The columns are made from HEB300 profiles, main beams from HEB280 profiles, and secondary beams from HEA180 profiles. Beam-to-column connections in the steel frames are extended end plate bolted connections with 4 rows of bolts M20, class 10.9 and 20 mm thick endplates, see Figure 4a. Connections are designed as partially restrained (partial strength, semi-rigid), see Figure 4b.

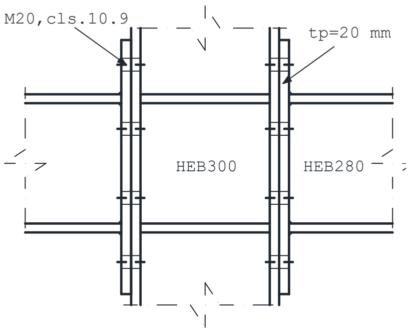


a)

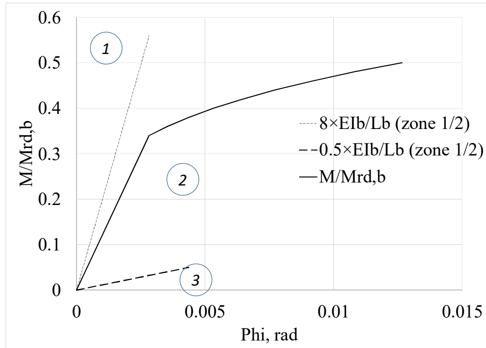


b)

Figure 3: Details of concrete wall (a) and concrete slab (b)



a)



b)

$M_{rd,b}$ = beam's moment resistance

I_b = beam's second moment of area

Zone 1 - rigid; Zone 3 – pinned; Zone 2 – semi-rigid (classification acc. to [2])

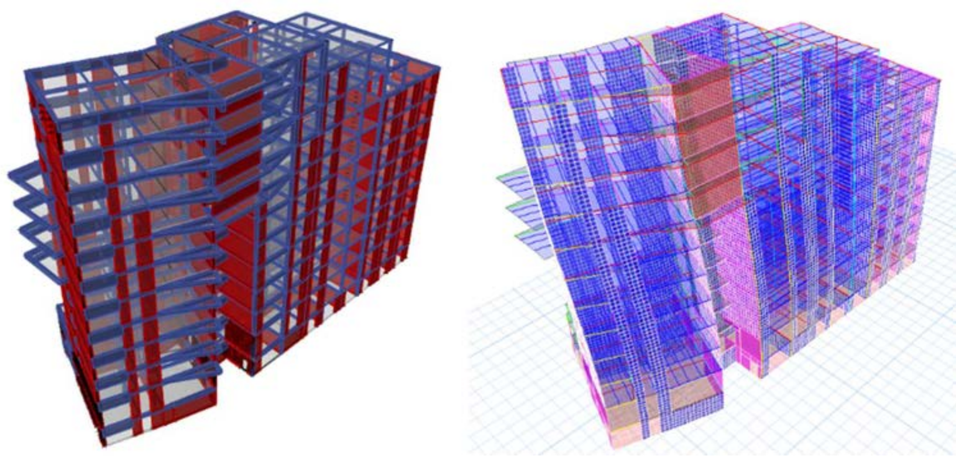
Figure 4: Beam-to-column connections in steel frames: (a) detail of the connection; (b) characteristics of the connections according to EN 1993-1-8 [3]

2 EVALUATION OF THE CONDITION OF THE STRUCTURE AND RECOMMENDATIONS FOR INTERVENTION

2.1 Structural deficiencies and deteriorations due to environmental exposure

The main structure has been completed in 2003, but without execution of the building enclosure and roofing. The structural assessment commenced with the evaluation of the existing building, the structural condition, and the code compliance. To verify the condition of execution, the state of damage in the elements, and the properties of the materials (concrete class in walls, frames, and slabs, steel grade in elements and bolts), site inspections and experimental tests were employed. The results confirmed the reinforcement and concrete class are those indicated in the project. For the steel structure, due to a mixed use of two steel grades, a single class, i.e., S235, was to be considered. The modal analysis was done using Etabs program. The results showed that the first three fundamental modes of vibrations have significant torsional components (T_1 = 0.74 s, T_2 = 0.64 s, T_3 = 0.44 s), due to high structural geometric irregularities both in plane and in elevation, see Figure 5.

The seismic action effects were evaluated using the response spectrum analysis and a behavior factor q = 3, which is the maximum value for a torsionally flexible system ([4], [5]). When compared to the old seismic code P100/92 [6], which was in force at the time of construction (i.e., 2003), the new code P100/1-2013 [5] brings an increase in seismic design acceleration of about 66 % for stiff soil conditions, see Figure 6.



a) b)

Figure 5: Structural analysis of the existing structure: (a) 3D structural model; (b) first mode shape

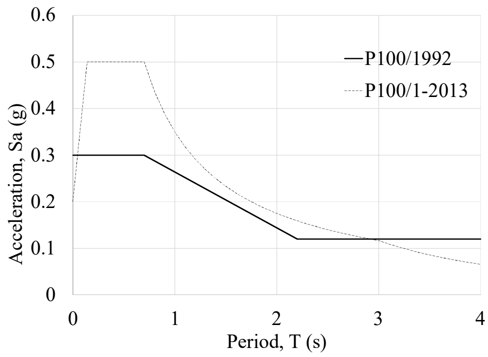
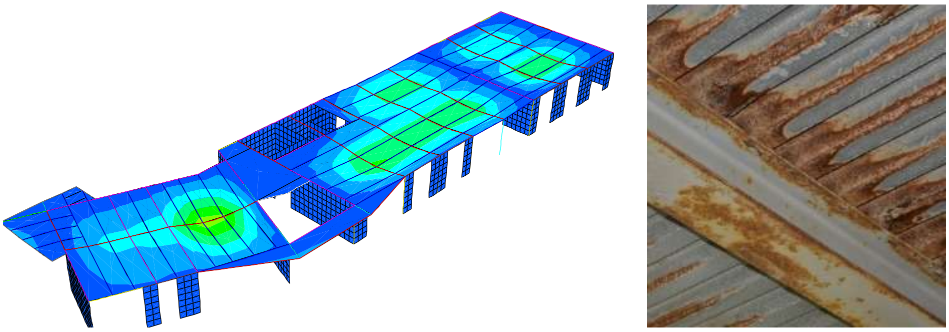


Figure 6: Seismic elastic response spectrum from [6], used in initial design vs actual response spectra from seismic code in force today [5]

The main conclusions that resulted from the structural evaluation were the following:
- The first modal shapes are predominantly torsional. This has a detrimental effect and increases the seismic demands on some members.
- The concrete walls and coupling beams are designed with safety margins for persistent and seismic design situations.
- The bearing capacity of the steel columns in the lower stories and of the beam-to-column connections are exceeded in the persistent design situation (note that they are designed for gravity loads only).
- The vertical deflections in the area 10-12/I-IV exceed the serviceability limit state conditions, i.e., L/250 for secondary beams and L/350 for the main beams (where L is the beam span) (see Figure 7a).

In addition, the aggressive site conditions (seaside) and absence of building enclosure caused extensive deteriorations of the profiled steel sheeting and localized corrosions in steel beams, columns, and bolted connections (see Figure 7b).



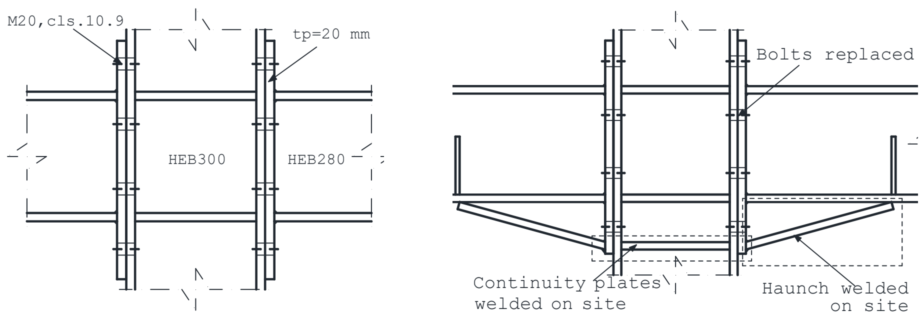
a) b)

Figure 7: Deformed shape due to gravity loads, current floor (a) and extensive corrosion in the steel sheeting, localized corrosion in the steel beams (b)

2.2 Recommendations for intervention

The basis for the intervention were the structural deficiencies and the deteriorations caused over time by the insufficient corrosion protection. Additionally, the completion of the setback was required by the client, and this would bring more gravity and seismic loads on the structure. As a result, the proposal for structural interventions included the following:

- Introduction of two-story vertical X braces between the ground floor and the 6th floor in the central area (axes A/9-10). These braces improve the overall behavior and link the two rectangular zones within the central area (Figure 9).
- Strengthen the columns by means of supplementary plates welded on the flanges; the completion of the setback increased the gravity loads on the lower steel columns and the decision to strengthen the columns applied to columns up to the 4th floor.
- Strengthen the beam-to-column connections by means of haunches and continuity plates in the columns (see Figure 8); the bolts have also been proposed for replacement with new ones. The size of the haunch was limited such that, after the intervention, the column remains stronger than the beam.
- Strengthen the main beams in the axes II/10-12 with a T section welded at the bottom flange.
- Due to the varying positions of the reinforcement and the extensive deterioration of the profiled steel sheet (which serves as the bottom reinforcement under sagging moments), the concrete slab was proposed for strengthening by pouring an additional concrete layer. The interaction between the existing slab and the new concrete slab would be done by means of chemical/mechanical anchor bolts. The strengthening solution and the experimental results may be found in [7].



a) b)

Figure 8: Strengthening/repairing of the beam to column connections: a) actual beam-to-column connection; (b) connection strengthened with additional haunches, continuity plates, and partial length beam stiffeners

The completion of the setback was done using same sections for steel beams and columns as those used in the existing structure. The beam-to-column connections were made using the solution proposed for retrofitting the existing connections (see Figure 8.b). The connection between the new steel columns and the existing ones can be done with direct welding, where the new column sits on an existing column (e.g., axis B2), or by using additional gusset plates connected with the concrete walls by means of chemical/mechanical anchor bolts, where the new column sits on the concrete wall, e.g., axis D1 (more details can be found in [8]).

The completion of the setback and the introduction of additional X-braces reduced the structural irregularities and improved the global behavior, as the first two modes of vibration changed from torsional to translational. However, the periods of the first three modes of vibration have not changed significantly (T_1 = 0.76 s, T_2 = 0.72 s, T_3 = 0.53 s).

a) b)

Figure 9: Building after interventions: (a) rendered view with the new architecture; (b) 3D structural model including the proposed interventions

3 EVALUATION OF THE RESISTANCE AGAINST PROGRESSIVE COLLAPSE

3.1 Numerical modelling

The structural robustness and prevention of progressive collapse represents specific safety conditions according to modern codes [9], [10]. The design for robustness of building structures can be done considering either the direct effects of an extreme action or a certain extension of the damage following an unknown/unforeseen event using the Alternate Path method ([11] [12]). The methods in the first category require clear identification and definition of the accidental action. Typical examples are fire, gas explosion, or impact. In some cases, the development of local damage may be allowed, but not to a disproportionate extent to the original cause ([13], [12]). For example, the initiation of a fire in a compartment (localized fire) can lead to high temperatures in the adjacent columns, which can fail in buckling due to reduction in stiffness and strength, being practically removed from the structural system. For most of the structures, the potential accidental actions are mainly unknown, so for this kind of situation the structural design would imply robustness strategies based mostly on limiting de propagation of failure.

The structural system investigated in the study is made of two main parts, i.e., a concrete wall part and a steel frame part. The reinforced concrete walls constitute the main lateral load resisting system and are designed to resist the wind and seismic loads without any contribution from the steel frame system. The moderate level of utilization in the concrete walls and the in-place seismic design requirements indirectly provide adequate levels of safety against other accidental loading conditions [13], [14]. The steel part however is more sensitive to local accidental failure, due to the possible lack of continuity and ductility, especially at the level of beam to column connections ([15], [14], [16]). As a result, the structural robustness was tested for column loss scenarios using the alternate path (AP) method and nonlinear static procedure (NSP), in accordance with provisions from [17], [9]. The AP method ascertains the capacity of a structure to resist the loss of one or more critical load-bearing elements without causing disproportionate collapse. In the NSP, the column is deleted from the model and the structure is subjected to gravity loading. For the analysis, the gravity load on the bays immediately adjacent to the lost element and on all floors above is given by:

$$G_N = DIF \times \left[DL + 0.5LL\right] \tag{1}$$

where GN is the increased gravity load for nonlinear static analysis, DL is the dead load, LL is the live load, and DIF is the dynamic increase factor accounting for the dynamic effects of the column loss (see eq. 3).

The combined load on the areas of the floor away from the lost column is given by:

$$G = \left[DL + 0.5LL\right] \tag{2}$$

where G is the gravity load.

The column loss locations included two scenarios, i.e., internal column B5 and perimeter column A5, see Figure 10. These two scenarios cover all possible scenarios involving a column loss from the ground floor. Thus, conditions of scenario B5 applies also to the other internal columns, which have similar tributary areas and steel frame members and connections. Also, scenario A5 applies to scenarios D5 and D6.

The numerical modeling of progressive collapse requires full three-dimensional models of the structure. However, in the particular case of the structure under investigation, the model can be reduced to the area between axis 3 and 8, with appropriate consideration of boundary conditions. Thus, the area between axis 3 and 8 and represented with dashed line in Figure 10 is bordered by the longitudinal concrete walls and piers that can stop the propagation of collapse beyond this area. Even not intentionally put in practice in the original design, the segmentation of a structure using strong borders is an efficient technique to limit the extent of collapse [13].

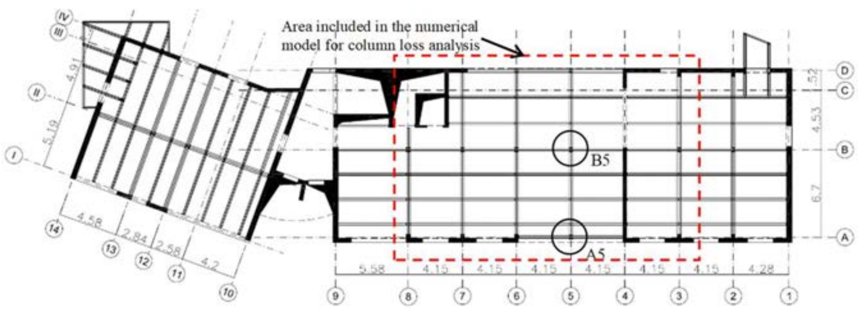


Figure 10: Plan view with the column removal locations, A5 and B5

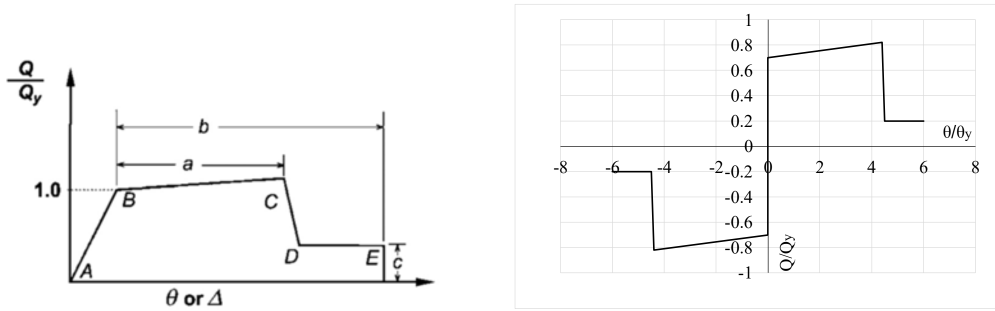
The progressive collapse analysis was done using Etabs program. Columns were modeled as elements with discrete plastic hinges located at both ends, of axial-moment interaction type P-M2-M3. The modelling parameters were taken from ASCE/SEI 41-13 provisions [18], see Figure 11a. Figure 11b plots the M3 plastic hinge behavior adopted for the beam ends in the transversal frames. Based on provisions from [18] and considering that the component that yields is the beam connection and the failure mode is yield of end plate (mode 1 according to [2]), the plastic rotation at maximum force amounted 42 mrad. Similar values are reported in experimental tests [19].

The calculation of the dynamic increase factor (DIF) was done according to UFC provisions [17] using eq. 3:

$$DIF = 1.08 + \frac{0.76}{\frac{allowable\ plastic\ rotation}{yielding\ rotation}+0.83} \tag{3}$$

where:
- allowable plastic rotation (connection) is 42 mrad
- yield rotation is 9 mrad

This gives a value of DIF equal to 1.23.



a) b)

Figure 11: Force-deformation relations for modeling the steel elements and components: a) generalized force - deformation relation [18]; b) M3 plastic hinge modelling parameters adopted for beam ends

3.2 Numerical analysis results

Figure 12 shows the results for B5 column loss scenario. The loss of an internal column leads to the failure of the directly affected part, which is the area between axis 4 and 6, see Figure 12a. The force - displacement curve plotted in Figure 12b shows that the structure does not resist the gravity loads calculated using eq. (1) and eq. (2). The plastic mechanism is global (see Figure 12c) but the partial strength connections do not provide adequate continuity to prevent the progressive collapse (Figure 12d). Similar results are obtained for A5 column loss scenario, see Figure 13. The loss of the perimeter column leads to the failure of the direct affected part A4/A6 – B4/B6.

As the building is already constructed (excepting the completion of the setback), there are limited options for interventions. Because the haunches and the beam-to-column connections cannot be further increased, the solution was to provide additional redistribution capacity in the new structure from the last story by introducing eccentrical vertical braces, EBF, in the longitudinal frames. The long links were preferred to stiffer and more ductile short links as they provide wider clear spaces unobstructed by bracing. These braced frames at the top of a structure are also called *hat trusses*. To cover all possible column loss positions, the new braces were inserted in axes A4/A6, B1/B9, and D4/D7, see Figure 14.

The new structure at the top transfer some gravity loads to the adjacent areas and reduces the demands in the beams and connections bellow. As seen from Figure 12b,d and Figure 13b,d, the intervention provides the capacity that is required to arrest the progressive collapse in case of a column loss.

For scenario B5, the maximum top displacement reduces from 288 mm to 156 mm and the plastic rotation in beams reduces from 42 mrad (allowable capacity) to 26 mrad. For scenario A5, the maximum top displacement reduces from 300 mm to 95 mm and the plastic rotation in beams reduces from 42 mrad (allowable capacity) to 12 mrad.

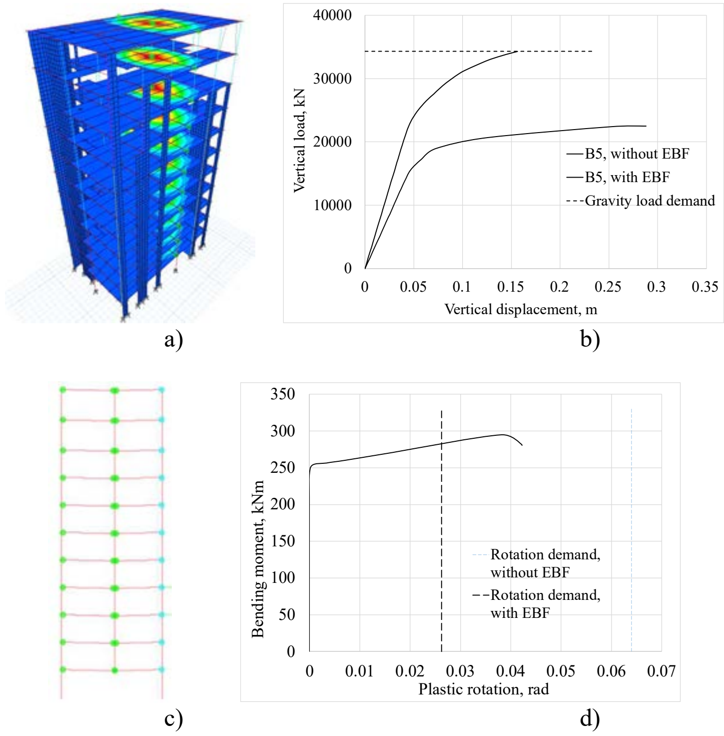


Figure 12: Column loss scenario B5: a) failure mode, retrofitted structure but no EBF; b) vertical load vs vertical displacement; c) plastic mechanism at failure, transversal frame in axis 5, retrofitted structure but no EBF; d) maximum plastic rotation demand at most loaded beam end in axis B5-D5, without and with EBFs

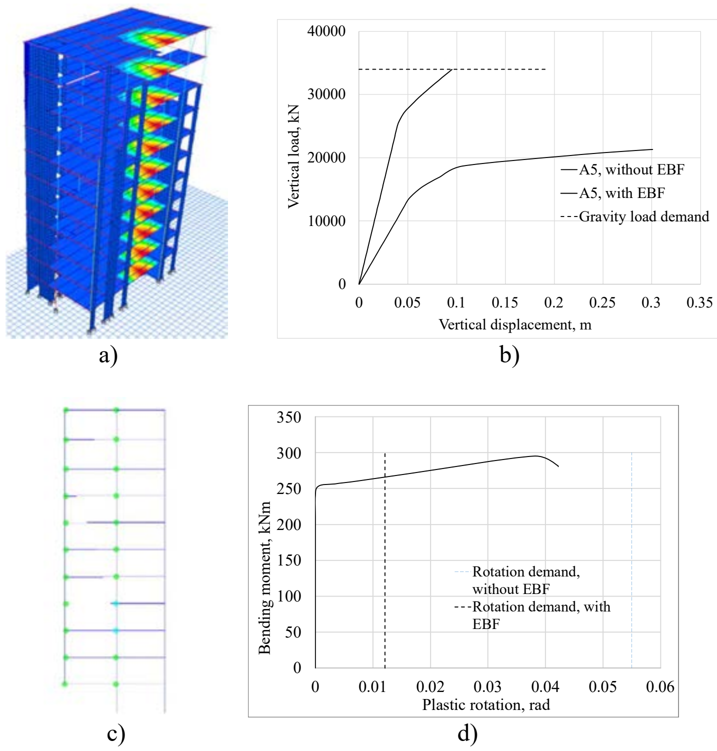


Figure 13: Column loss scenario A5: a) failure mode, retrofitted structure but no EBF; b) vertical load vs vertical displacement; c) plastic mechanism at failure, transversal frame in axis 5, retrofitted structure but no EBF; d) maximum plastic rotation demand at most loaded beam end in axis A5-B5, without and with EBFs

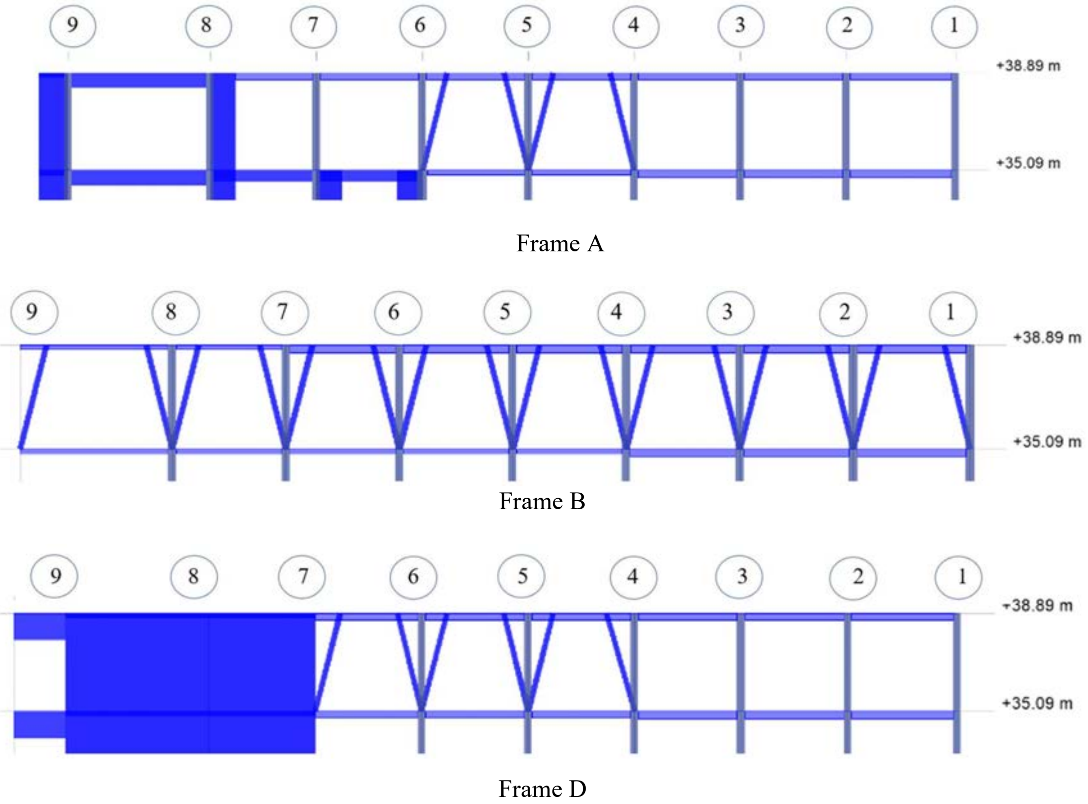


Figure 14: Strengthening the structure for improving progressive collapse resistance by means of eccentrical brace frames on longitudinal frames

4 CONCLUSIONS

An existing 11 story building with a setback on one side, constructed in 2003 but without completing the enclosure walls and roofing, was evaluated. The building is in a moderate seismic area and in the proximity to the sea. The structural system is made of concrete walls, steel frames, and composite slabs supported by profiled steel sheeting.

The first inspections made on site did not reveal serious deficiencies, excepting the corrosion of the steel elements and connections. However, when a comprehensive inspection was performed (destructive testing on steel elements and bolts, samples from concrete slabs), more deficiencies were revealed, some of them capable of inflicting serious safety problems. The results of the structural performance assessment, which also considered the possibility to complete the stories in the setback area, indicated that several elements and connections do not satisfy the strength and deformability requirements resulting from load combinations associated with persistent design situations. The decision of intervention included the replacement of most of the bolts from the connections, strengthening the steel beam-to-column connections and bottom stories steel columns. In addition, the concrete slab floors were proposed for retrofitting.

The robustness of the structure against accidental actions was checked using the AP method and two column loss scenarios. The nonlinear static analyses showed the gravity load designed steel structure can be vulnerable to progressive collapse in case of a first story column loss. This is mainly because the beam-to-column connections provide limited continuity (partial strength, limited ductility). Because the haunches and the beam-to-column connections cannot be further increased, the solution was to provide additional redistribution capacity in the new structure from the last story by introducing eccentrical vertical braces, EBF. The long links were preferred to stiffer and more ductile short links as they provide wider clear spaces unobstructed by bracing.

The introduction of longitudinal braced frames at the top story provides supplementary load transfer routes and increase the capacity to resist the loss of a column. This strategy is used especially for existing buildings, where there is a limited access to the global structural system. It is also efficient for new structures with low or limited level of continuity at beam-to-column connections.

Ongoing studies will also consider more advanced analyses using non-linear dynamic procedure.

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