SEISMIC RETROFIT AND OFFICE-TO-CLEAN ROOM BUILDING CONVERSION: DESIGN AND CHALLENGES

A. Cohen[(1)], A. Shohat[(2)], Y. Offir[(3)]

[(1)] *Structural Engineer, Yaron Offir Engineers, avi@yoe.co.il*
[(2)] *Structural Engineer - Partner, Yaron Offir Engineers, alexs@yoe.co.il*
[(3)] *Structural Engineer - Partner, Yaron Offir Engineers, yarono@yoe.co.il*

Abstract

The conversion of a 40,000 sq-m office building into a clean room manufacturing facility required creative structural retrofit solutions. The owner, a large multi-national high-tech corporation demanded the seismic retrofit of the five-story structure while maintaining uninterrupted operational capacity. This led the design team to develop the seismic retrofit using Performance Based Design principles, following the ASCE/SEI-41 guidelines to meet the stringent Immediate Occupancy acceptance criteria for tailor-made reinforcement detailing.

The seismic evaluation and design of the structure consisted in a detailed analysis of the structural performance of the overall building and the capacity of its supporting members. The retrofit included increasing the ductility and shear capacity of existing concrete shear walls using externally bonded fiber reinforced polymer sheets, a grid of new minipiles for foundation reinforcement and a complete system of collector elements to strengthen the slab-wall connection and ensure adequate diaphragm force transfer, among others. Nonlinear pushover analyses are conducted to confirm the target performance and select the most appropriate retrofit scheme. A leading renowned seismic engineering firm has reviewed and approved this retrofit design.

The strict requirement that part of the building is to continue its daily operations, and the strict deadlines, pushed the design and construction teams to develop creative solutions to difficult and complex engineering problems. Steel, concrete and CFRP reinforcements were carefully detailed and fabricated within schedule. Extensive use of BIM software was used to coordinate and detect clashes amongst other disciplines, which greatly facilitated the execution.

This paper will present the project obstacles, design assumptions, analyses methods, proposed retrofit schemes and their comparisons, as well as solutions to execution problems that were encountered during the design and implementation process.

Keywords: Retrofit; collectors; construction; CFRP; reinforcement.

1. Introduction

This paper describes the design and construction challenges of an office building conversion into a high-tech manufacturing and cleanroom facility, where the seismic retrofit quickly became a key element in the overall project. The building in question is retrofitted to meet both Immediate Occupancy and Collapse prevention performance criteria for a design and maximum considered earthquake respectively, following the ASCE/SEI-41[1] guidelines as requested by the client.

The team of engineers who led the design supervised the construction execution as well, constantly modifying reinforcement details to adjust to unexpected site conditions and interdisciplinary clashes. Redesigns were common, and largely driven by budget constraints and changing construction sequences. A constant effort was also done to introduce innovative materials and retrofit techniques, such as CFRP sheets and rods not familiar to local contractors.

The overall seismic retrofit designed was Peer-Reviewed by San Francisco-based *Rutherford + Chekene* through an 18-month rigorous and challenging process.

In addition to the strict client's budget requirements, important construction milestones were set to establish an completion date to begin manufacturing, with reduced business interruption and without relocation of the office staff. This led the design and construction teams to formulate innovative and cost-effective solutions with minimal intervention during its execution. The main technical and execution challenges are presented herein.

2. Case Study Building

The case study structure is a four story concrete building supported on a concrete pile foundation layout. The first and second floors consist of two-way cast in place reinforced concrete slabs, while the upper floors are comprised of hollow concrete precast planks with a 5 cm topping layer. Vertical support is provided by concrete columns, elevator and stairs piers and by five reinforced concrete walls used as shelters and stairways. A construction joint divides the building into two structurally independent structures. An isometric view of the analytical model of the building is shown in Fig. 1. The building is supported on a concrete pile foundation.

Under the new design, the upper two floors are to remain office spaces, the second level will convert to a cleanroom manufacturing floor, with a 2.5-factor increase in the required design live load; and the first floor is to become a utility level. The new roof will support heavy mechanical equipment required to pump air into the cleanroom. In addition, a nine-meter wide and three-meter deep new utility trench was excavated from the ground floor for process pipes in and out of the facility.

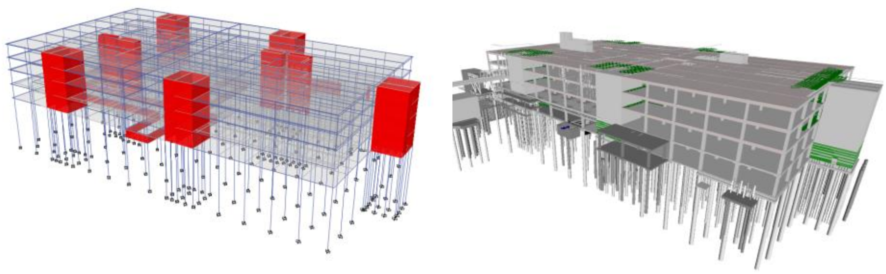

Fig. 1– Isometric view of the analytical (left) and the BIM models (right).

In order to connect the high-tech equipment on the cleanroom with the machinery downstairs the existing 430mm deep reinforced concrete slab was perforated at an established pattern to allow piping and cables to run through. The location of these popouts was carefully set in order to minimize damaging the existing steel and allow for carbon-fiber sheet reinforcements.

3. Seismic Evaluation

3.1 Target Building Performance Levels

The importance of the building as a high-tech manufacturing facility demanded strict seismic performance levels of both structural and nonstructural elements. The structure was evaluated and strengthened to meet the following performance criteria: Immediate Occupancy for the BSE-1N Earthquake Hazard Level and Collapse Prevention for the BSE-2N Earthquake Hazard Level. Additionally in order to limit the damage of architectural, mechanical and electrical systems as well as building equipment, the building was classified to meet Position Retention and Life Safety nonstructural performance levels for BSE-1N and BSE-2N respectively. Hence, the Target Building Performance Levels for which the retrofit design was based are 1-B and 5-C for a Hazard return period of 475 and 2475 years respectively.

3.2 Nonlinear Pushover Analysis

A nonlinear static analysis of the existing building was performed at early stages of the design to better understand the chain-failure modes and seismic performance of the structure. A two dimensional-equivalent half-model was created for the transverse direction, with plastic hinges modelled as per ASCE/SEI-41 guidelines, using collected data from record drawings, construction logs and both destructive and non-destructive site testing. The foundation was explicitly modelled using nonlinear P-y curves obtained from the local geotechnical consultant.

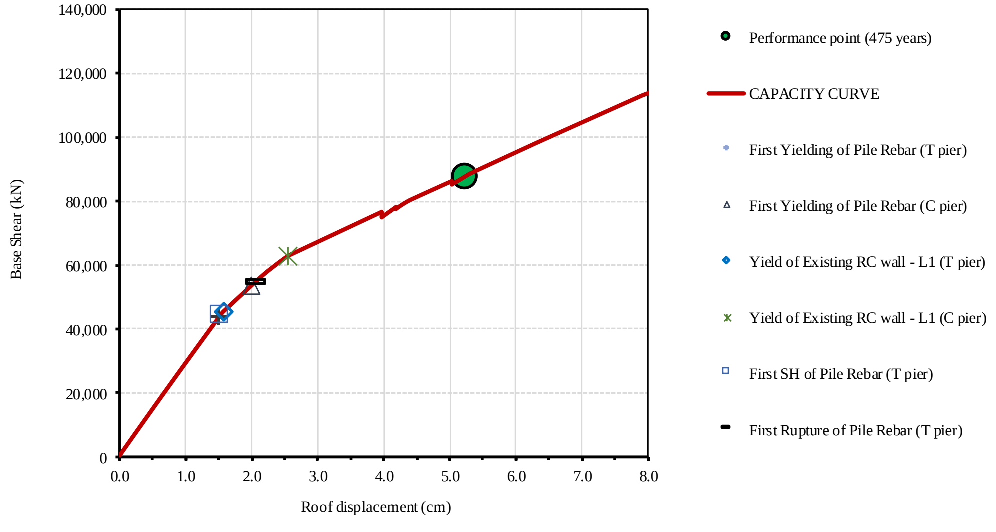

Fig. 2– Pushover capacity curve and failure mechanism for the transverse direction of the existing building. Yielding in reference to wall pier under tension (T pier) and under compression (C pier).

As shown in Fig. 2 pile failure is expected at small displacements, followed by plastic hinge formation on the shear walls, all of which occur before the performance point. Even though there is no significant loss of stiffness or of load carrying capacity, these failure modes are non-compliant with the IO performance criteria. In addition, the capacity curve above highlights the weakest elements in need of retrofitting.

With the objective to minimize the intervention during construction works, both the client and the contractor requested a retrofit scheme consisting of external shear walls. This preliminary assessment also served as an important tool for the designers to reject the concept since the new concrete walls will have to be too large to prevent any foundation or existing wall failures. Moreover, the collector reinforcement required will not have been reduced but rather increased due to the larger wall dimensions and thus increasing the intrusion for the occupants. Lastly, the detailing requirements to meet the IO criteria called for shear retrofit of the existing walls anyway for an increase in ductility, and therefore practically eliminating the apparent efficiency in an external wall system.

3.3 Retrofit Scheme Selection

The selection of the final retrofit scheme was based on the results obtain from the nonlinear analysis as well as construction and time constraints. The design philosophy consisted on enhancing the ductility of the existing concrete shear walls, which were not detailed nor designed to meet current code requirements. Added tensile capacity to the pile foundation was required to ensure the development of plastic hinges at the walls. Collector reinforcement was virtually nonexistent for the upper three floor slabs, including the roof, for which extensive retrofit was needed. Since the existing structural joint at the center of the building was insufficient, it was decided to connect both structures to avoid pounding. This was not only favorable for the seismic retrofit but also beneficial for mechanical and process designers with cost savings of expensive seismic joint pipe connections.

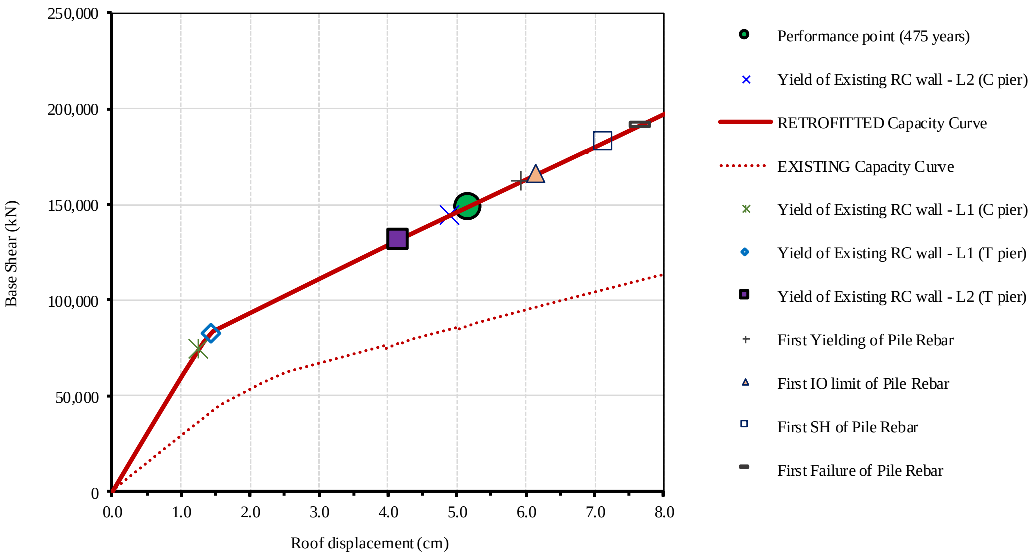

Fig. 3– Pushover capacity curve and failure mechanism for the transverse direction for retrofit state. Yielding in reference to wall pier under tension (T pier) and under compression (C pier).

The retrofit concept was verified by running additional nonlinear analyses. As observed in Fig. 3, the main failure modes have been shifted to develop after the target performance point, by enhancing the ductility of the elements. It can also be noted that there is no significant increase in stiffness of the structure, but rather that of the foundation, and almost no fundamental period shortening. Further technical detail for each reinforcement is detailed in following sections.

4. Foundation Retrofit

Tension yielding of longitudinal steel on the piles is the first failure mode to be expected during a seismic event for the existing structure. An in-depth evaluation shows that while the existing piles have substantial uplift capacity, there is no sufficient steel reinforcement to develop any plastic hinging of the walls or columns it supports.

The foundation of the existing reinforced concrete walls, a grid of meter wide piles at approximately three meter spacing, was explicitly modelled using flexible-base assumption following ASCE/SEI-SEI 41 guidelines, including soil-structure interaction. Sufficient record data was made available to classify the Level of Knowledge as Comprehensive and thus minimize the material on-site testing.

Following capacity design principles, the maximum tensile force that is expected to be transferred from the reinforced concrete piers to the foundation equals to the expected yielding force of all steel bars for the IO criteria and the expected rupture force for CP taking into account strain hardening. The foundation retrofit ensures there is sufficient capacity to develop the expected plastic hinges in the walls, by guarantying enough tensile capacity increased by an over strength factor of 1.25.

Numerous options were considered for the foundation retrofit, including core-drilling longitudinally through the pile and epoxy-grouting additional reinforcement, an alternative which was rejected mainly due to lack of locally skilled contractors and equipment. The selected retrofit scheme is based on a series of minipiles drilled along the walls and through the exiting pile cap. These 200mm-diameter and 20-meter long minipiles are skewed at a 15 degree angle in order develop the design tensile forces and avoid soil group effects with the existing piles. The capacity of a small enough drilling machine to fit in the necessary interior rooms determined the size and length of these piles.

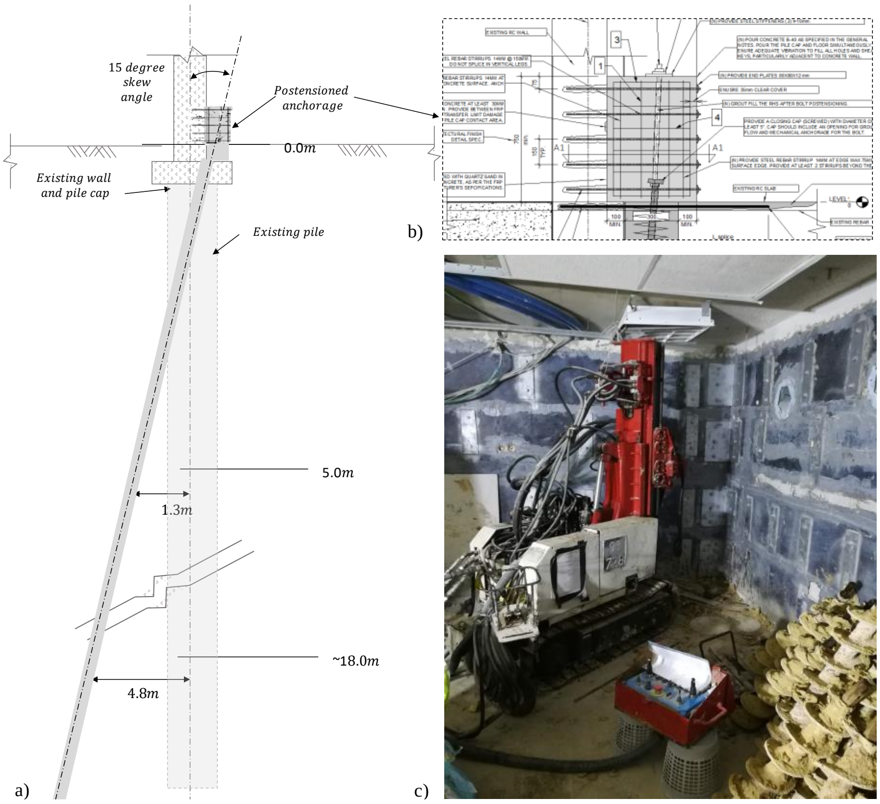

Fig. 4 – a) Initial concept sketch of skewed minipiles for foundation reinforcement, b) typical post-tensioned anchorage detail; and c) pile drilling machine adjacent to an interior wall during construction works.

The anchorage of the mini-piles to the concrete walls is located above level of the existing pile cap in between the existing piles, allowing flexibility during construction. The plastic hinge location is displaced upwards, above the anchorage and away from the vertical rebar splice area. Between three and four minipiles were drilled for each existing pile, as required for load transfer due to axial stiffness equivalency. At its core, each minipiles had a 3-inch steel hollow section inside helical rebar to withstand the tensile forces. The anchorage was designed to resist, through shear strength, the expected tensile capacity of the mini-pile, including the acting eccentricity due to the geometry of the connection; by including a connection over-strength factor. Finally, the newly excavated utility trench served as a *large shear key* which help withstand part of the overall base shear of the structure.

5. Shear Wall Enhancement

The main lateral seismic force resisting system of this building is provided by existing concrete shear walls, which comprise the five shelter-and-stairway cores distributed on the structure's layout. These shear walls were not initially designed to withstand earthquake forces, and thus have inadequate seismic-resistant details, but were designed and detailed for blast-loading, providing a certain degree of ductility.

The wall sections were analyzed as both separate elements with effective flanges and as *box* sections, to account for bidirectional loading separately for tension and compression cycles. Shear, axial and moment interaction was considered using the Simplified Modified Compression Field Theory [2]. Further in-depth analysis revealed the need to retrofit the walls located on the first story only, significantly limiting the intervention and construction schedule.

Both shear and flexural capacities required reinforcement in order to meet the stringent IO criteria, as well as enhanced ductility capacity to achieve CP acceptance criteria. This was accomplished through the use of externally bonded CFRF sheets placed horizontally along the walls following ACI 440 [3] and TR 55 [4] guidelines. By increasing the shear strength of the wall piers well above the expected flexural yielding capacity, the desired energy dissipation mechanism is achieved through a sustained plastic hinge formation at the bottom walls.

Mechanical steel details were developed to prevent sheet delamination along the 10 meter long shear walls, as well as to enhance boundary confinement and anchorage and reentrant corners.

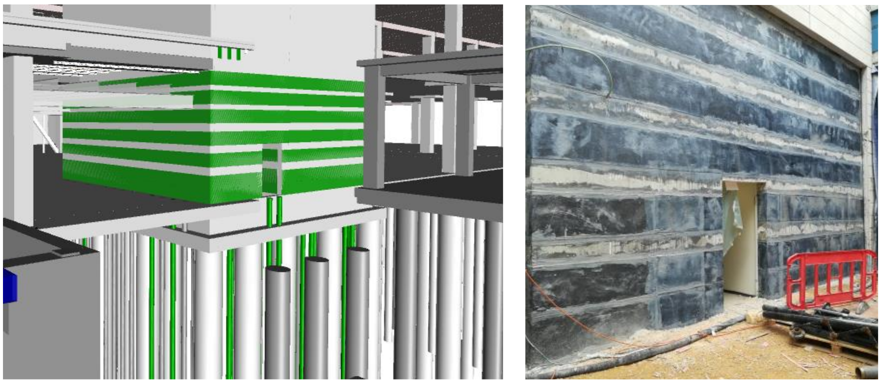

Fig. 5 – BIM model of CFRP sheet shear wall reinforcement (left) and wall during installation (right)

6. Collector Reinforcement

The collector design for this case study building is by far the most iconic retrofit measure, due to its extent and impact in the construction sequence and schedule. The clear absence of chord and collector reinforcement details is evidence that seismic design was not originally accounted for. The slab-to-wall connection of the top three precast slabs consisted solely of the lightly reinforced topping layer.

There was no governing acceptance criteria for the diaphragm analysis and retrofit, calling for separate parallel analyses for Immediate Occupancy and Collapse Prevention; the former requiring unreduced diaphragm seismic forces, while considering post-cracking concrete shear capacity for the latter. The reserve capacity of the existing beams was exhausted after several design iterations in order to maximize the efficiency and optimize the reinforcements and labor force.

The use of bolted steel plates on top of the concrete slab was selected as the preferred collector reinforcement method, transferring forces from the floor to the adjacent walls through post-installed anchor bolts. Where required, these plates were embedded into the concrete topping and fixed with countersunk bolts to avoid any obstructions in the final state.

6.1 Collector design and detailing

The chord and collector reinforcing elements, which at some locations reached 18 meters, were detailed to accommodate the strain incompatibility between the steel plates and the existing concrete. Special attention was given to the deformation capacity of the steel relative to the deformation of the concrete diaphragm. This is highlighted and particularly important in long connections such as these collectors. Since the steel plate will experience large deformations due to the induced load, the concrete will not reach the same strain levels, as a consequence the first row of anchors can be overloaded to failure and result in an undesirable zipper effect.

In order to control this phenomenon special considerations in the detailing have been included: the use of stepped plate sections, to have a relatively uniform elongation along the collector length; interface friction and deformation capacity; epoxy filled holes for increase in rigidity; and deformation capacity of anchors under seismic loading. The seismic shear strength of the anchors was taken from the manufacturer's technical sheets and adjusted with the factors abovementioned. Design and ultimate seismic deformation capacities were also obtained from the manufacturer and adjusted for the grout layer at the plate and concrete interface, reducing its strength but enhancing its ductility by 8% and 20% for a ¼ and ½-inch layer respectively. As shown in Fig. 7. Steel failure of the anchors is ensured by specifying spacing and diameter of the bolts. The ductility of the system is affected by a grout layer at the plate and concrete interface. Moreover, all steel plate faces in contract with the concrete were required to be sandblasted to increase friction and first-slip capacities.

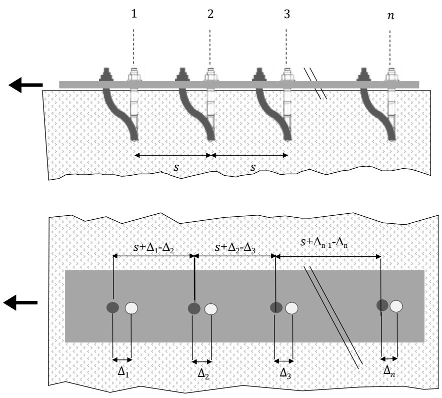

Fig. 6 – Schematic deformation distribution of multiple anchor connection of steel-to-concrete [5]

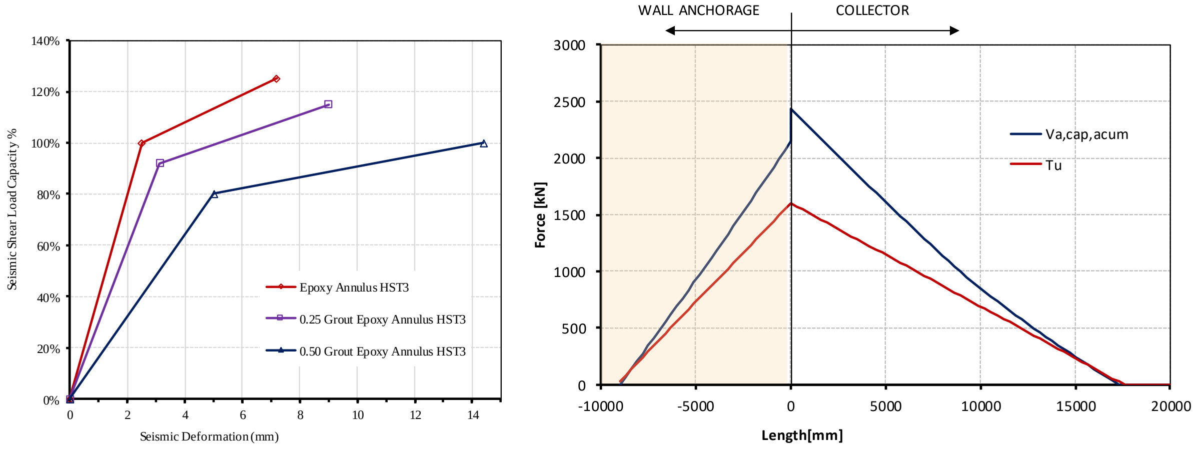

Fig. 7 – An example of the modified load-deformation curves used for a M12 anchor for long collector design (left); collector load and capacity curves (right).

Following a displacement based principle criteria, the steel plates and bolting configuration was detailed to minimize the concrete and steel plate strain incompatibility. The displacement of each anchor bolt is calculated considering the deformation of the steel plate preceding the anchors. The deformation of the subsequent bolt row results in the elongation of the steel plate times the length between rows deducted from the deformation if the previous bolt row, as schematically illustrated in Fig. 6 [5]. The strength of each row of bolts is then calculated from the load-displacement curves.

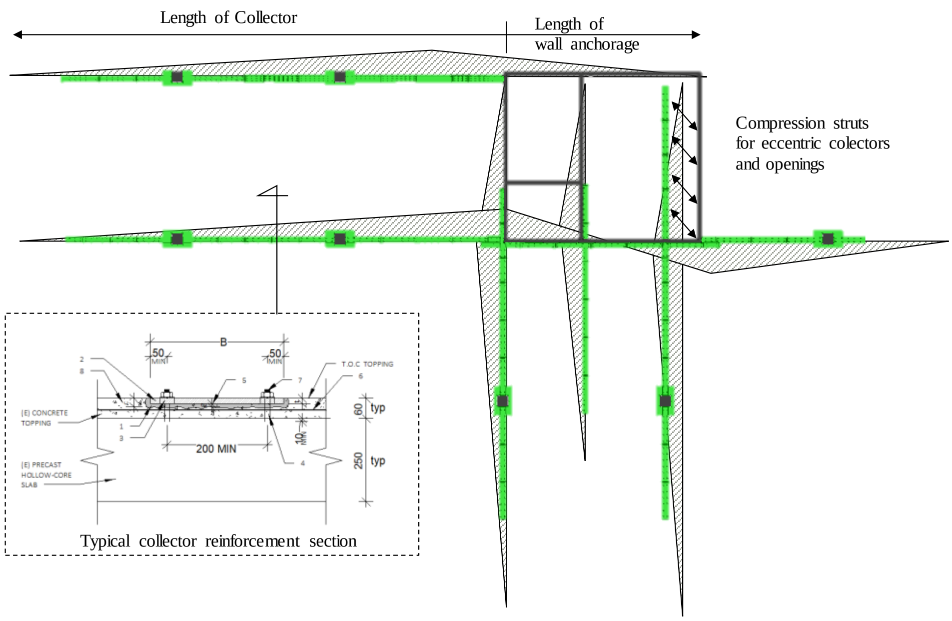

Fig. 8 – Collector axial force diagrams and typical reinforcing detail, example.

A representative finite element model was constructed and analyzed using software package SAP2000 to verify the design calculations and shear distribution among anchor rows in along the collectors. This resulted particularly useful for complex cases where bypassing existing columns and for cases with eccentric load transfers.

Fig. 9 – Collector reinforcements: During installation works (left) and BIM model view (right).

6.2 Diaphragm Shear Transfer

Analysis results indicated high shear stress concentration on the diaphragm around the concrete walls, in the slab-to-wall connections, as expected. The in-plane shear around these areas is critical to ensure adequate force transfer to the collector elements and thus required strengthening, which was provided by installing epoxy bonded CFPR strips installed at 45-degree angles. A composite topping and precast section was used for shear capacity since sufficient friction shear strength was shown to exist at the interface.

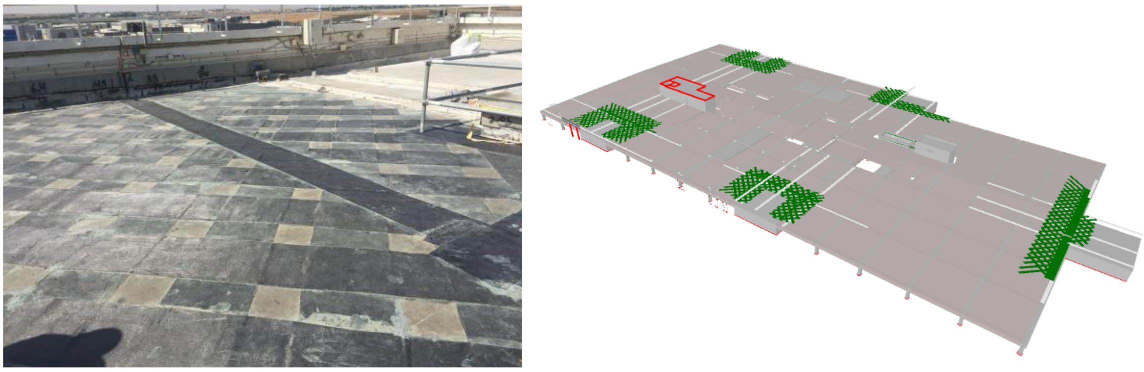

Fig. 10 – Diaphragm shear strengthening on roof: during installation works (left) and BIM model view

7. Construction Challenges

The final retrofit implemented was a result of a series of redesigns to adapt to changing circumstances, where strict budget and schedule constraints, as common for most projects, provided both obstacles and opportunities. The design and construction teams worked closely to provide innovative solutions to a dynamic construction site, making each iteration more efficient and clash-free. Since the return to business was the client's number one priority, seismic retrofit works took second stage to process and mechanical priorities required to operate the new cleanrooms, which constantly clashed with the reinforcements. In addition unexpected site conditions also forced the design team to modify the design and detailing as construction works advanced.

Night shifts were common throughout the project to reduce the impact of the daily operation of the building. The gradual installation of vibration sensitive equipment dictated construction methods and equipment to be used on site. Special machinery such as minipiles drillers small enough to fit through an office door were employed as well as creative FRP detailing to adapt to inaccessible areas. Tight interdisciplinary collaboration as well as transparency with the client were key to the successful installation of the seismic reinforcements.

8. Conclusions

The retrofit of the case-study building was essential in order to certify the case-study office building into a high tech manufacturing cleanroom facility, to one of the highest seismic performance standards. The main deficiencies were detected at early stages through a series of nonlinear pushover analyses, where the seismic performance of the structure was observed and failure mechanisms were identified. The foundation was evaluated and retrofitted by drilling a series of skewed mini-piles in between existing caissons. The shear and flexural capacity, as well as its ductility, of the existing structural walls were enhanced through externally bonded CFRP sheets. Finally, chords and collectors were strengthened to ensure adequate force transfer and allow plastic hinging and energy dissipation at the concrete wall bases. Collector reinforcements were developed following displacement-based design principles to ensure all anchor bolts are engaged.

The success of this retrofit project is a result of strong interdisciplinary collaboration between the design and construction teams. Unexpected site conditions, strict milestones and changing construction priorities required creative and innovative solutions resulting in out-of-the-box type details. This case also shows that the design process is dynamic and adaptive as construction works advance. Shop drawing detailing was developed by the contractor in conjunction with the Engineer of Record and the construction manager in order to minimize on-site clashes. The implementation of all retrofit works is designed and executed to minimize the disturbance of the occupants without any interruption to the daily operation of the building.

9. References

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