THE FLOATING GYM: ENGINEERING A NEW BUILDING ABOVE AN EXISTING STRUCTURE THAT CANNOT BE ALTERED

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Abstract

Completed in 2023, the new Malloy Pavilion at the University of San Francisco (USF), shown in Figure 1, is a state-of-the-art, 16,800-square-foot NCAA practice gymnasium with basketball and volleyball courts, as well as office space. Due to limited space on campus, the site chosen for the new gymnasium posed a major challenge—it is also home to a below-grade concrete parking garage built in 1965 that is attached to a mid-rise Hayes-Healy Residence Hall. The site is also bordered closely on the other side by the War Memorial Gym at Sobrato Center.

The new gymnasium would need to be seismically isolated from the parking structure below to prevent triggering a mandatory seismic upgrade to the parking garage and adjacent residence hall. If upgrades were triggered, the project would be infeasible due to cost and downtime for installation.

The solution? The gymnasium would be "floating" above the below-grade parking garage and positioned between the nine-story Hayes-Healy Residence Hall and the War Memorial Gym at Sobrato Center.

To ensure that the parking garage was not affected, only one steel frame and one standalone steel column penetrate through the garage's three levels below. Oversized penetrations in the parking garage floors were created to allow seismic movement of the new structure above. The new gymnasium extends beyond the footprint of the parking garage, allowing robust perimeter steel frames to occur outside the garage structure.

The project required a holistic understanding of the University's goals on a complex campus that has been developed over decades. Project success therefore required creative solutions, iterative design processes, and extensive analyses of lateral systems, gravity framing, vibrations, and foundations to work around the unique project constraints. The project exemplifies full integration of structure and architecture to achieve 100-foot-long spans, large cantilevers, and acceptable vibration performance. An innovative structural design was critical to transform this extremely challenging site into the new gymnasium that this University needed.

Figure 1: Malloy Pavilion, the "Floating Gym," Looking South

Introduction

Located in the heart of the city, the University of San Francisco serves 9,000 students on a hilltop campus spanning 55-acres. Years of successful sport programs led to scheduling challenges for the original War Memorial Gym at Sobrato Center, and the University gained interest and funding to build an additional gym to support practice functions. Space on the dense, hilly campus was limited, and though multiple sites for a new gymnasium were explored, the best fit was the empty space adjacent to the existing gym. However, there was a major challenge with this location—it was above an existing three-story below-grade parking garage that is attached to the nine-story Hayes-Healy Residence Hall. Any major changes to the parking garage would trigger a seismic retrofit not only to the parking structure itself, but also the attached residence hall. Retrofitting these structures would not be feasible due to cost and downtime. Because the project would not have occurred otherwise, the entire project revolved around this reality.

Due to the challenging conditions, the University chose to solicit the design as a competition between four local reputable structural engineering firms, from which ZFA was selected. C+A Architects and Cahill Contractors formed the remainder of the primary team. Outside structural review during the permitting process was required by the Authority Having Jurisdiction, the San Francisco Department of Building Inspection (SFDBI), and was provided by Nabih Youssef Associates (NYA).

ZFA's innovative solution involved utilizing the space adjacent to the existing gym and "floating" the new gym over the existing parking garage. The footprint of the new gym above the parking garage included only three structural supports that penetrated the existing garage; the remaining columns that supported the new building occurred outside the footprint of the parking garage. The three support locations were chosen intentionally to minimize disruption to parking spaces and still meet the University's goals.

The unusual support conditions generated a variety of structural design challenges, each that needed to be addressed, some iteratively. The gravity system involved long spans and large cantilevers, requiring full-story height custom trusses. To avoid having these trusses act as part of the lateral system, fuses needed to be built into the truss framing to allow lateral forces to be resisted only by the lateral frames. Also, largely due to the long spans and irregular framing patterns, the vibration analysis required consideration in three dimensions, as effects from one area of the building could propagate to other areas that would typically not be linked. The installation of new foundations was a challenge both within the parking structure, due to low overhead conditions and challenges maneuvering materials and equipment into place, and outside the parking structure, due to sequencing requirements and adjacent retaining walls that could not be surcharged.

Overall, the structure was an exciting engineering challenge that achieved extraordinary results by first conceptualizing a structural methodology to make this project feasible by "floating" the new building above the existing, and then by executing the construction seamlessly with foresight, care, and consistent collaboration. This delivered a beautiful building meeting the needs of the client while keeping the project on schedule and under budget.

University Goals

The University presented three critical goals at the start of the project. The first goal was that the new gymnasium needed to have direct access to the adjacent War Memorial Gym at Sobrato Center. To provide this access, the west side of the new building abuts and provides passage to the existing gym, with a seismic gap detailed between the buildings. The link between buildings offered a secondary benefit of providing a location to house and support the new gym's mechanical, electrical, and plumbing systems.

USF also needed to maintain the existing California Fire Code-required fire lane located between the older gym and the below-grade parking garage. By elevating the finished floor level of the new gym floor approximately 15 feet above the grade of the existing surface parking, code-required overhead clearances could be satisfied for continued use of the fire lane, as shown in Figure 2. Structure depths in this area were closely coordinated to ensure the minimum clearances were met.

The third goal was to maintain as many parking spaces—both at-grade and below-grade—as possible. Minimizing penetrations through the parking structure to avoid retrofit also served to keep the parking areas clear of obstructions to maintain as much parking as possible.

Figure 2: North Building Elevation, Fire Lane Highlighted

"Floating" Above the Existing Structure Below

The driving design challenge of this project was placing a new structure over an existing structure that could not be significantly altered. It was known that the existing structure—the below-grade parking garage attached to the nine-story Hayes-Healy Residence Hall, built in 1965—would not meet current building code requirements and that a retrofit to the building would be expensive and time consuming. Triggering a mandatory seismic retrofit to the structure was a non-starter for the project. While the building code does permit some alteration to an existing building without triggering retrofit, significant changes in rigid diaphragm buildings are more challenging to accommodate and remain under these thresholds. While it could be argued that surcharging a below-grade-only structure would not trigger a retrofit, the grade at this location is sloped, and the basement below the Hayes-Healy Residence Hall is only partially below-grade, daylighting along the east side of the residence hall. This condition made the code interpretation less straightforward for this application. As it was critical to the project not to trigger a retrofit, the team chose to minimize impact on the existing structure to help streamline the approvals process.

It was clear that this solution would not allow for conventional framing. As the building would need some support over the footprint of the existing parking garage, the first step was to determine where those supports could occur. Holes could only (easily) be punched in the pan portion of the waffle slabs. Each level of the parking garage had a different floor plan, and one of the University's main goals was to maintain as much parking as possible. After aligning the floor plans for the three levels of parking, there were only three locations that were feasible to place columns that could penetrate from grade level all the way to the foundation. The rest of the design of the structure stemmed from this decision.

During the design phase, a survey of the existing parking garage was not performed; only existing plans were used to locate the existing garage on the site with respect to the new building. Once construction began, the general contractor commissioned a survey and discovered that the existing adjacent parking garage was approximately 11 inches further east and 16 inches further north than shown on the site plan. As the new building was already planned to be located on the site with respect to the above-grade site elements and structures, it could not be moved. Therefore, the spans, framing, lateral system, foundations, and vibrations all needed to be re-checked and quickly adjusted to accommodate the updated column locations so as not to delay construction. Reallocating resources internally to address this redesign effort quickly, ZFA was able to update the design to coordinate with the existing conditions without any delay to major construction milestones.

Gravity System

The building's main gravity system consists of steel beams, girders, and trusses, that are supported by steel columns that continue down to new foundations below. With only three columns able to be located within the footprint of the parking garage below, the majority of columns were located outside the perimeter of the parking garage, as shown in Figure 3. The three columns that penetrate the existing concrete waffle slabs at the two suspended parking levels were detailed with a gap around the steel columns, based on the calculated building drift, to ensure the steel columns remained clear of the concrete structure, as shown in Figure 4.

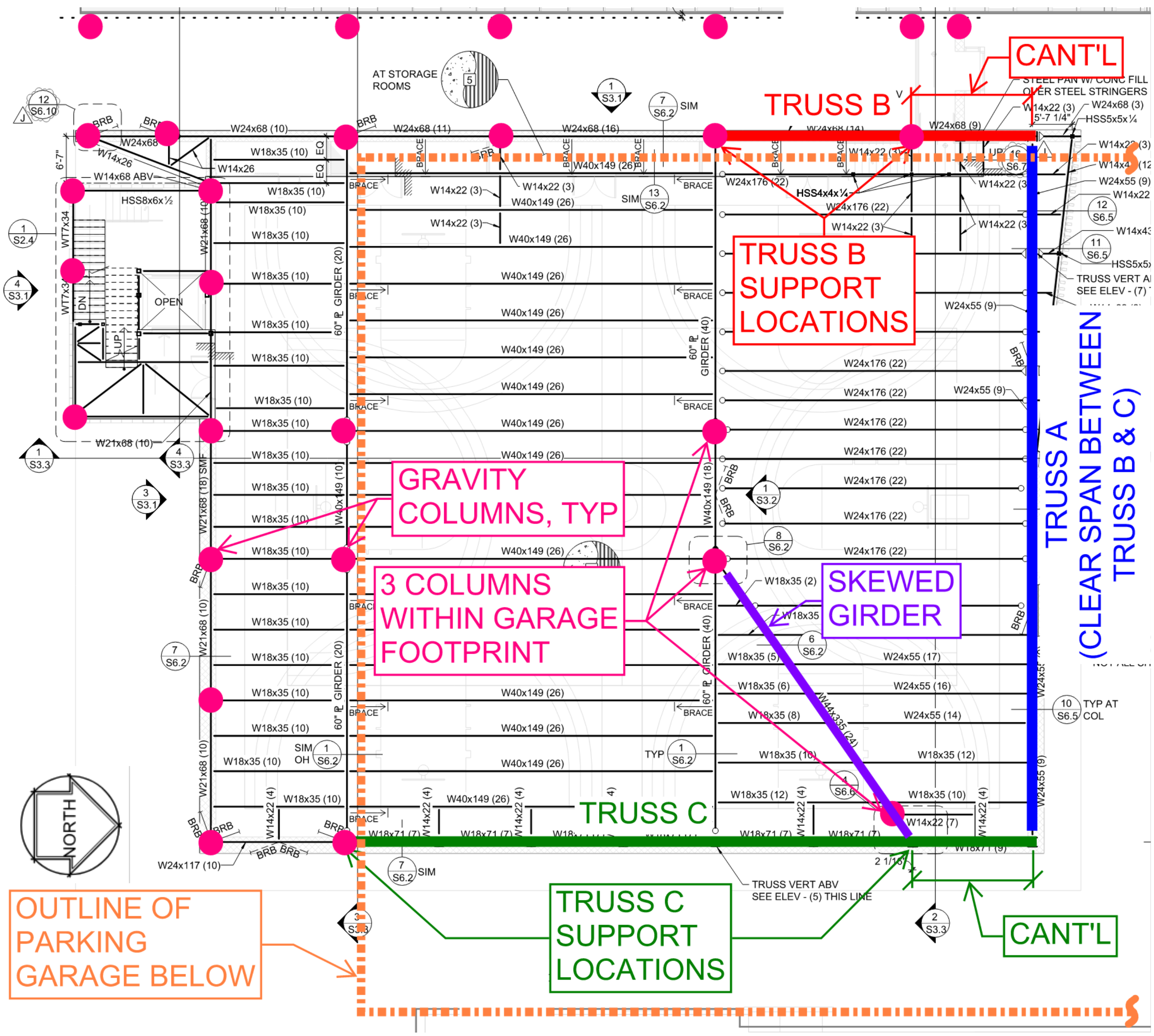

Figure 3: Floor Plan Showing Major Gravity Elements

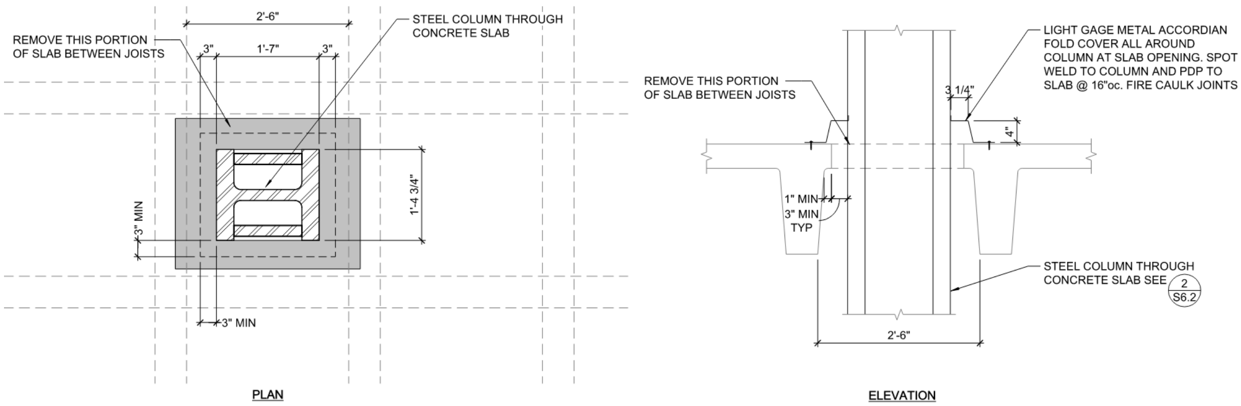

Figure 4: Steel Column Detail Through Waffle Slab

Once the location of the columns was determined, the floor and roof framing systems were designed. The minimal columns over the parking garage resulted in large spans and cantilevers for the floor framing. To achieve the large spans, deep trusses were designed within the exterior walls of the gym, as shown in plan in Figure 3. Truss A runs the entire length of the northern wall and spans approximately 100 feet between the cantilevered ends of Trusses B and C. Truss B is supported by two columns outside of the footprint of the garage and cantilevers 17 feet over the drive aisle of the existing grade-level parking area. Truss C is supported at two locations: the backspan frames into a column outside the footprint of the garage, and the north end is supported on the end of a skewed girder below, which cantilevers off one of the columns penetrating through the structure of the garage. The floor framing then spans between the perimeter trusses and columns.

Plate girders were utilized in multiple locations to provide the required strength and stiffness, while ensuring the overhead clear height was maintained above the parking areas below. At the roof, custom open web joists span 100 feet over the court, while allowing ductwork to run through the webs and maintain the required clear height over the court below, as shown in Figure 5.

Figure 5: Custom Open Web Roof Joists Over the Main Gym Floor; Truss A at the Far Wall, Truss C to the Right

Lateral System

The building's lateral system consists of Buckling-Restrained Braces (BRB) and one Special Steel Moment-Resisting Frame. However, there were several obstacles that had to be overcome to develop a logical, code compliant design. These challenges included a large offset in the lateral system, a frame that had to remain elastic for two stories and yield in the third, and decoupling the complex gravity system from the seismic elements. This decoupling was essential to ensure that the lateral elements could undergo inelastic deformations while not overstressing the gravity frames during a large earthquake. The roof level was laterally braced with BRB frames on each of the four perimeter elevations.

The north elevation, Truss A, is shown in Figure 6. Truss A is a long-span steel truss (1) that spans approximately 100 feet across the full width of the building and is supported at each end by a cantilevered outrigger truss (2). The vertical web members (3) are wide flange sections that extend below bottom chord of the truss to support the floor girder (4), which hangs from Truss A. The depth of Truss A left little space for the yielding BRB elements (5) between the roof and floor. Vertical web members were oriented with the weak axis in the plane of the BRB frames to minimize (but not fully eliminate) lateral resistance in the plane of the truss. The BRBs were designed for the code-prescribed forces from analysis, and then the building was analyzed using a nonlinear pushover analysis to verify that the gravity system would remain elastic until the maximum probable forces in the BRBs had developed. This resulted in notable increases in member sizes and connection forces for Truss A, which is pictured during construction in Figure 7. The north end of the floor diaphragm is cantilevered, and the diaphragm transfers seismic loads to a frame below the floor to carry loads into the foundation, as shown in Figure 8.

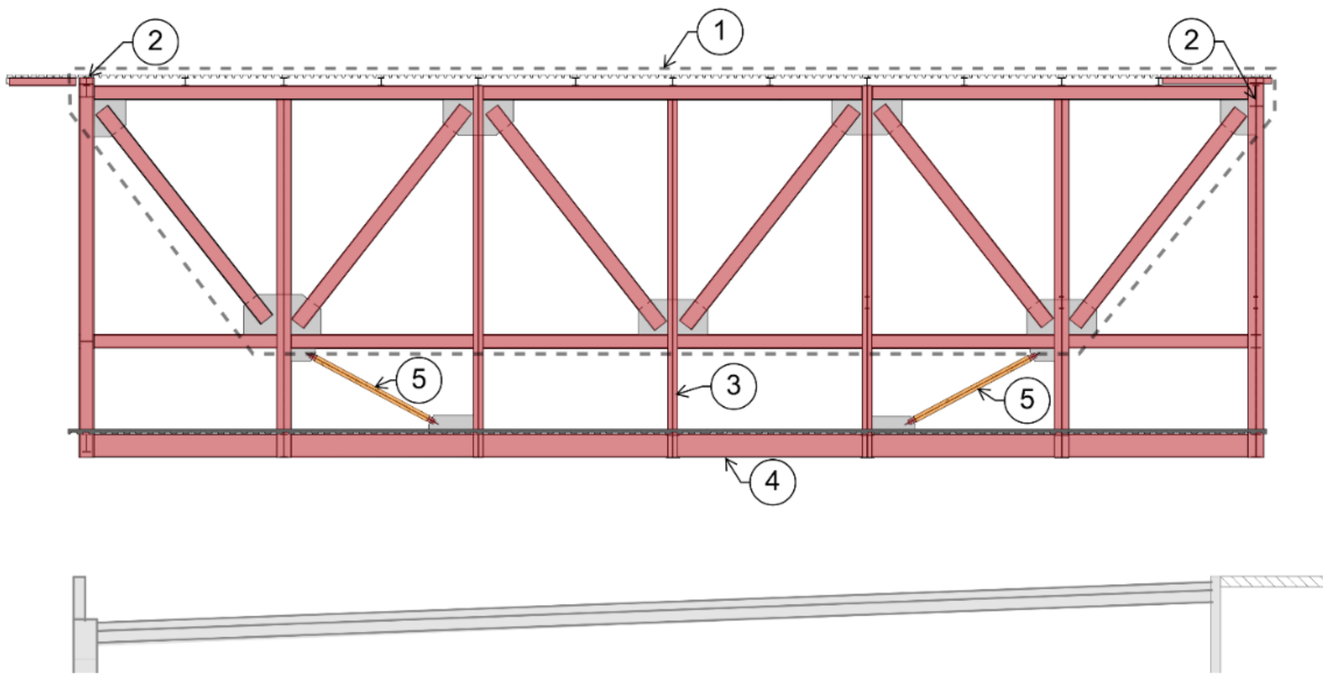

Figure 6: North Elevation Showing Truss A

Figure 7: North Elevation Showing Truss A During Construction

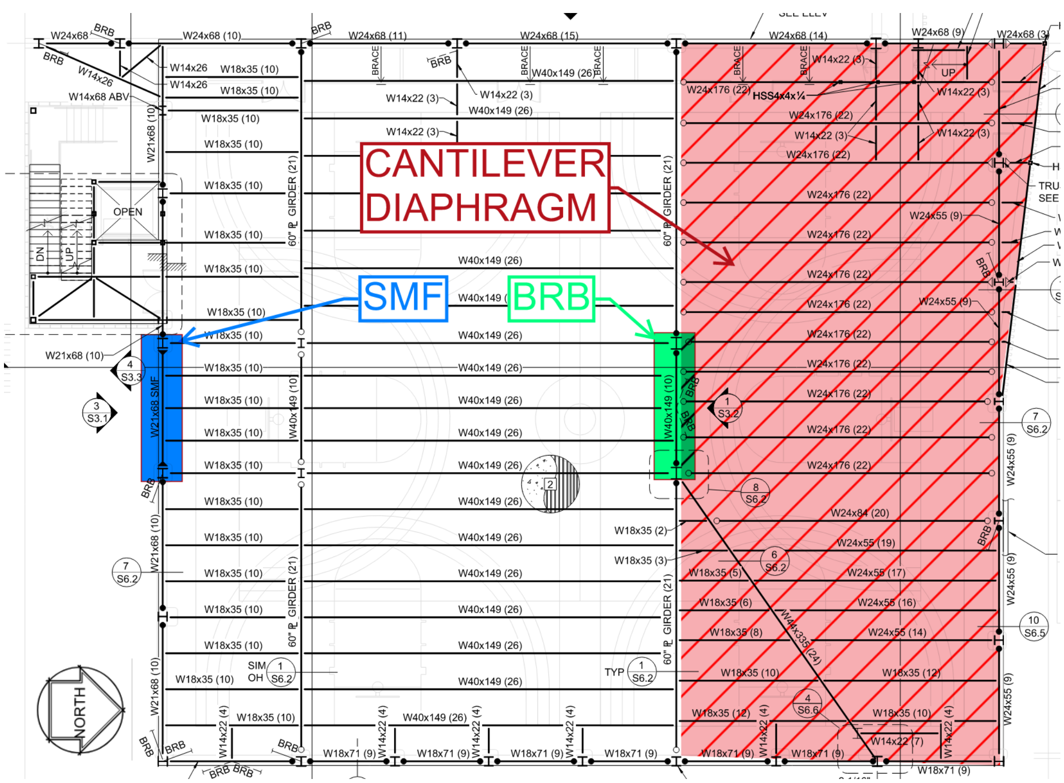

Figure 8: Floor Plan Showing LFRS Below and Cantilever Diaphragm

The east elevation, shown in Figure 9, consists of a two-story inverted "V" BRB frame (1) at the south end of the building to resist lateral loads. The north portion consists of an outrigger truss, Truss C, (2) that is supported by the BRB frame on the south end and a cantilevered skewed steel girder (3) on the north end. Truss C cantilevers beyond the skewed girder to support Truss A along the north elevation. A 60-inch-deep steel plate girder (4), which supports a large portion of the gym floor, hangs from Truss C near the mid-point of the backspan. Like Truss A, the vertical web members of Truss C extend below bottom chord of the truss to support the edge of the gym floor below and provide out-of-plane support for wind loading.

The vertical web members of Truss C were designed to accommodate the story displacements within the height that extends below the truss. This height ended up being relatively short due to the depth of the truss. During the design of the lateral system, using linear modal response spectrum analysis (MRSA), it was determined that despite being oriented to minimize lateral resistance, these truss verticals were resisting a significant portion of the lateral loads, which limited the effectiveness of the BRBs to deform inelastically and dissipate energy. To mitigate this issue, the verticals were detailed with a hinge connection in the weak axis (in-plane with the truss) that would allow for weak axis rotation, as shown in Figure 10. This still allowed the vertical members to provide moment capacity in the strong axis, which was required for out-of-plane wind loading and stability bracing of the truss bottom chord, which was in compression under gravity loads.

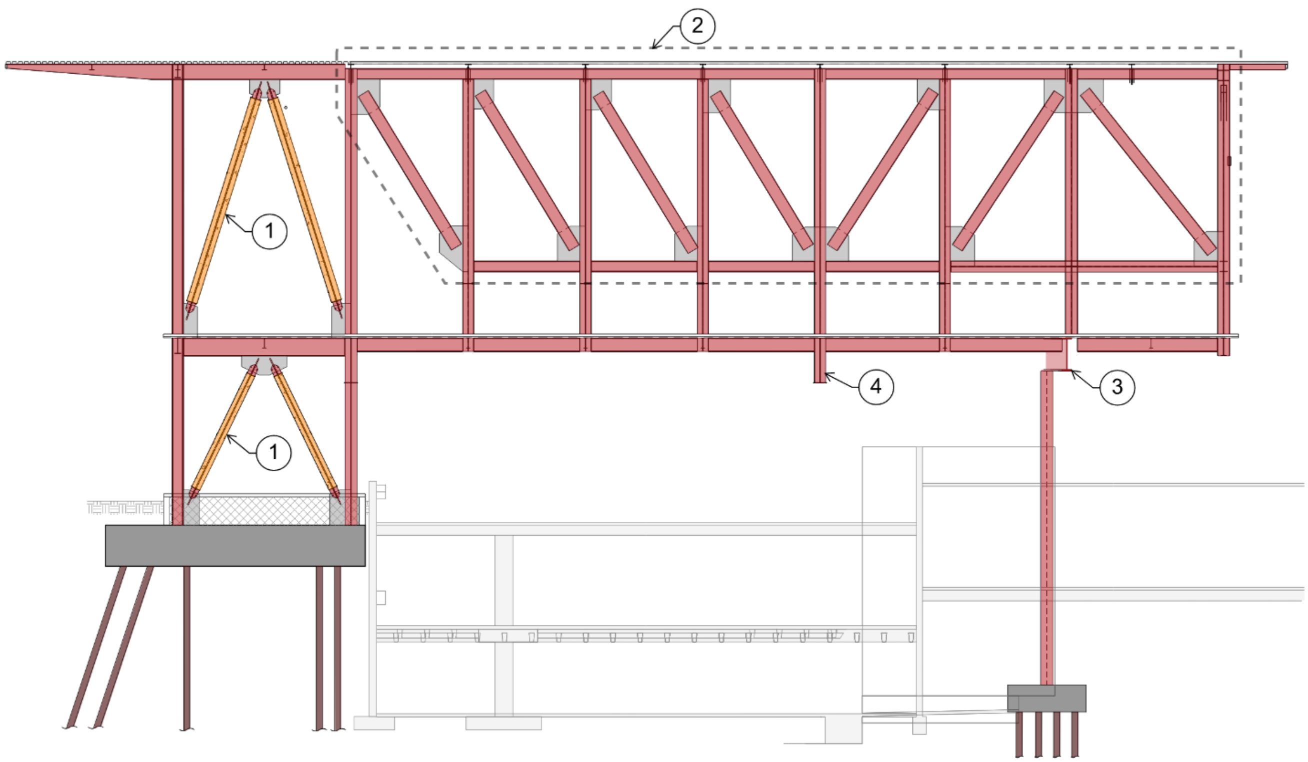

Figure 9: East Elevation Showing Truss C and BRB Frames

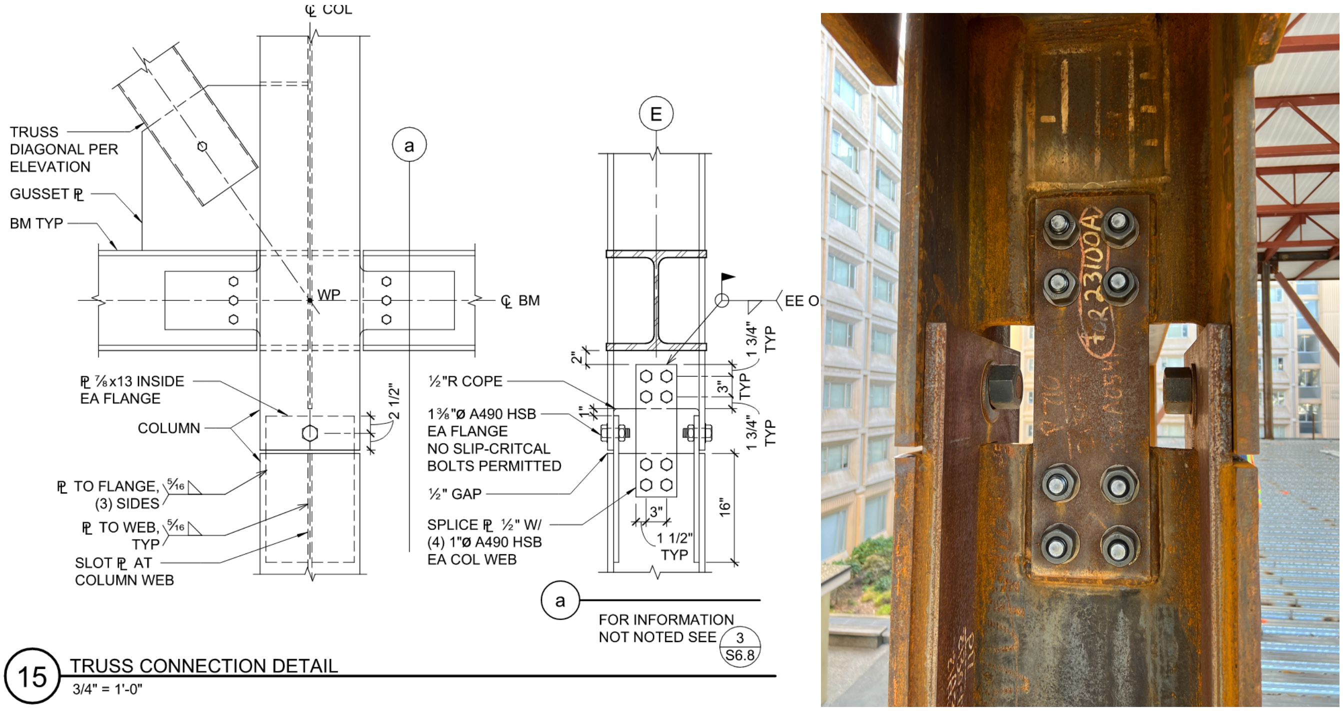

Figure 10: Hinge Connection at Truss C

The west elevation, shown in Figure 11, consists of BRB frames (1) for lateral support. An outrigger truss, Truss B, (2) provides vertical support Truss A along the north elevation. Conventional gravity beams and columns (3) provide vertical support of the roof, mezzanine floor, and gym floor framing. A portion of the gym floor at the north end (4) hangs from the outrigger truss vertical web members. The mezzanine floor and roof are laterally supported in the north-south direction by the BRB frames and rely on gravity columns bending in the strong axis (out-of-plane orientation) for lateral support in the east-west direction. These columns were designed using amplified seismic loads to ensure they remain elastic.

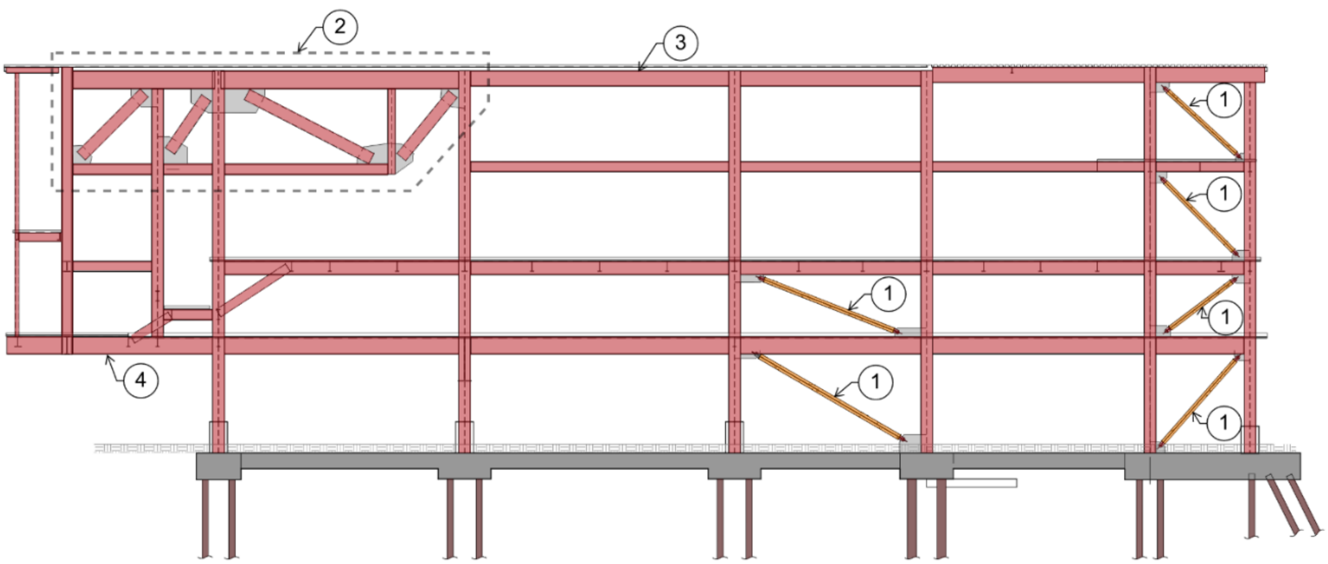

Figure 11: West Elevation Showing Truss B and BRB Frames

The south elevation, shown in Figure 12, includes BRB frames (1) at the gymnasium (upper) level and a Steel Special Moment-Resisting Frame (2) below the gym floor (lower level). The reason for the change in system types was driven by stiffness and a desire to minimize inherent torsion.

At the upper level, BRB frames were used along all four elevations. All elevations were relatively equidistant from the center of mass of the roof, so the use of consistent systems was logical as they would all have the same deformation characteristics.

At the lower level, the lateral element at the north of the building occurs closer to the middle of the building (horizontal offset), shifting it closer to the center of mass of the gym floor, as shown in Figure 13. The lateral element at the south elevation was located at the perimeter of the building. To keep the center of rigidity and center of mass as close to each other as possible, the stiffness of the frame at the south elevation was reduced. This resulted in the upper-level stiffness at the south elevation being significantly higher than the level below. However, due to the high stiffness of the center (north) frame at the lower level, the overall story stiffness at the lower story remained greater than the stiffness of the story above such that no Soft Story Irregularity was created. This reduction in stiffness at the south elevation was critical for minimizing inherent torsion at that level, though it did increase the sensitivity to accidental torsion. This was a necessary consequence of the building configuration.

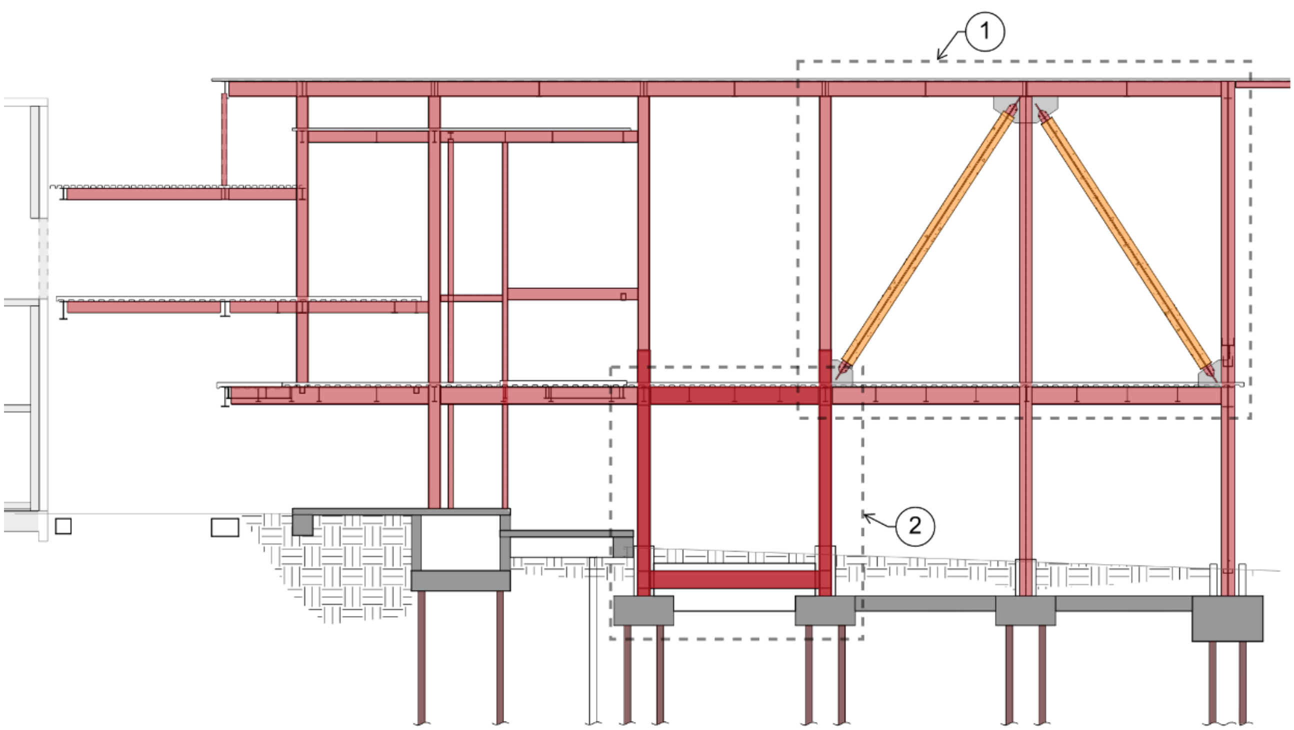

Figure 12: South Elevation Showing BRBs and Moment Frame

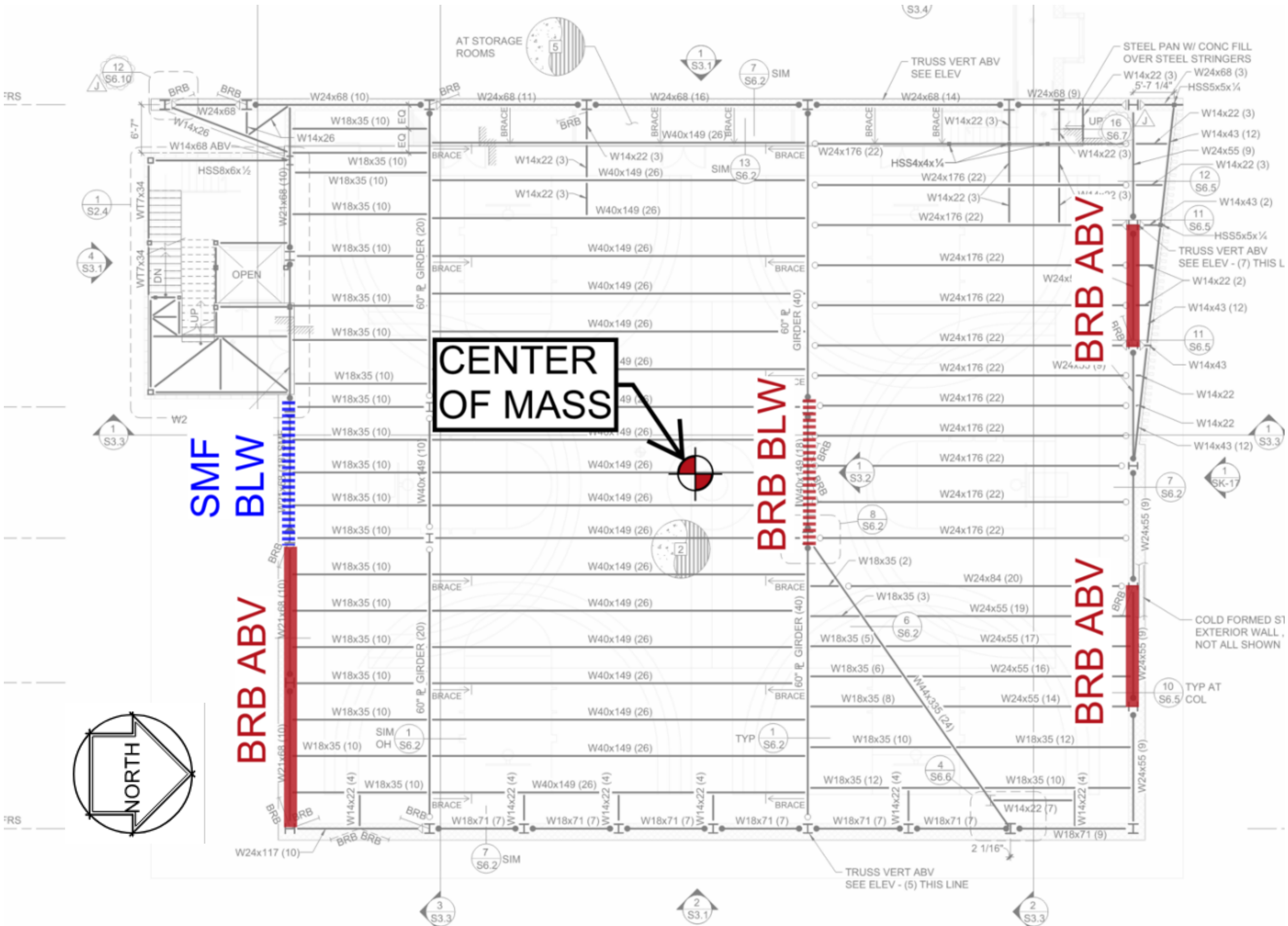

Figure 13: Floor Plan Showing LFRS in the E-W Direction

The horizontal offset of the lateral frame in the north elevation at the lower level presented an additional challenge. The location of the north BRB frame on the lower level was located over the existing parking garage, which made it 40 feet tall compared to the 18-foot-tall special moment frame on the south elevation. And, the north frame had to penetrate two levels of parking to clear the garage and terminate in new foundations. To minimize disturbance to the garage, the columns of the frame were located between pan joists of the garage floor, which left limited clearance for seismic displacements. Therefore, the north BRB frame was designed and detailed to remain elastic for the lower portion in the garage and all inelastic deformation was concentrated between top of the garage and bottom of the gym floor, as shown in Figure 14.

The elastic frame consists of a braced frame from the foundation to the underside of the lower garage level, shown in Figure 15, a small gap where columns penetrate the garage floor and flexural deformations define the response, then another rigid frame between the lower and upper garage levels. Above the garage, another horizontal beam forms the base for the gussets of the inelastic BRB frame.

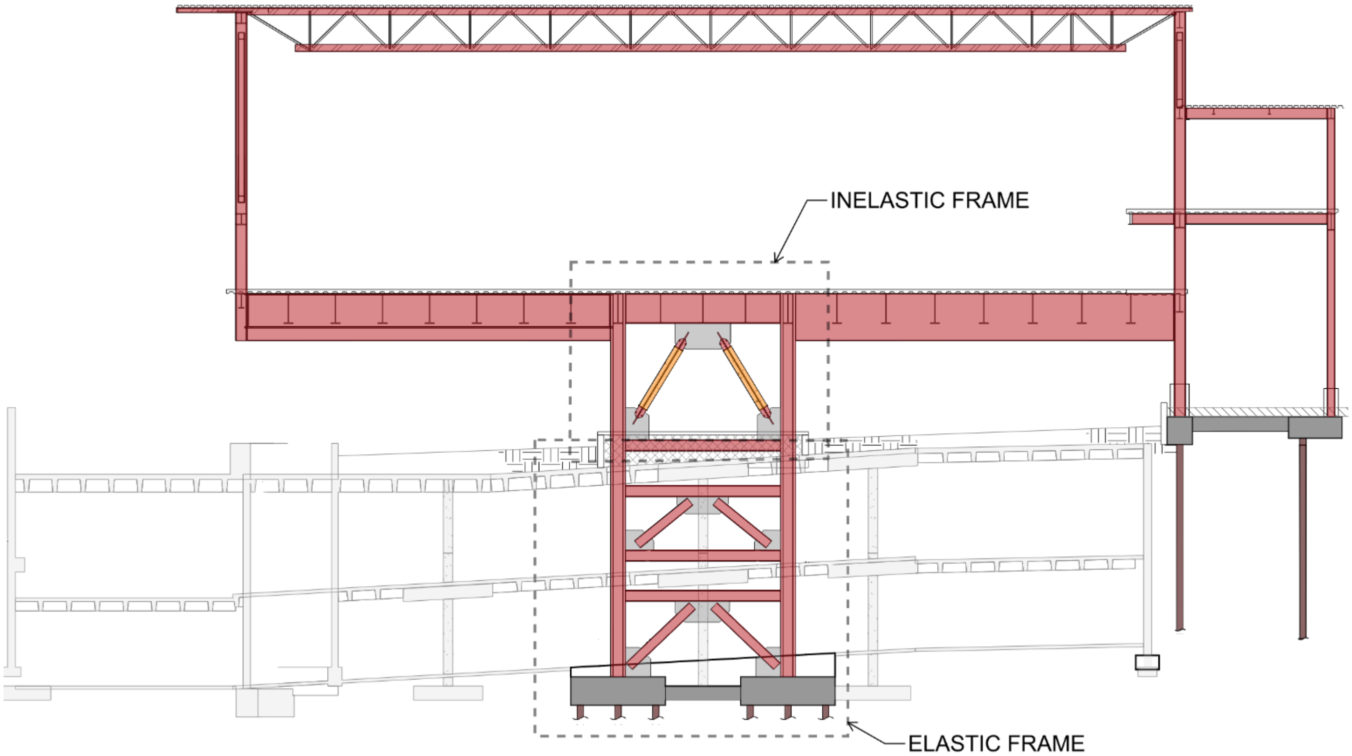

Figure 14: Elevation of Central ("North") Frame that Penetrates Two Levels of Existing Parking Garage Below

Figure 15: Braced Frame Installed at the Lowest Level of the Parking Garage

The design of this frame was based on the nonlinear analysis demands to determine the minimum area of the BRB core, then the elastic frame was designed for the capacity-limited seismic demand, based on the expected strength of the BRBs. Finally, the inelastic deformations of the frame under seismic loading were checked against the available space that could be created around the column in the garage floor slab to ensure they did not exceed the 3-inch gap around the edges of the column.

To meet the required deformation limits, the columns specified were box sections built up from W14x455 wide flanges with 1½-inch plates on each side to form a box section, as shown in Figure 16. This provided the necessary rigidity to minimize flexural deformations of the columns as they pass through the floors.

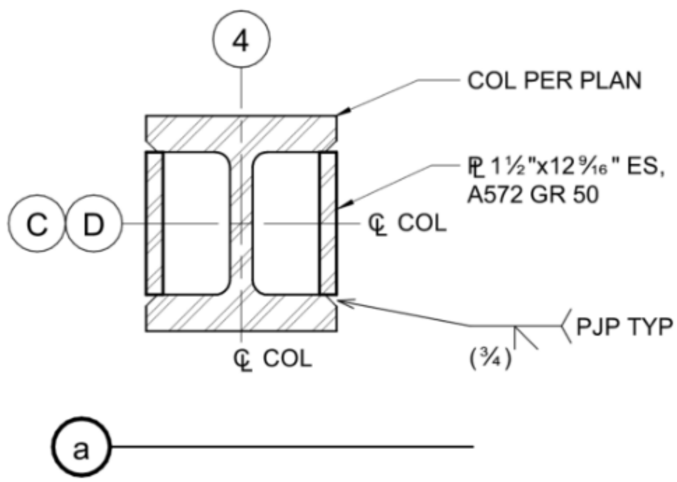

Figure 16: Reinforced W14x455 Columns Forming the Inelastic Braced Frames

Foundation Systems

The site of the new gymnasium structure is in an area with varying depths of dune sand and hillside deposits (approximately 15 to 20 feet) over Franciscan Complex bedrock below. To reach the supportive bedrock, the structure is supported on a deep micropile foundation system. The micropiles are 8-inch diameter, approximately 40 feet long, and extend through the layer of unsupportive soil into the existing bedrock below, as shown in Figure 17. Steel pipe casings are provided around the central rods down to the level of the bedrock to provide shear and compression capacity through the unsupportive sand layer.

Due to the configuration of the superstructure, a majority of the micropiles are located outside the perimeter of the existing below-grade parking garage and were installed with a conventional drill rig. To support the three columns that penetrate through the parking garage, three pile caps, each with multiple piles, are located within the lowest level of the parking structure. These three pile caps were required to be installed with a low-overhead drill rig that could access the lowest garage level, as shown in Figure 18.

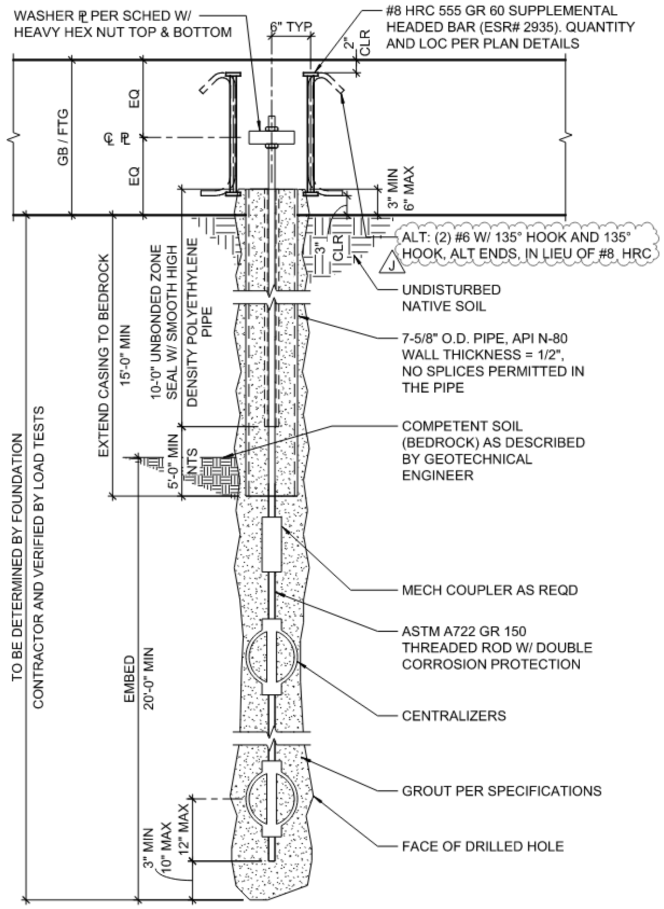

Figure 17: Micropile Detail

Figure 18: Low-Overhead Drill Rig to Install Micropile Foundations at the Bottom Level of the Existing Parking Garage

Due to the varying depths between the bottom of the new pile caps to the bedrock below, the piles were divided into four groups, as shown in Figure 19, with different design criteria for each group. The micropiles in Groups 1 and 2 were installed from grade and had minimal shear capacity due to the larger distance down to bedrock. The micropiles in Groups 3 and 4 were installed in the lowest parking garage level, within a few feet of the bedrock layer, and had much larger shear capacities.

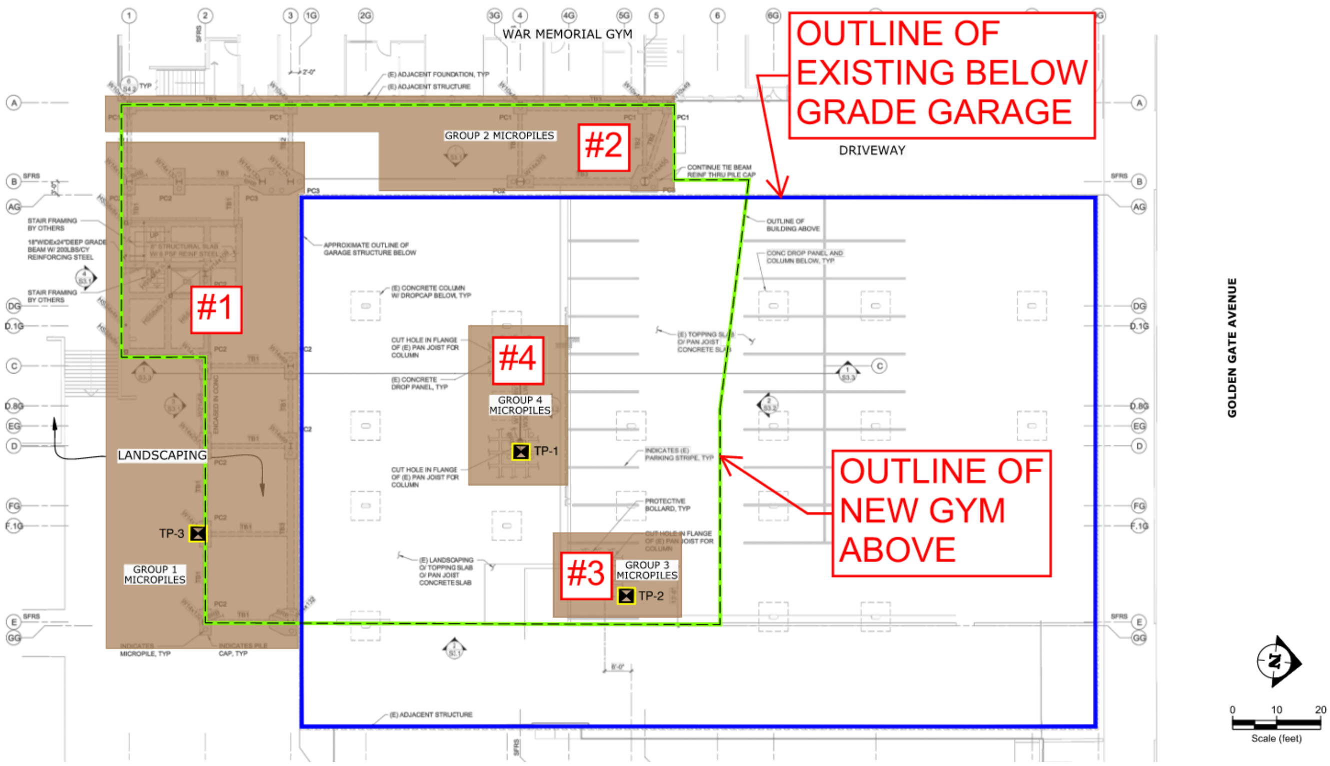

Figure 19: Plan View Showing Micropile Groups, Each with Different Design Criteria

The Group 1 and 2 micropiles that were installed closest to the perimeter of the existing garage are double-cased with steel pipe to avoid surcharging the below-grade basement walls, as shown in Figure 20. The outer casing is 12-inch-diameter, with an 8-inch-diameter inner casing, both extending down to bedrock. The void between the inner and outer casing is filled with a compressible foam to prevent any incidental lateral movement at these piles from imposing load on the surrounding soil or adjacent parking garage retaining wall.

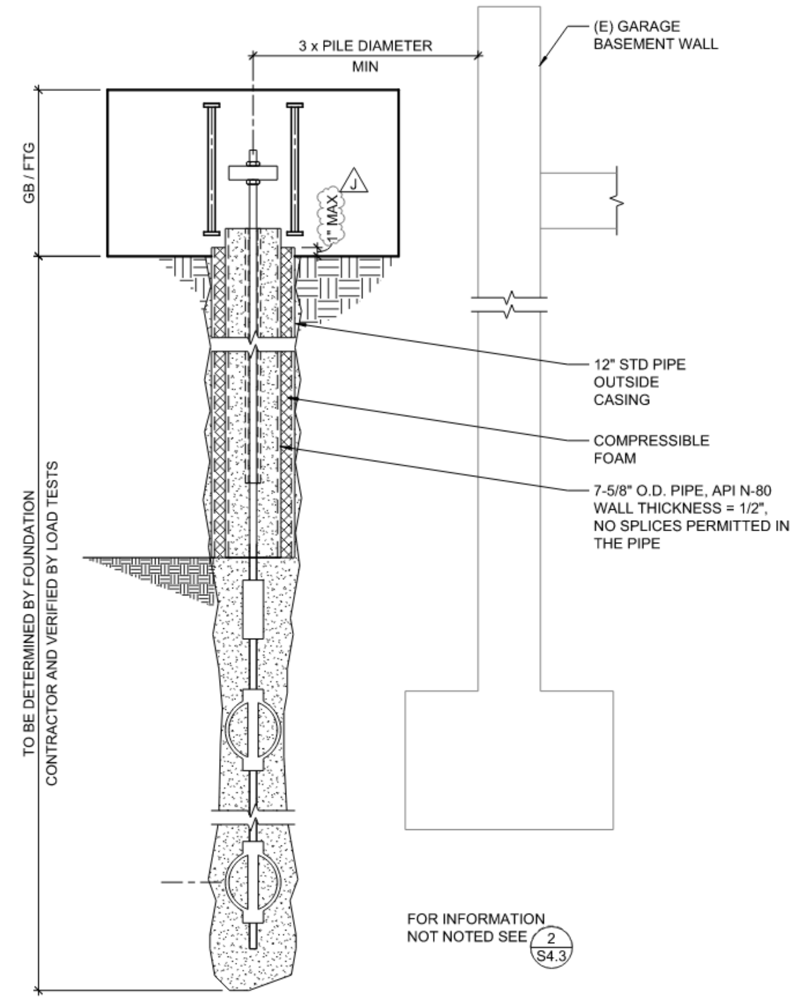

Figure 20: Double-Cased Micropile Detail

At the braced frame located within the parking garage, which is supported by the Group 4 micropiles in close proximity to bedrock, the shear capacity of the piles was adequate to support the induced lateral loads. For the remainder of the braced frames located outside the perimeter of the parking garage, lateral loads were transferred down to bedrock via raked micropiles. The raked micropiles were angled away from the existing parking garage and coordinated with other existing adjacent structures and utilities to avoid conflicts. The raked piles were integrated into pile caps at the bases of the lateral frames to provide stability. Concrete grade beams were added to interconnect all at-grade foundation components.

Where lateral frames outside the parking garage footprint were oriented perpendicular to the existing below-grade garage walls, seismic gaps were detailed between the pile caps and exterior face of the existing walls. These gaps were then filled with compressible foam, allowing a calculated amount of lateral movement to occur without surcharging the basement walls. The compressibility of the foam was used to calculate the required gap needed to avoid loading the walls during a seismic event.

Within the pile caps, a combination of headed stud reinforcement and steel embed plates were used to transfer the larger uplift and compression loads from the steel superstructure to the micropiles below, as shown in Figure 21.

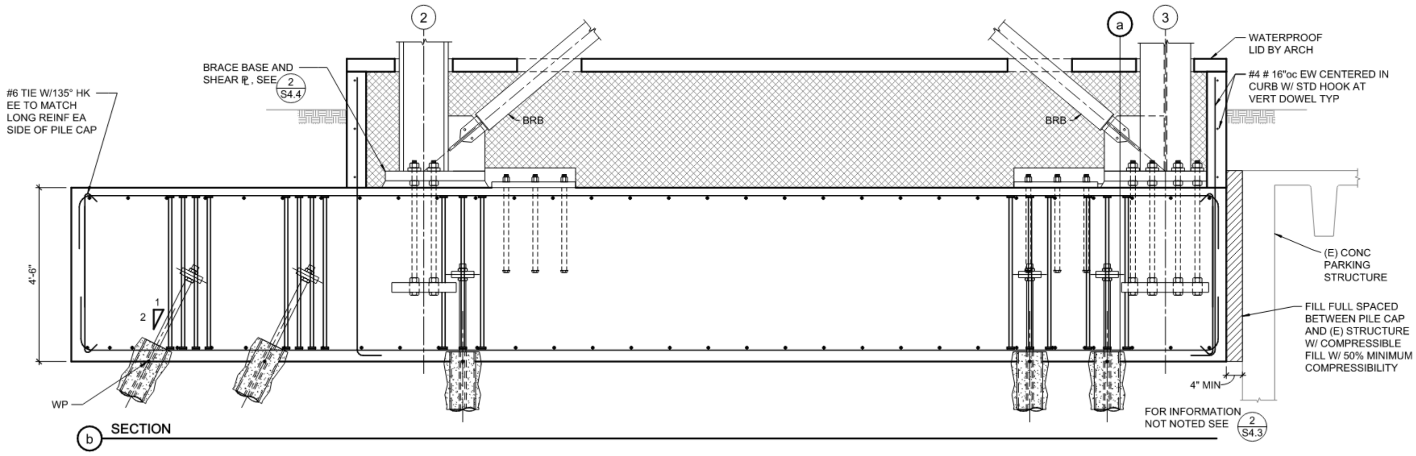

Figure 21: Pile Cap Detail at BRB Frame

Vibration Analysis

Sensitivity to vibration was a major concern for the University from the onset. The configuration of the gym was challenging for vibration sensitivity due to the long spans, proposed use for athletic activities, and essentially no interior partition walls to contribute to damping. As a result, analysis for excitation to vibration loads was a key design driver from the beginning of schematic design. After a straightforward design for strength of typical floor beams and girders, the floor layout was analyzed in FloorVibe for vibrations. As anticipated, the minimum member sizes required for strength design did not meet the tolerance limits for human comfort proposed in *AISC Design Guide 11*, *Vibrations of Steel-Framed Structures Due to Human Activity*.

Following the initial vibration analysis, several options were studied. Deeper and heavier beams could be provided to meet the vibration targets. This was the simplest solution and the first to be explored (considered the baseline). Second, the use of castellated beams was studied to provide acceptable performance at a reduced steel weight. This option proved to be structurally feasible from a vibration and strength design standpoint. Castellated beams are made from a single wide flange section that is cut in a hexagonal pattern in the web and then joined together through welding to provide additional depth. The process results in a member that is much stiffer than the original section but requires additional processes that adds fabrication cost. To thoroughly vet this option, ZFA coordinated directly with the American Institute of Steel Construction (AISC) to determine the fabricators in the region who provide castellated beams. The nearest fabricator on the list was in Oregon. After initial discussions about the project, the fabricator agreed to provide limited pricing information to help guide the decision-making process. They were able to provide general comparative pricing for the castellated beam option compared to the baseline option, assuming that they would be providing steel for either option. Shipping costs were not included, and this exercise was performed early enough in the design process that only best guesses could be made about overall quantities. Despite the unique challenges, this project was deemed not large enough to benefit from any economy of scale. Still, castellated beams appeared to be slightly favorable for material costs and fabrication with caveats that included increased lead times, limited availability, and higher shipping costs.

A third option was studied that allowed the floor system to be designed for strength alone and utilize Tuned Mass Dampers (TMDs) to dampen vibrations to acceptable levels. An initial feasibility study was performed using the test case from the FloorVibe model and minimum member sizes were shared with a TMD manufacturer for consideration. After confirming feasibility, the TMD concept (along with others described above) was shared with stakeholders. Weighing the pros and cons of each alternative, the decision was made to proceed with W40x149 floor beams and 60-inch-deep steel plate girders (option 1, baseline option). Since the project was being funded by donations, there was a sensitivity to potential schedule delays since donors would be more interested in seeing the project completed quickly as opposed to marginal savings. As for the use of TMDs, the relative savings on the overall project costs was not well defined and ZFA's best estimate was not substantial enough to outweigh the perceived risk. The decision was made, and design of the rest of the building commenced.

As permit documents were being finalized, ZFA revisited the vibration design to confirm the original conclusion accounting for the variety of changes that had occurred during the design period. After further review, it became clear that the fundamental assumptions of the previous vibration analysis did not capture the behavior of this unique structure. Traditional methods, and the analysis software used in schematic design, are based on a floor system of beams and girders supported by columns in a rectangular grid. This project had a framing system that consisted of a floor hanging from a full depth roof truss, which was in turn supported by two outrigger roof trusses, one of which was supported by a cantilevered skewed floor girder. The floor was supported by the roof, which was supported by the floor—a complex load path that exceeded the limitations of the analysis software that had previously been used. For half the gym floor, there was now uncertainty about how the system would perform for vibrations.

To address this issue, a Finite Element Analysis for vibrations was performed following the guidelines outlined in Chapter 8 of *AISC Design Guide 11*. The analysis model for the design of the lateral system was adapted for vibration analysis by eliminating pinned connections and applying stiffness modifiers to perimeter elements that had rigid spandrel elements attached. A modal analysis was performed to determine hot spots where vibrations could be an issue, and one particular point of interest became apparent—an area of the gym floor adjacent to the center span of the north elevation, as shown in Figure 22. Next, a series of linear time history analyses were performed to simulate vibration loads on the gym floor over an area that influenced vibrations of the point of interest. Using the floor response to these time history analyses along with the results of an eigenvalue analysis, a Frequency Response Function was developed, which enabled computation of peak floor acceleration. Through this analysis, it was confirmed that the roof's contribution to floor deflections was causing floor acceleration limits to be exceeded. After a couple design iterations, a stiffened roof truss was developed that limited floor accelerations to recommended limits, and the drawings were quickly updated for final submittal.

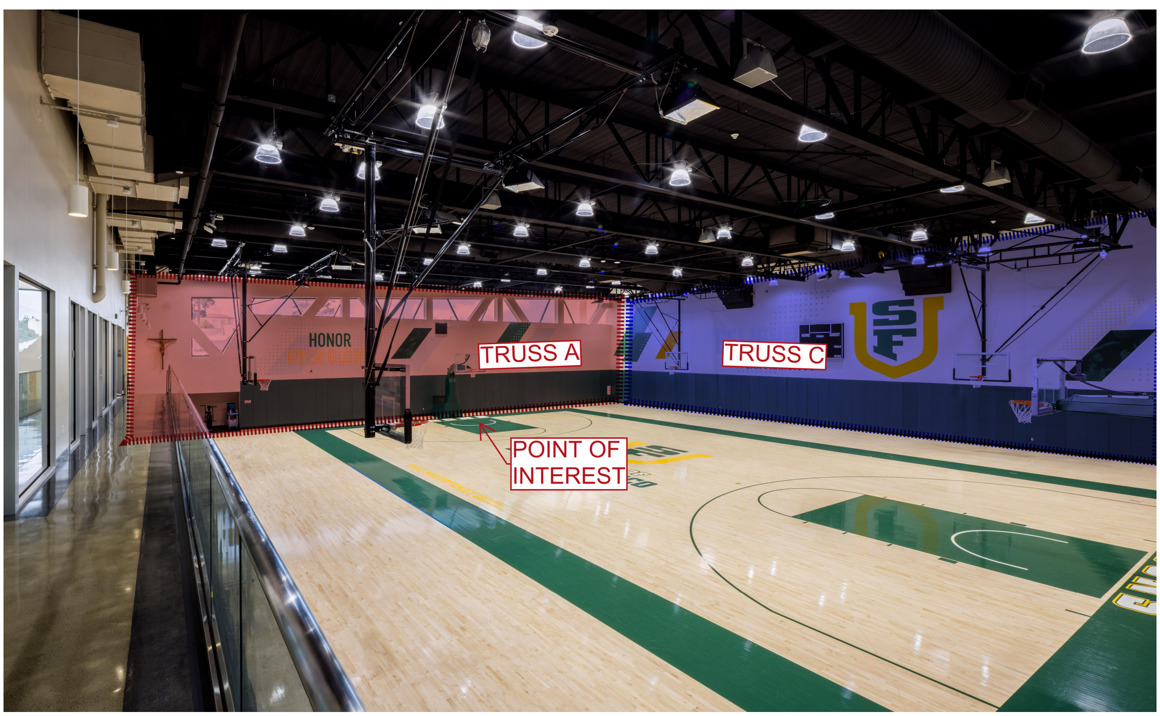

Figure 22: Interior of Finished Gym Looking North, Showing "Point of Interest" for 3D Vibration Analysis

Sequencing and Construction

Due to the complexity of the design and required coordination with existing site conditions, thoughtful sequencing was a critical part of the construction process. Additionally, multiple field conditions were discovered during construction that required modifications to the design.

One field condition was the discovery of underground utilities below the fire access road along the west face of the building, as shown in Figure 23. The utilities conflicted with several grade beam and pile locations, requiring localized modifications to the pile layout and foundation configurations.

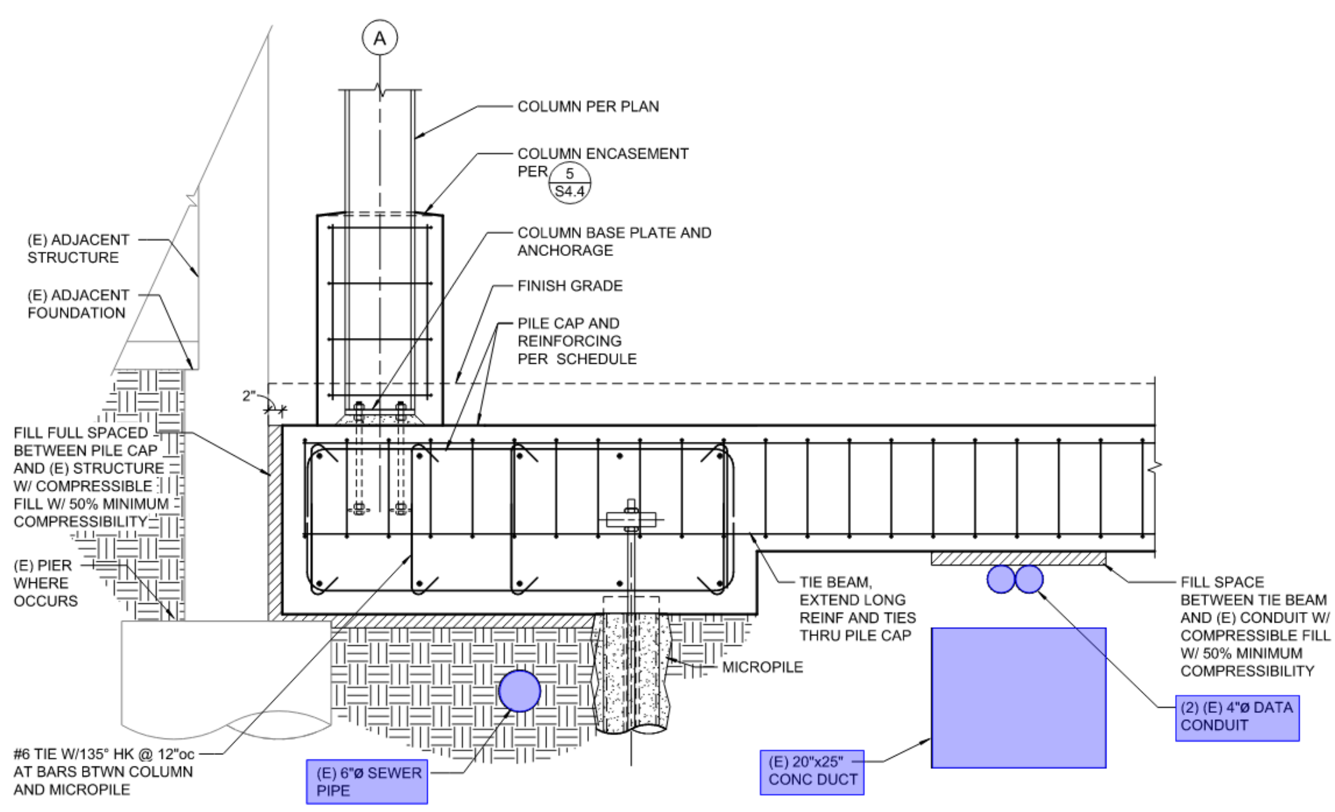

Figure 23: Existing Utilities Discovered Under a Proposed Pile Cap Location

Due to the limited space at the construction site, along with the required construction activities, the top level of the existing parking garage was required to be used for both the storage of construction materials and for temporary support of the new structure. These temporary loading conditions required shoring to be installed within all levels of the underground parking structure. At the storage areas, shoring posts were installed in a grid throughout each level. For the temporary support of the new structure, shoring posts were installed below exterior wall lines to enable the wall trusses to be erected and field welded piece by piece, as shown in Figure 24. These shoring posts were also required to be installed at each level of the garage to transfer loads down to the supportive soil below. Once all the components of the wall trusses were installed, fully welded, and inspected, the shoring posts were removed to allow the superstructure to support itself as designed.

Figure 24: Temporary Shoring Posts (Green) To Support Long-Span Truss During Erection

Additionally, the following construction management strategies enabled the project to come in under budget and on schedule:

- Effective time management – The project team worked with its trade partners to maximize workflow efficiency and reduce downtime. This was accomplished by holding pull planning sessions well in advance of construction. As a result, the project was able to avoid significant schedule delays.
- Proactive approach to information control – When an unforeseen condition was encountered, the project team worked to identify 'no-cost' solutions. This was accomplished by collaborating with the design team and the University to find suitable solutions that reduced costs while maintaining the design intent.
- Early release of materials – The project team identified long lead items well in advance of construction and worked with the University to provide early authorization to order these materials. In some cases, the project team stored these materials on-site for months. The result of these actions reduced the risk of escalation and delays in material procurement.