A 3-D SEISMIC ISOLATION SYSTEM TO PROTECT THE LOMA LINDA UNIVERSITY HOSPITAL FROM NEAR-FAULT EARTHQUAKES

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Abstract

The Loma Linda University Medical Center (LLUMC) Replacement Hospital is a new first-of-its-kind three-dimensional (3-D) seismically isolated 1,000,000 square foot (100,000 square meter) acute care hospital and Level 1 trauma center serving the 4 million people living in the Inland Empire region of Southern California. The replacement hospital is located 1 km from the M7.9 capable San Jacinto Fault and 12 km from the M8.2 capable San Andreas Fault, which site-specific seismic hazard analysis predicts to generate MCE spectral accelerations of $S_{M1,horizontal} = 2.31g$ and $S_{MS,vertical} = 2.92g$. The hospital structure incorporates a 3-D seismic isolation system to reduce the impact on the structure and protect the critical contents which must remain functional after the design earthquake. The 3-D isolation system couples a traditional lateral isolation system of triple friction pendulum isolators and large stroke fluid viscous dampers with a novel vertical isolation system (VIS) of steel pedestals, helical coil springs, fluid viscous dampers, and low friction sliding shear pins. This paper discusses the conception and design of the VIS, effectiveness of the VIS to adequately reduce the vertical ground motion effects on the building structure and contents using LS-DYNA for NLRHA, and the phased installation and permitting of the VIS underneath a newly constructed building.

Keywords: near-fault excitation; 3-D seismic isolation; performance-based design

1. Introduction

The replacement hospital for Loma Linda University Medical Center is the centerpiece of a Campus Transformation Project for Loma Linda University, the most ambitious capital campaign ever undertaken by this institution. When completed in 2021, the new acute care hospital building will replace the 1970s era SPC-1 nonductile-concrete adult hospital towers (dubbed the "cloverleaf" due to its distinctive plan) which will no longer be in compliance with California hospital seismic safety standards per SB 1953 and SB 90. The building is comprised of two patient towers above a shared diagnostic and treatment podium. The Pediatric tower will have 349 licensed Pediatric beds and the Adult tower will have 288 licensed Adult beds. Shell floors in both towers allow for future expansion. The total building floor area is approximately 1,000,000 square feet and the taller Adult tower reaches 267 feet tall. Fig. 1 shows a rendering of the new replacement tower with the existing hospital towers in the background. The design team is comprised of NBBJ (architecture and medical planning), Stantec (electrical and low voltage engineering), and Arup (structural, mechanical, plumbing, fire protection, and civil engineering). The general contractor is McCarthy Building Companies and the project manager is Jtec, HCM.

Fig. 1 – Architectural rendering of the replacement hospital with the SPC-1 cloverleaf building in the background (*Image: NBBJ*)

The structural system of the building comprises of a steel frame on top of a 3-D seismic isolation system, which in turn is founded on a concrete mat foundation. The 5-story podium is a single contiguous structure with a footprint of approximately 350' by 250'. The towers are seismically separated above the last contiguous podium floor with a movement joint at the shared tower elevator core. The open nursing core concept selected for the layout of the medical/surgical and ICU floors in the patient towers led to long but narrow 60' wide floor plates. This results in relatively slender towers and, to achieve the required structural stiffness, buckling restrained braced (BRB) frames are used in the tower short direction. SidePlate moment frames are used in the tower long direction. These frames spread outward across the width of the podium footprint to provide overturning stability on top of the seismically isolated base.

2. Seismic Site Context

The site is within 1km of the San Jacinto Fault (SJF) zone, capable of a M7.9 characteristic earthquake. Fig. 2 illustrates the relationship of the site with the surrounding fault context, which includes the San Andreas Fault (SAF) zone approximately 5 km away. Studies by Stephenson, et al (2002) indicate that the site is located on the edge of a basin between the San Jacinto and San Andreas faults, with a series of minor fault traces throughout the basin. This highly seismic fault zone underlies most of the Inland Empire region of Southern California making the entire region highly susceptible to catastrophic damage in the event of a fault rupture along either the SJF or SAF. Since LLUMC is the primary provider of acute care in this region of 4 million people, and the only pediatric trauma center, the resilience of the new facility in the face of this the severe seismic hazard is of critical regional importance.

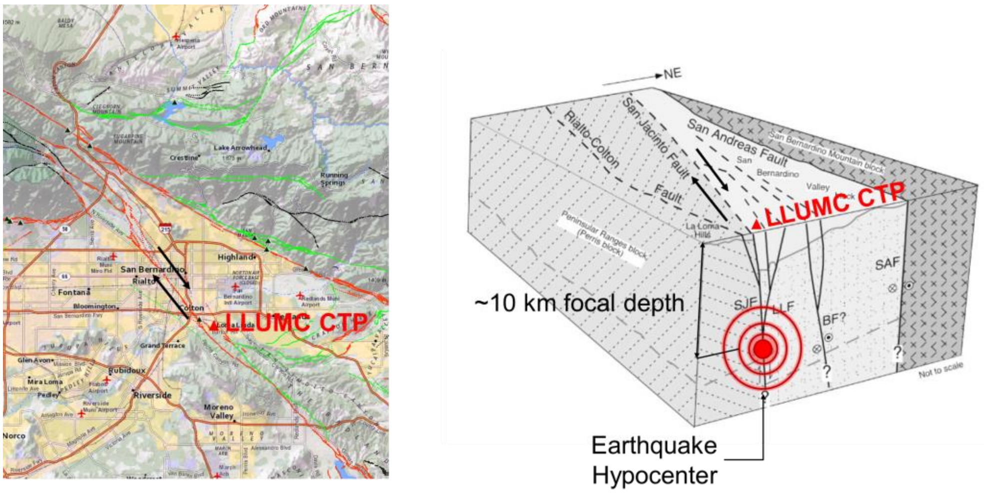

Fig. 2 – Site location relative to the local fault zone in 2-D (left) and 3-D (right) block diagram adapted from Stephenson, et al [1]

A site specific seismic hazard assessment was performed by GeoPentech resulting in the following design parameters:

- Site Class D
- Seismic Design Category F
- SMS = 2.10g
- SM1 = 2.31g

Due to the site-specific procedure specified in ASCE 7-10 to generate the S_{MS} and S_{M1} ordinates and the particular shape of the response spectrum which has high energy at long period, the S_{M1} ordinate is greater than the S_{MS} ordinate [2]. The response spectrum at Maximum Considered Earthquake (MCE) is plotted in Fig. 3 and is given for horizontal fault-parallel, horizontal fault-normal, and vertical ground motion directions. The design team opted to use nonlinear response history analysis and 11 sets of three-dimensional spectrally matched MCE ground motion time histories were provided by GeoPentech to Arup. These 11 sets of ground motions were factored by 2/3 to arrive at the Design Earthquake ground motion time histories. The ground motions included a high proportion of near-fault pulse-like features given the immediate proximity of the site to the major fault.

Two items are of particular note in the response spectrum. First, the fault-normal MCE spectrum is above 1.0g lateral spectral acceleration at 2 second period, corresponding to a high spectral displacement demand for a tall building. This is exacerbated by the pulse-like nonstationary characteristics of the project specific ground motions which act to further increase the displacement demand when applied in the time domain to a nonlinear response history analysis of a yielding lateral system. Given the architectural design called for a slender 17-story 267-foot-tall hospital building, a fixed base structural system resulted in impractical member sizes and a structural layout that would have been non-functional for medical planning due to requiring lateral framing on nearly every grid line.

The second item of note is the severe vertical ground motion, which peaks at 2.69g at 13.3 Hz and remains above 1.0g until approximately 2 Hz. Such a severe vertical hazard is poorly represented by the $0.2S_{DS}$ (effectively 0.28g) vertical ground motion allowance in the building code since the vertical frequencies of the floor system and columns are between 4 Hz and 10 Hz.

Based on these two drivers, it became clear to the design team that a seismic isolation response was required to achieve the immediate occupancy requirements at Design Earthquake (DE) stipulated by the CBC while also protecting the nonstructural contents and occupants from excessive floor accelerations.

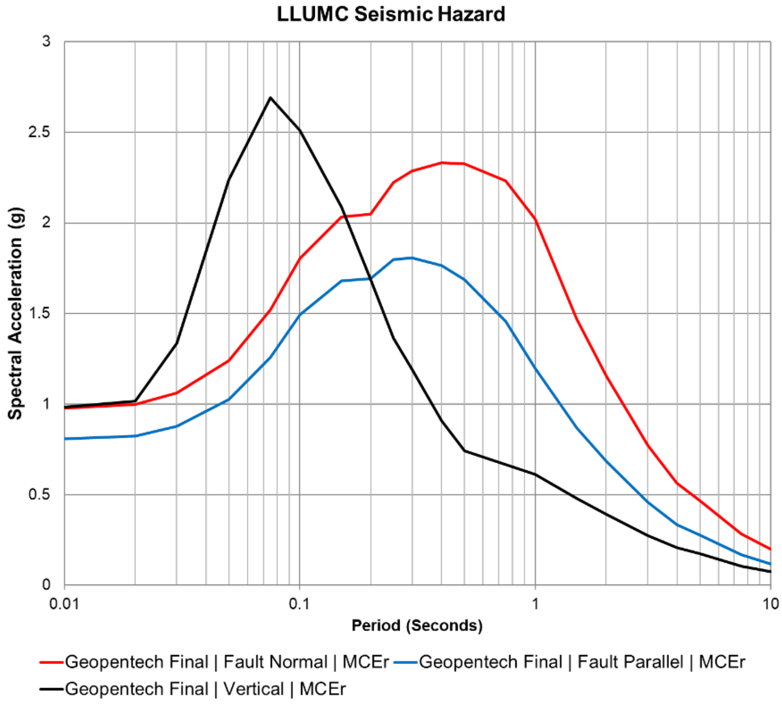

Fig. 3 – Horizontal (fault normal and fault parallel) and vertical seismic hazard response spectrum at MCE

3. Structural System Above the Isolation Plane

A structural steel frame was chosen for the superstructure of the new hospital tower due to its lower seismic mass and increased flexibility relative to a concrete frame. The architectural design of the patient towers called for a plan offset at the center, creating two distinct wings, each of which is only 60 feet wide but 150 feet long. The lateral framing strategy places SidePlate® moment frames along the long faces of each of these wings, where there are enough contiguous bays to make a moment frame effective, and places the stiffer and more intrusive BRB frames at the core of the tower and at the ends of the wings. Pushing the lateral structure to the perimeter of the tower and adopting shallow column sections as the horizontal beam elements of the BRB frame (a necessary feature to achieve the immediate occupancy plastic rotation acceptance criteria of ASCE 41-06 for heavy axially loaded beam elements [3]), allowed the floor to floor heights to be compressed. An isometric view of the structural Revit model is shown in Fig. 4. The interstory drift limit for the building is 1.5% per ASCE 7-10 based on the use of nonlinear response history analysis on top of a seismic isolation system. This is slightly more flexible than the allowable 1.25% interstory drift limit permitted for a fixed base building using nonlinear response history analysis. In addition to the drift limit, the nonlinear components of the building were limited to the acceptance criteria of ASCE 41-06 modified by Chapter 16A of CBC 2013, listed in Table 1 [4]. For additional information regarding the practical implementation of ASCE 41 and nonlinear response history analysis, including the cloud processing workflow and management of 110 LS-DYNA models, refer to Nielsen et al and Zekioglu et al [5][6].

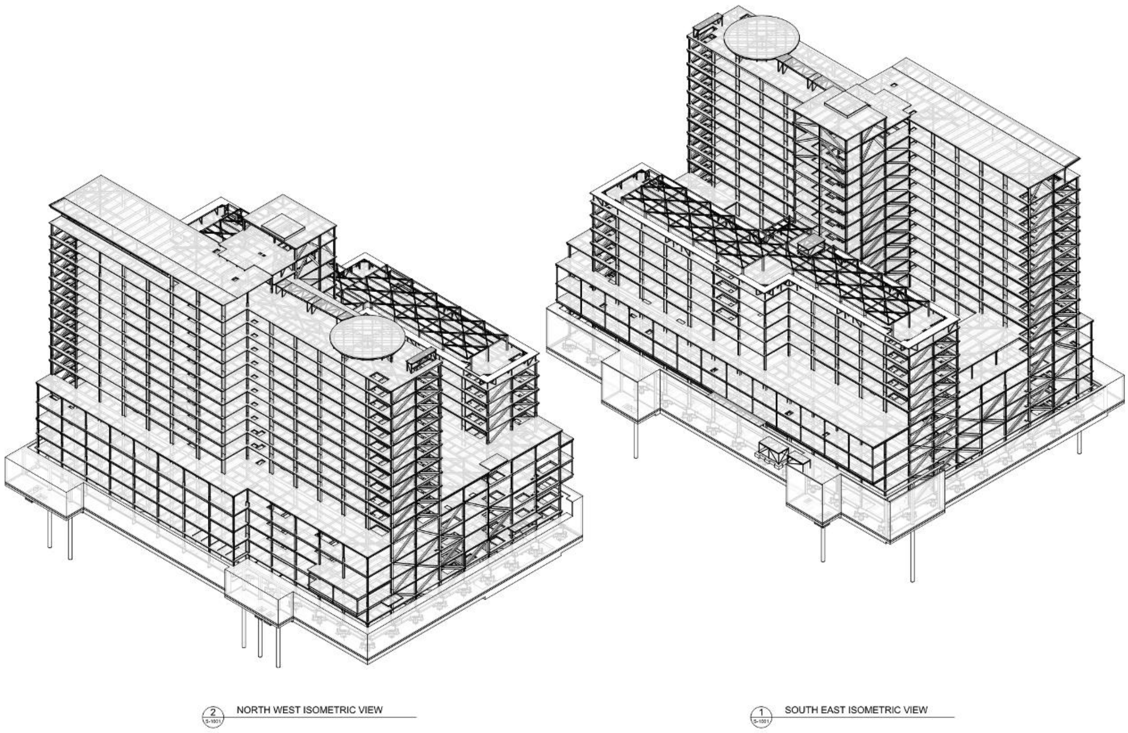

Fig. 4 – 3-D isometric of structural Revit model

Table 1 – Performance requirements for NLRHA

Element	Performance at DE	Performance at MCE
FPT Isolator	LRFD Design	Expected Strength Design
FV Damper	LRFD Design	Expected Strength Design with 1.5 FOS
Mat Foundation	LRFD Design, Settlement < 1.5”	Expected Strength Design, Settlement < 6”
Level A Isolator Framing	IO, 1 θy	IO, 1 θy
SidePlate Columns	IO, 0.25 θy	IO, 0.25 θy
SidePlate Beams	IO, 0.02 radians	LS, 0.03 radians
BRBF Columns	IO, 0.25 θy	IO, 0.25 θy
BRBF Beams	IO, 0.25 θy	IO, 0.25 θy
BRBF Braces	IO, 3 Δy	LS, 10 Δy
Drag Connections	LRFD Design	Expected Strength Design
Diaphragms	LRFD Design	Expected Strength Design

To prevent excessive uplift and maintain overturning stability of the towers even under the reduced base shear of the seismically isolated building, the braced frames are linked within the podium creating a multistory outrigger base that spans the entire North-South width of the podium. Four of these outrigger lines cross through the podium with two of the lines surrounding the core and the other two lines on the perimeter faces of the podium. This divides the diagnostic and treatment portion of the podium into two natural programmable halves on the East and West, with a natural Pediatric and Adult separation occurring in the North and South due to the placement of the two towers. The moment frames also widen in the podium to keep the beam section sizes within the 300 lb/ft beam weight AISC 358 prequalification limit for the welded SidePlate® connection. Sample elevations of the BRB and moment frames is shown in Fig. 5. ASTM A913 Grade 65 material was used for the gravity and frame columns, as well as for certain BRBF beams to delay the onset of yielding due to imposed frame deformations.

The final selected structural design uses a lateral base isolation system comprised of 126 triple friction pendulum bearings manufactured by Earthquake Protection Systems (Vallejo, CA) with +/- 42” displacement capacity and up to 15,000 kips axial capacity, and 104 fluid viscous dampers with 800-kip MCE capacity manufactured by Taylor Devices (North Tonowanda, NY). The friction pendulum isolators have an effective period of 4.5 seconds and the dampers have a velocity exponent of 0.7. The total equivalent system damping coefficient is 50% of critical damping. High damping using supplemental dampers was selected to control the overall building displacements and reduce reliance on the friction pendulum system for system damping, which is affected by the changing vertical load due to the high site-specific vertical ground motion component. Isolator displacement would have been 84” without supplemental damping. Reducing the isolator displacement to 42” resulted in an optimal cost solution by controlling the isolator and damper component costs, the costs associated with stability framing above and below the isolators, and the costs associated with expansion joint covers and flexible service connections.

Stabilizing the top of the isolators is a grid of 60” deep steel moment connected plate girders designed for Immediate Occupancy performance at MCE. Most of these moment connections are bolted connections to control weld induced distortion in the field during steel erection.

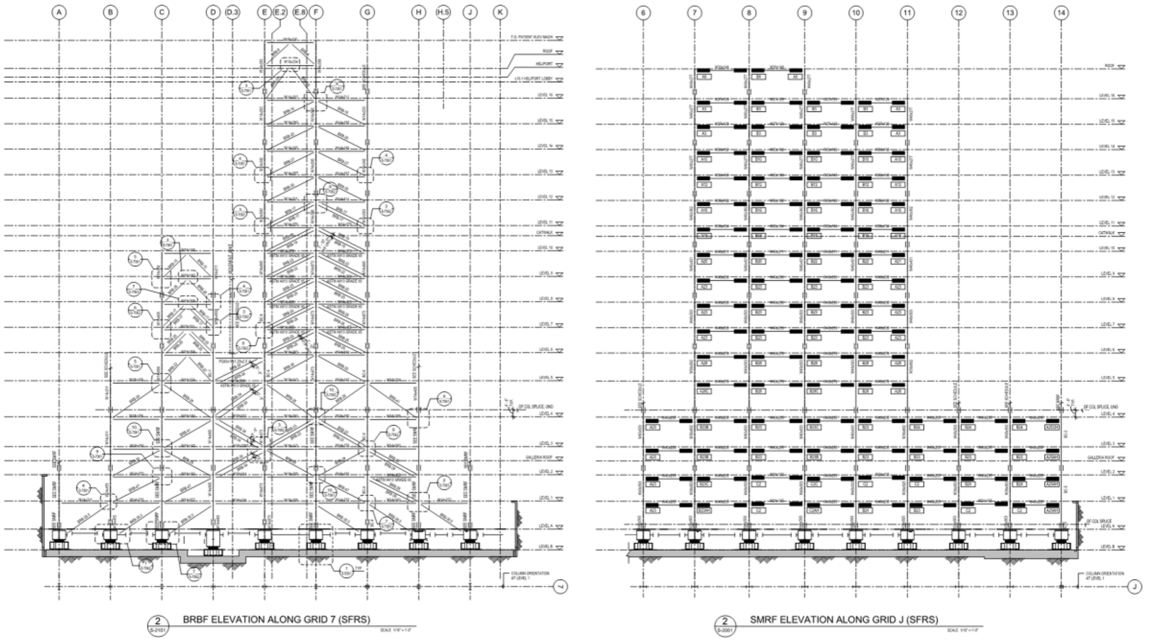

Fig. 5 – Lateral frame elevation of BRB frame (left) and SidePlate® moment frame (right)

4. Vertical Ground Motion Study and 3-D Isolation System Response

Investigation of the effect of the vertical ground motion time histories on the overall structural behavior indicated that there was a risk that without mitigation, the gravity column and beam capacities, using code minimum design forces, would be exceeded. In addition, it was observed that applying the vertical ground motions to the friction pendulum bearings alone (without the contribution of supplemental damping) could lead to increases in isolator displacement of ~15% due to the biased reduction in shear resistance due to vertical acceleration with an amplitude more than 1g. While the isolator effects were addressed by the addition of supplemental damping, the impact of the vertical ground motion on the vertical force resisting system was deemed to be consequential, and while not code mandated, impossible to ignore. Two possible design approaches were developed to address the vertical ground motion (VGM) hazard:

The first approach was a hardening and strengthening approach to address this ground motion not mandated by the building code. It was noted that the intent of this approach was simply to prevent collapse of the structure at MCE or detachment of heavy components at the DE which might otherwise result in significant loss of life under the predicted VGM. This proposed hardening approach included the following:

- Critical elements, the failure of which would be brittle and lead to collapse, will be strengthened so that their capacities are not exceeded under the predicted VGM.
- Secondary elements which may fail in a ductile manner will not be strengthened but their connection details will be made more robust to ensure ductile response.
- Exterior envelope design criteria will be amended to include provisions for ductile instead of brittle behavior so that panel fallout does not occur under the predicted VGM. This will affect only the attachment details of the envelope components.
- Anchorage of non-structural components also strongly recommended to also adopt an approach to keep heavy components from detaching from the structure.

The second possible approach was to implement a sprung foundation, or what became known as the vertical isolation system (VIS). Preliminary analysis showed that implementation of the VIS would result in significantly reduced VGM being transmitted into the building (see) and that it was feasible to limit the induced forces in the vertical load resisting structure to a similar level as the code LRFD load combinations (e.g. LRFD combinations with the inclusion of 0.2SDS*D). Fig. 6 compares the vertical floor spectra at Design Earthquake level for the case of a hardened structure without VIS (left) and a structure with VIS (right) and demonstrates the effectiveness of the VIS to significantly reduce the floor spectral accelerations.

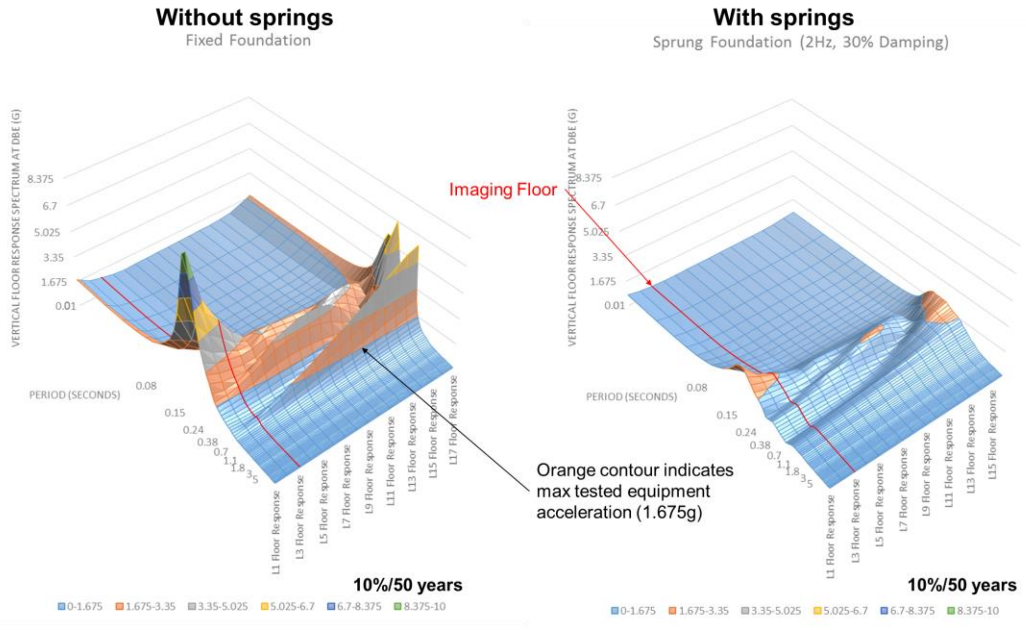

Fig. 6 – Comparison of vertical floor spectra at DE level: a hardened structure without VIS (left) and a structure with VIS (right)

However, the hurdles to achieve approval of this innovative system, which had not been implemented on a hospital in the US before, and the apparent schedule delays did not initially allow it to be seriously considered. After pursuing the hardening and strengthening VGM approach to a level where the design could be costed, it became clear that the schedule and cost implications of this approach were quickly becoming insurmountable, particularly considering the nonstructural element anchorage design.

This led the design team to consider the VIS as a potential strategy again, with a breakthrough coming in the form of an idea to separate the construction schedule of VIS from the overall building permitting and construction schedule. This idea is illustrated in Fig. 7. Instead of placing the VIS directly underneath the triple pendulum bearings, a steel pedestal was created supported on steel shims that would allow the springs and dampers to be mounted to the side later (Fig. 7c). When the VIS was ready for installation, the pedestals could be lifted approximately ¼” to allow the shims to be removed and the VIS fully engaged (Fig. 7d). Decoupling the VIS design, permitting, testing, and installation from the overall construction of the building allowed a separate permit to be obtained for the construction of the building absent VIS, and another later permit process to be undertaken to complete the full due diligence, prototype testing, and analysis required for the VIS. In this way, the VIS became a pre-planned “voluntary seismic upgrade” to an otherwise code compliant building.

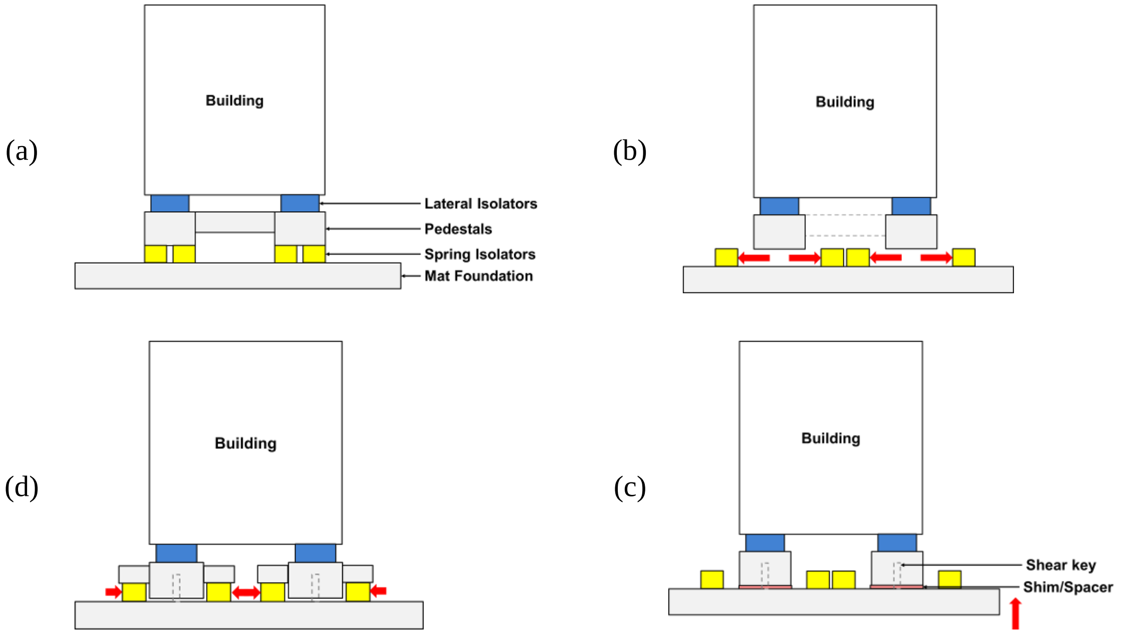

Fig. 7 – Concept development of VIS installation: (a) idealized design (b) requisite removal of springs from initial vertical load resisting path, (c) temporary construction stage, (d) final construction stage with VIS installed

5. Practical Implementation of 3-D Isolation

Implementation of the full 3-D isolation system required two separate OSHPD permit increments as part of the design and construction process. Increment 2 included the primary structural system with the horizontal seismic isolation component included. Increment 9, which is still in review as of this writing, includes the VIS components. The VIS comprises the following discrete components: helical springs manufactured by Gerb (Essen, Germany) and vertical fluid viscous dampers and custom low-friction sliding shear pins manufactured by Taylor Devices. The design of the VIS has a target sprung system frequency of 2.5 Hz with a system damping of approximately 50% of critical damping. The system frequency was selected to provide sufficient isolation effect while keeping the spring displacement under anticipated variation in live load to a manageable ¼” or less. The spring deflection from unloaded to the sprung weight is approximately 1.5”. Additional viscous damping was used to further control the dynamic motion under the VGM. To counter the local overturning effects due to isolator offset, as well as the global overturning effects due to the superstructure lateral system, a stiffening spring behavior was adopted where the initial spring stiffness corresponding to 2.5 Hz is effective up to twice the sprung weight, at which point a second spring is engaged with 4 times the stiffness. This stiffening behavior prevents excessive rocking instability of the VIS system. The arrangement of the VIS is illustrated in Fig. 8, Fig. 9, and Fig. 10.

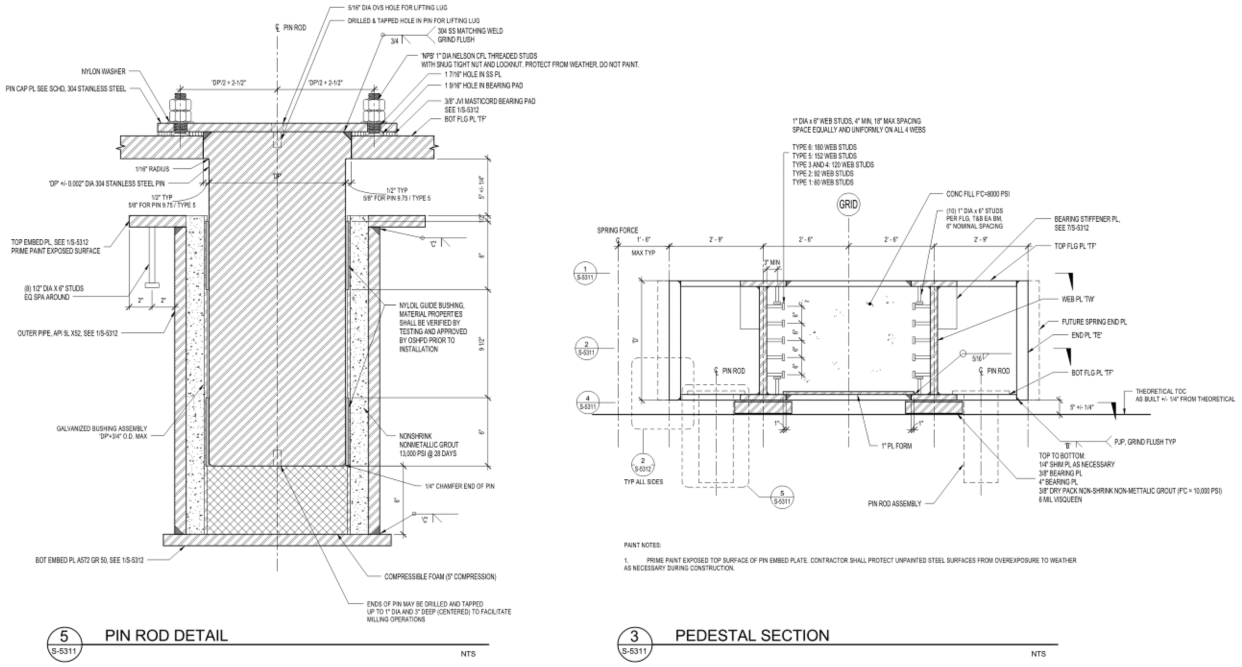

Fig. 8 – Structural details of VIS pedestal and sliding shear pin

As part of the Increment 9 plan review of the VIS, a series of analytical validation and prototype testing is being performed and listed below:

- LS-DYNA analysis of the overall structure with the inclusion of VIS, subjected to wind and combined 3-D earthquake effects
- LS-DYNA validation analysis of a 3-D isolated transformer component previously tested at full scale at the University of Buffalo [7]
- Full scale prototype and production testing of the sliding shear pin devices
- Full scale prototype and production testing of the vertical fluid viscous damping devices
- Full scale prototype and production testing of the helical spring devices
- Full scale actuator testing of a 1500 kip sprung weight VIS pedestal assembly, including triple pendulum isolator, sliding shear pins, vertical dampers, and helical springs, at the SRMD facility at UC San Diego

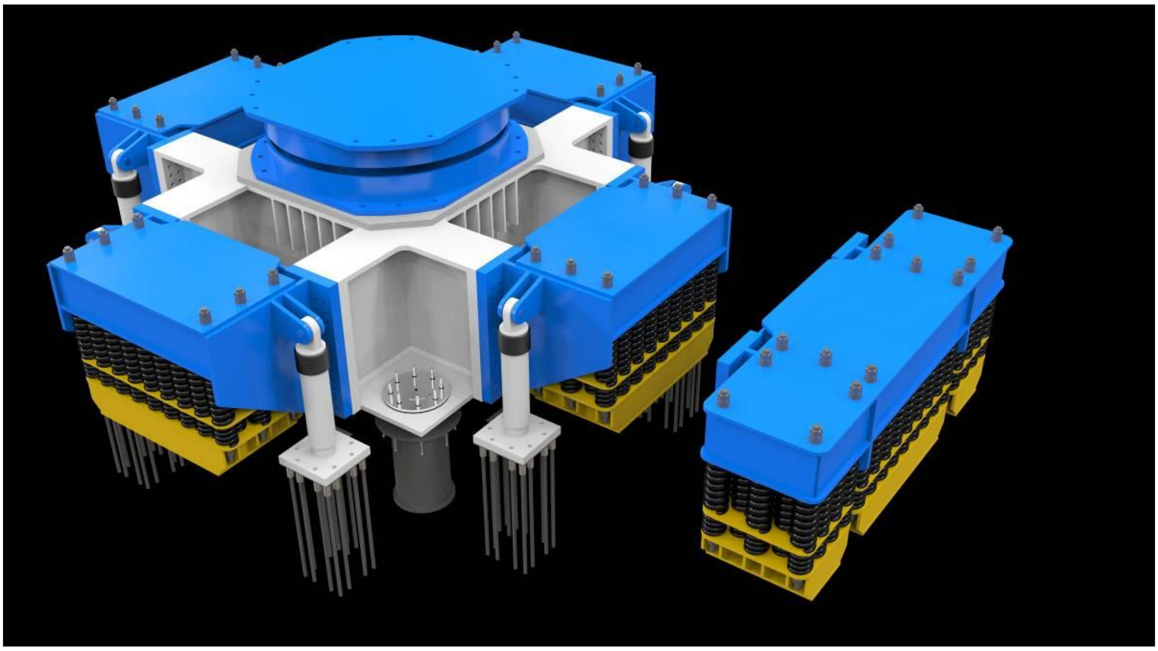

Fig. 9 – Rendering of the VIS system: a set of four 49 springs unit mounted to pedestal; a single 84 spring unit shown on the side.

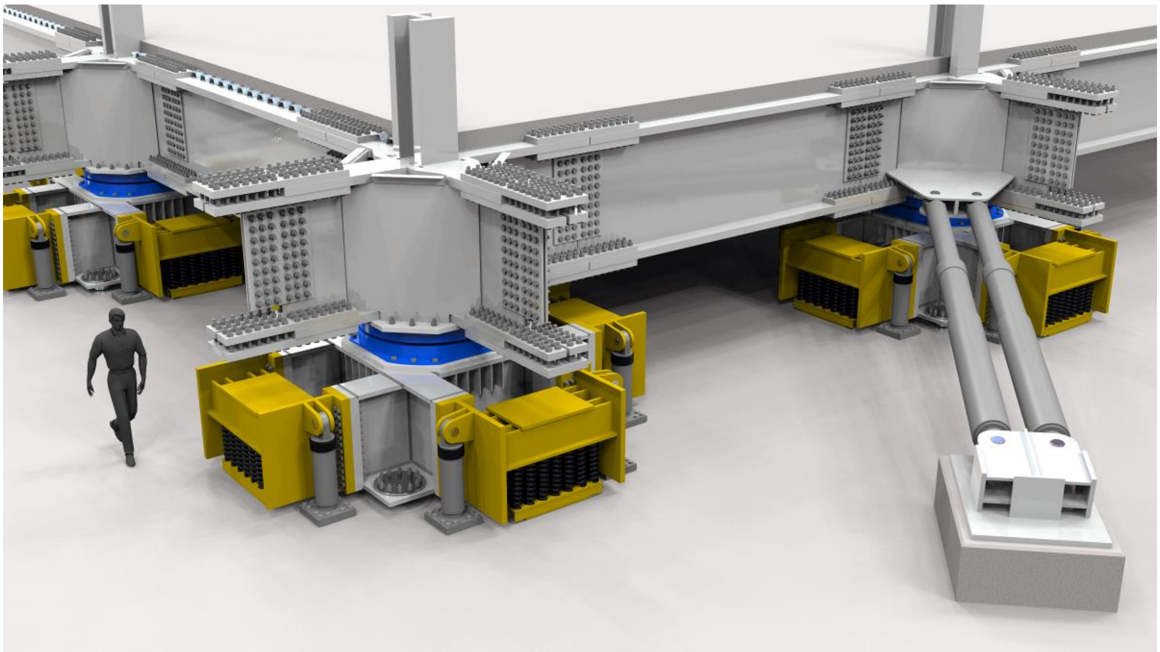

Fig. 10 – Rendering of the installed condition with the VIS in place

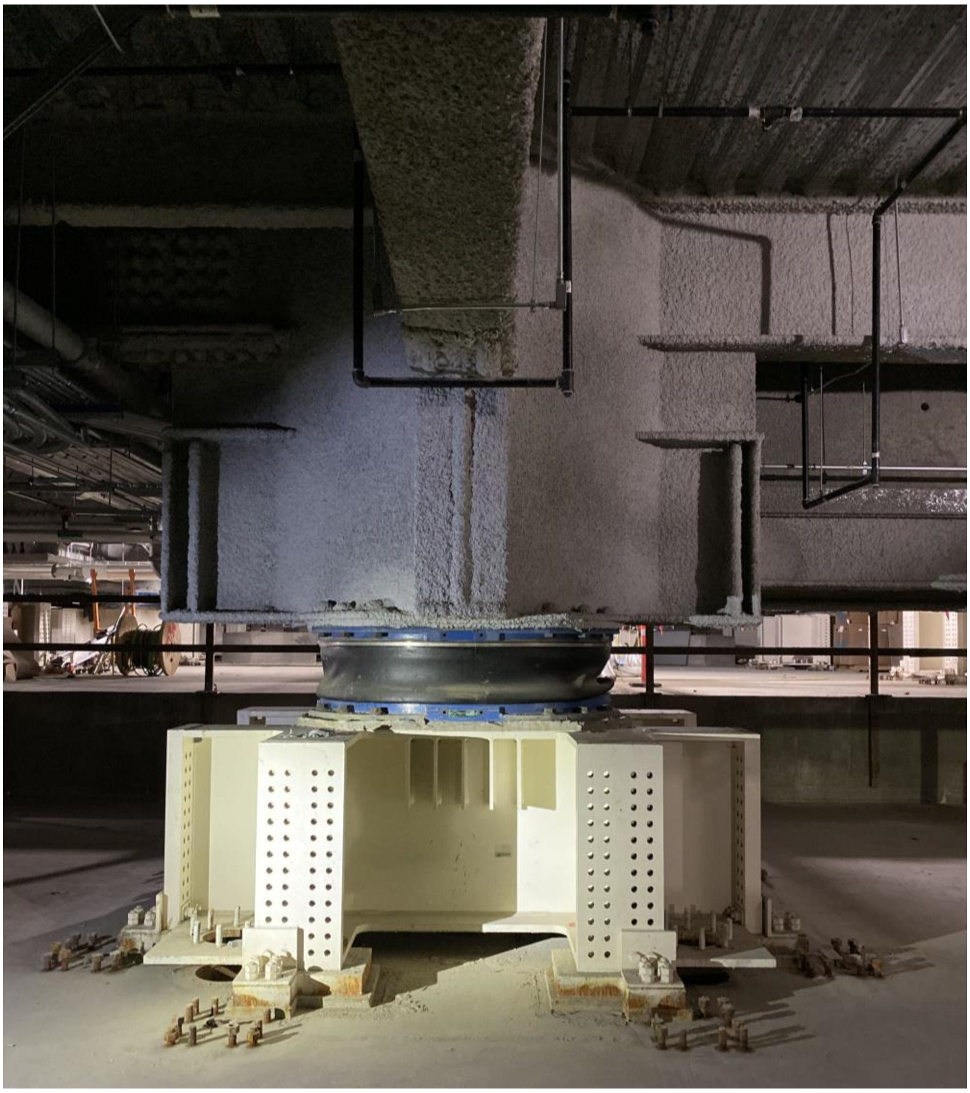

Fig. 11 – As-built condition of the VIS pedestals supported on grouted steel shims

5. Acknowledgements

The authors would like to express their gratitude to the following parties for their contributions in developing the LLUMC Campus Transformation Project: Loma Linda University Health, Jtec HCM, NBBJ, McCarthy, Dr. Roy Lobo and Mr. Tom Hale at OSHPD, Dr. Michael Constantinou, Dr. Farzad Naeim, Mr. Bret Lizundia, Taylor Devices, Earthquake Protection System, and Gerb.

5. Copyrights

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