Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1379

BASE ISOLATED STRUCTURE – THE NEW SAN FRANCISCO GENERAL HOSPITAL & TRAUMA CENTER

E. Ko[1], T. Morgan[2], M. Bello[3], R. Bailey[4], R. Golesorkhi[5], L. Lam[6], and S. Tong[7]

ABSTRACT

The new San Francisco General Hospital & Trauma Center (SFGH) will be a base-isolated structure located 10 miles from the San Andreas Fault, on Potrero Avenue between 22nd and 23rd Streets. This state-of-the-art healthcare facility will house acute care services and support space for more than twelve departments that provide critical functions for the City and County of San Francisco. The nine-story steel-framed structure will be 140 feet tall and will provide 448,000 square feet of floor and roof area. A three-dimensional analysis model was built and non-linear response history analyses were performed; the overall structural performance objective was to ensure that the superstructure would remain elastic when subjected to ground motions at the Maximum Considered Earthquake (MCE) level (2% probability of exceedance in 50 years).

An innovative approach that deviated from the traditional design sequence was undertaken to meet the stringent scheduling, regulatory, and budgetary requirements for this project. There were three interrelated aspects to this approach. First, the structural design team started their construction document phase while the other design team disciplines were still in the design development phase. Second, a Concurrent Plan Review (CPR) process was conducted with the state's reviewing agency. The third component of this integrated strategy was the implementation of a master-planned prototype bearing testing program during the structural design document phase. This allowed the structural design team to design with prototype bearing stiffness and damping values and thus design a more efficient structural system. The combination of allowing the structural design team to start their work early, of working collaboratively with the state agency's CPR process, and of implementing an early prototype design, fabrication, and testing program provided a successful and cost-effective means to deliver a building with superior structural performance while adhering to a stringent schedule.

[1] Principal, Arup
[2] Assistant Professor, Tokyo Institute of Technology
[3] Principal, Bello & Associates, Structural Engineers
[4] Consulting Engineer, Bello & Associates, Structural Engineers
[5] Principal, Treadwell & Rollo
[6] Senior Engineer, Arup
[7] Associate Principal, Arup

Project Overview

The San Francisco General Hospital and Trauma Center (SFGH) project began in 2001, when the San Francisco Health Commission, the governing and policy-making body of the Department of Public Health, decided to replace the existing Main Hospital because of significant seismic deficiencies in the existing hospital. To meet the project schedule, the design team worked with the Owner (City and County of San Francisco), the Architect, the Office of Statewide Health Planning and Development (OSHPD), the Plan Checkers, the Peer Reviewers, and the Cost Planners to select the optimal structural system. Base isolation, using a pioneering bearing type, was chosen.

Building Description

The SFGH building is a seven-story steel-framed tower over a two-story steel-framed podium supported on 115 base isolation bearings above a reinforced concrete mat foundation (see Figures 1 and 2). The building is designed in conformance with the 2007 California Building Code (CBC). The program area is roughly 448,000 square feet and provides 284 acute care beds, diagnostics and treatment areas, and emergency care services. The tower's plan configuration is oblong, defined by two interlocking circular forms. The podium is generally rectangular with an extension at the northeast corner. The site slopes approximately 16 feet downward from east to west; this results in two levels below grade on the east side of the building and one level below grade on the west side. A one–story pedestrian bridge at level L2 and an underground tunnel at level B1 link the new hospital to the existing Main Hospital Building 5.

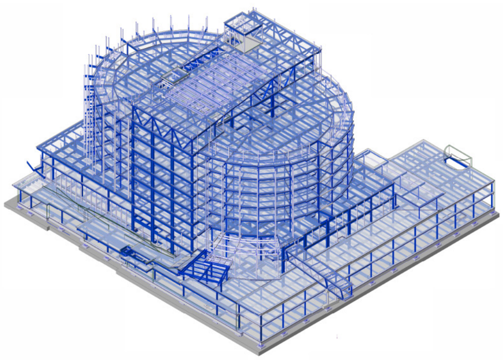

Figure 1. Rendering of SFGH

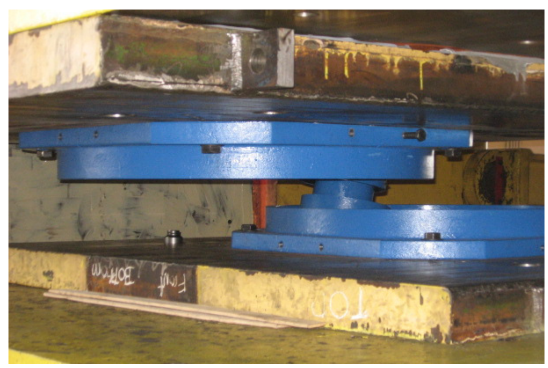

Figure 2. Fully displaced isolator

Structural Systems above the Isolators

The gravity load–resisting system includes concrete fill on metal deck floors and roof diaphragms supported on steel beams and girders. The steel beams and girders are composite structural elements with metal deck and concrete fill floors and roofs supported by steel wide flange and built-up cruciform steel columns. The lateral load–resisting system includes steel intermediate moment frames, deck and fill diaphragms, and reinforced concrete walls at the basement perimeter level B2. At the mechanical penthouse, located above the main roof level (Level L8), lateral loads are resisted by a concrete fill on metal deck roof diaphragm and by ordinary concentric steel braced frames.

Each bearing will be connected to a concrete pedestal with eight bolts passing through the bearing's bottom plate and threaded into eight 6" diameter shear lugs cast in the top of the concrete pedestal. Shims and high-strength non-shrink metallic grout between the bearing and top of the concrete pedestal will be used to level the bearings; cruciform shaped column base assemblies are designed to fit on top of the bearings. To facilitate constructability, beams with field-bolted moment connections will then be attached to the cruciform assemblies.

Structural System below the Isolators

The foundation system is a 4-foot-thick reinforced concrete mat with 148 hold-downs. The top of mat is 7 feet below Basement B2. The mat is locally sloped and depressed below the elevators to accommodate the elevator pits. The bottom of the mat foundation is approximately10 feet below the groundwater table.

Isolation System

Each building column is supported on a Triple Pendulum (TP) bearing, which in turn is supported on a 6-foot-square concrete pedestal above the mat foundation. The concrete pedestals allow maintenance access to each isolator. Two types of isolator bearings are used in the building, designated Type 1 (small) and Type 2 (large); each type is assigned based on column axial loads. There are 43 Type 1 and 72 Type 2 isolators with maximum rated lateral displacements of 33 inches and 32.6 inches, respectively. From the top of the mat foundation to the finished grades at levels B1 and L1, there is a 36-inch-wide moat around the entire perimeter of the building. Articulated moat covers bridge the moat and are typically attached to the building on one side and resting on sliding surfaces atop the perimeter retaining wall on the other side.

Four prototype bearings were tested during the design phase of the project—two each of Type 1 and Type 2 (see Figure 2). The prototype bearings were subjected to a range of vertical loads (including Pu, max from the time history analyses) and displacements for varying cycles at real-time (also termed full-speed) test rates. In addition to the real-time prototype tests, a second set of "slow-speed" tests were performed at peak cyclic test velocities of 1-inch/second on the same prototype bearings, in accordance with section 17.8 of American Society of Civil Engineers (ASCE) Standard 7-05. Based on the prototype test results, a bounding analysis was performed to develop bearing properties for use in the analysis and design of the isolated building.

Site-Specific Ground Motions

Beginning in 2007, the geotechnical engineer, Treadwell & Rollo, developed several different site-specific MCE response spectra over the course of the project, in response to changing code requirements and Peer Review comments. The first MCE spectrum was developed in 2007 (MCE 2007) in accordance with the International Building Code 2006, the governing code at that time. The second response spectrum (MCE 2008) was developed to reflect the requirements of Code Application Notice 2 (CAN2-1802A.6.2), issued by OSHPD in September 2008. Specifically, CAN2 requires that the site-specific spectra using Next

Generation Attenuation relationships consider the maximum rotated component of the ground motion and not the geometric mean. The third and final response spectrum (MCE 2009) was also developed to reflect CAN2 but incorporated OSHPD-accepted proposed Code changes to ASCE 7-05. At a period of 4 seconds, the MCE 2009 spectral accelerations are about 30% higher than those in MCE 2007.

Treadwell & Rollo also developed several sets of time histories (THs) corresponding to the MCE spectra over the course of the project. The first (original) set from 2007 consisted of seven recorded pairs of THs from mostly strike-slip events. CBC 2007 mandated that the square root of the sum of the squares (SRSS) of the average of the scaled THs in the period range of interest not be less than 10% of 1.3 times the target spectrum; this yielded a scaling factor of 1.17 (computed from 1.3×0.9). This criterion may be achieved by either spectrally matching each component of the suite to the target spectrum or by developing single scalar factors for each pair of THs. The first set of THs conformed with the CBC 2007 requirements and was spectrally matched to 1.17 times the MCE 2007 spectrum. After CAN2 was adopted by OSHPD, a second set using the same suite of seven THs was developed also using the spectral matching approach but to 1.17 times the MCE 2008 spectrum. The second set of THs yielded significantly higher responses because maximum bearing displacements were roughly 12% higher than those for the original set.

In March 2009, a third set of THs was developed that was spectrally scaled to 1.0 (not 1.17) times the MCE 2009 spectrum. This set represented a change from the first two sets in the number of THs in the suite (ten instead of seven), the selection of time history records used, and the development approach (single scalar versus spectrally matched). To avoid potential schedule delays that a re-review of the selected records might trigger, a fourth and final set of THs was developed in April 2009 using the original suite of seven TH records and the single scalar approach to 1.0 times the MCE 2009 spectrum. The final set resulted in a reduction of the average isolator displacements of 13–36% when compared with the average displacements from the original 2007 THs. See Figure 3 for a plot of the MCE 2009 spectra and the corresponding scaled THs used in the design.

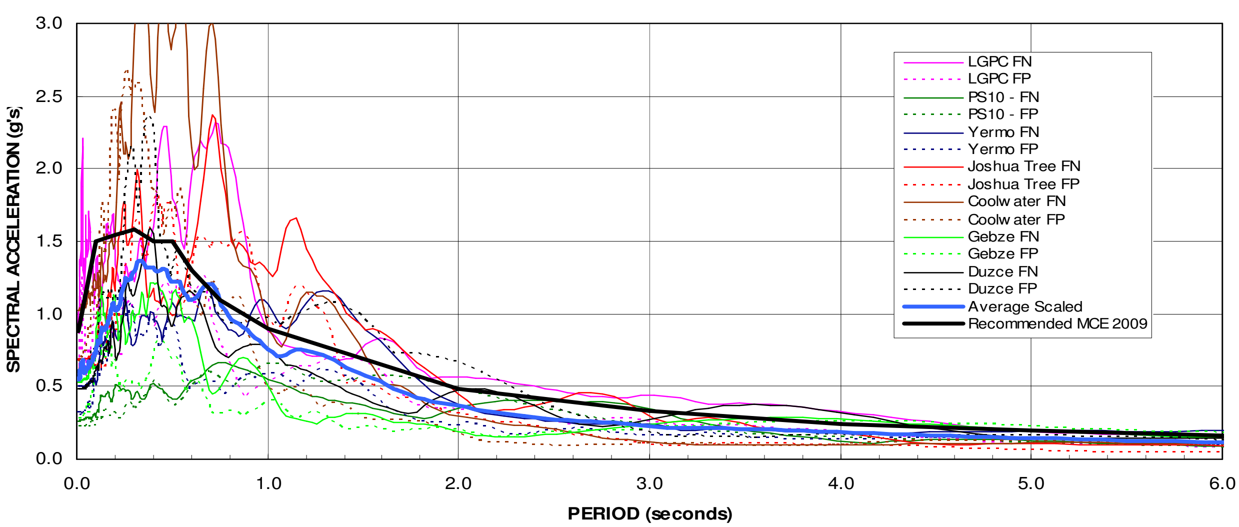

Figure 3. MCE Spectra and corresponding time histories

Seismic Design and Analysis

SFGH is designed to comply with the 2007 CBC, which references ASCE 7-05, and to comply with the requirements specified in the Seismic Design Criteria (developed by the project structural engineer). A brief summary of the seismic design parameters include Occupancy Category IV; Seismic Design Category D; Importance factor = 1.0 (for isolated buildings); Response Modification Factor R_I = 1.0. In addition to the site-specific ground motions, soil and site parameters include: Site Class C; S_{MS} = 1.547 g; S_{DS} = 1.031g; Type A fault > 10 km from the site. Two levels of seismic hazard were considered, MCE and DE (Design Earthquake = 2/3×MCE). Non-linear response-history analyses were performed using the analysis and design computer program ETABS Non-Linear.

Description of the Triple Pendulum Bearing

The Triple Pendulum (TP) bearing is a recently developed sliding isolator produced by Earthquake Protection Systems, Inc. (EPS), of Vallejo, CA. This bearing incorporates four concave surfaces and three independent pendulum mechanisms. A section through a typical TP bearing is shown in Figure 4. The properties of these three pendulum mechanisms can be selected to optimize the performance of the seismically isolated structure. Further details regarding the behavior of Triple Pendulum bearings have been described by Morgan [2007] and Fenz and Constantinou [2008].

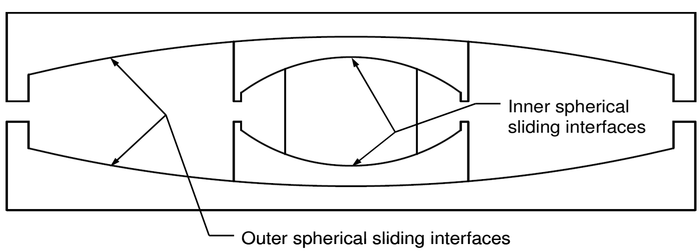

Figure 4: Section through typical Triple Pendulum bearing

Bearing Properties Used in the Analyses

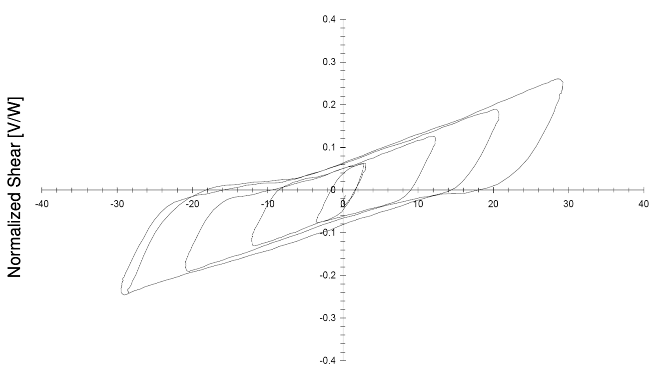

Figure 5: Sample hysteresis from full-speed prototype test program

For the SFGH project, both sizes of bearings, Small (Type 1) and Large (Type 2), have equal friction coefficients on the outer spherical sliding interfaces, and thus exhibit essentially bilinear hysteretic behavior. The force–displacement behavior of a typical bearing from one of the SFGH full-speed prototype tests is shown in Figure 5. The hysteresis shown represents a cyclic test conducted at several amplitudes of displacement and at a velocity corresponding to the expected velocity under actual earthquake excitation.

The Prototype Test Report (EPS 10/23/08) gives the following tolerances for acceptance of Quality Control (QC) tests for each isolation unit. Note that stiffness parameters are normalized by the supported weight.

1. Small Bearing: for Test QC1
Avg 3-cycle k_{eff} = 0.00997±0.00150 (k/in/k)
Avg 3-cycle β_{eff} = 0.266±0.053 (% critical)

2. Large Bearing: for Test QC2
Avg 3-cycle k_{eff} = 0.01017±0.00156 (k/in/k)
Avg 3-cycle β_{eff} = 0.252±0.052 (% critical)

These tolerances represent a ±15% allowable variation for effective stiffness, and a ±20% allowable variation for equivalent damping. However, for determining system-level bounding properties, a tighter variation may be assumed because of decreased variability of random systems. For simplicity, this system variation is assumed to be ±10% for both stiffness and damping. Using this allowable variability, Table 1 summarizes the upper- and lower-bound effective stiffness (normalized by supported weight) and damping properties for each bearing type:

Table 1: Bounding properties for each isolator type

	Upper Bound		Lower Bound	
Isolator	k_{eff}	β_{eff}	k_{eff}	β_{eff}
Small	0.01097 k/in/k	29.3%	0.00897 k/in/k	23.9%
Large	0.01118 k/in/k	27.7%	0.00914 k/in/k	22.7%

Based on the system variation tolerances, properties were developed to satisfy the bounding requirements for ETABS NLLINK Isolator2 elements. For upper-bound friction, the property modification factors for aging, λ_a = 1.1, and contamination, λ_c = 1.05, were applied in addition to the increased friction from natural variation. Upper bound properties yield maximum moment frame forces and drifts. Lower bound properties yield maximum bearing displacements. Analytical properties of isolators used in the ETABS analyses are shown in Table 2 (where P_{ave} = average dead load on each bearing, K = initial stiffness, μ_{slow} = friction on inner slider, μ_{fast} = friction on outer slider, α = parameter defining the transition from slow to fast friction, and Radius = effective radius of the outer pendulum mechanism):

Table 2: ETABS upper- and lower-bound isolator properties

	Upper Bound		Lower Bound	
Isolator	Small	Large	Small	Large
P_{ave}	400k	1000k	400k	1000k
K	27.4 k/in	54.3k/in	27.4 k/in	54.3k/in
μ_{slow}	0.047	0.051	0.028	0.026
μ_{fast}	0.094	0.102	0.055	0.052
α	1.28	1.28	1.28	1.28
Radius	167 in	163 in	167 in	163 in

ETABS does not have an explicit element for modeling a TP bearing that accounts for all stages of sliding. However, since the SFGH bearings have equal friction coefficients on the outer spherical concave surfaces, they only incorporate two stages of sliding, thus, ETABS

Isolator2 NLLINK element with effective bilinear properties is valid in representing the TP behavior. The effective bilinear properties were developed such that the total energy dissipated per cycle and the effective stiffness match that of the actual bearing (as determined from prototype test results). A study by Morgan [2009] comparing TP modeling and calibrated bilinear modeling shows that the bilinear modeling yields slightly more conservative estimates of superstructure response parameters, such as floor acceleration and interstory drifts, and slightly lower (by about 2%) estimates of bearing displacements.

Modeling Assumptions and Non-linear Response-History Analyses

All lateral and gravity superstructure framing members were modeled as elastic elements in the analysis models because they should remain within the linear range of response (i.e., not yield). Diaphragms were modeled as semi-rigid. Foundations below the bearings were assumed as fixed against translation but free to rotate. The TP bearings were modeled as NLLINK elements located at the bottom of the lowest level of framing. Panel zone deformations at the moment frame joints were computed internally by ETABS. Dead loads, distributed throughout the structure as area and frame loads, were applied as a ramp time history case from which the seismic time-history cases were started. Vertical seismic loads equal to $0.2S_{DS}\times$Dead were applied statically. To account for accidental torsion, the seismic mass at each level was shifted by adding "accidental torsion mass" (area load) over roughly half the floor area and subtracting "accidental torsion mass" over the remaining half such that the net "accidental torsion mass" applied is zero; the magnitudes of these "accidental torsion masses" were computed as required to shift the center of mass by 5% of the larger building dimension (typically the east-west dimension) at that level. The "accidental torsion masses" were assigned using ETABS Mass Source and were decoupled from the gravity loads on the bearings. Thus, three models for each mass configuration—unshifted, mass shifted west, and mass shifted east—were run for both MCE and DE, for both upper and lower bound bearing properties (12 TH models). Additional models were created from these to evaluate varying parameters (e.g., fixed base; cracked versus uncracked basement shear walls).

Non-linear response-history analyses for both MCE and DE were performed for the suite of seven time-histories. For each ground motion, response parameters (e.g., member forces, drifts, accelerations and bearing displacements) were determined from the maximum of two averages: (a) the average of seven ground motions with FN (fault normal) at 0 deg and FP (fault parallel) at 90 deg; and (b) the average of seven ground motions with FN at 90 deg and FP at 0 deg. The number of Ritz vector starting modes specified was 500 with 2% modal damping for the first three modes (governed by the isolator deformations) and 5% damping for all other modes; a study was performed using 2% damping (instead of 5%) for the superstructure's fundamental modes and was found to yield insignificant differences.

Building Behavior and Design Checks

Building Periods and Base Shears

The fundamental building periods for the isolated building are $T_x = T_{EW} = 3.72$ seconds and $T_y = T_{NS} = 3.66$ seconds. For comparison, the periods from the fixed base model are Tx = 1.91 seconds and Ty = 1.85 seconds. The base shears from the TH analyses (unshifted mass) are as follows: V,MCE,EW = 0.171W, V,MCE,NS = 0.164W, V,DE,EW =0.135W, and V,DE,NS = 0.128W.

Per ASCE 7-05 sections 17.5.4.2 and 17.6.4.2, the structure above the isolation system was designed to withstand a minimum base shear of 80%Vs (the structure is irregular in plan at the lower levels) where Vs = (kd,max)×D_D/R_I×W. Due to differing interpretations of the current Code provisions, the minimum base shear used in the design of SFGH was conservatively computed using the highest value of kd,max from the prototype tests along with the corresponding test displacement D_D at that value, so 80%Vs = 0.8×(kd,max = 0.116)×(D_D =14.0")/(Ri = 1.0) = 0.130W. Thus, the V,DE,NS base shear was scaled up to the minimum base shear by 0.13/0.128 = 1.02.

Bearing Displacements

The maximum bearing displacements were computed at each time step of each time history for the X and Y directions and SRSS of X and Y; the maximum of these for each of the THs were then averaged. This average value was multiplied by 1.1 for torsion (per ASCE 7-05 section 17.5.3.5) and by 1.05 (an adjustment for the bilinear modeling versus TP modeling) to yield an MCE maximum displacement of 26.29" which is well within the bearing's displacement capacity of 32.6". Similar computations yield a maximum DE displacement of $D_{TH,DE}$ = (12.04" × 1.1 × 1.05) = 13.91" which is greater than the Code-required minimum displacement of 1.1 × D_{TD} = 11.67", where D_{TD} is computed from Code equation 17.5-1. The maximum uplift at the bearings from MCE analyses is 0.715"; this is much less than the 1.5" that the bearing can accommodate before the ring clears the slider at the displacement capacity of the bearing.

Structural Elements above the Isolators

The moment frames were sized for stiffness rather than strength; thus, the moment frame members were found to behave elastically for DE loads (average beam Demand Capacity Ratio (DCR) = 0.54, average column DCR = 0.50) when checked per the factored load combinations of ASCE 7-05 section 12.4. Similarly, for MCE loads, with ϕ = 1.0, the members were essentially elastic (average beam DCR = 0.66, average column DCR = 0.58). Panel zones were analyzed and doubler plates were not required. The column/beam joint just above the bearings were also designed for the additional P-delta forces that result from the maximum column load applied at an offset equal to half of the maximum bearing displacement. As required per section 17.2.4.8, the beams at the lowest (B2) level framing into the column at each bearing were also designed for jacking loads associated with removing the supported building loads from a bearing (in the event that the bearing needs to be removed and replaced).

The interstory drifts were computed at each time step of each time history at the outermost points of the building; the maxima of each suite of seven THs were then averaged. The average drifts are 0.91% and 1.11% for DE and MCE, respectively. The maximum DE and MCE drifts are 1.20% and 1.44%, respectively, both less than the Code limits of 1.5% for DE and 2.0% for MCE per ASCE 7-05 section 17.6.4.4. Torsional irregularity was checked using the MCE response spectrum drifts and displacements. Other structural irregularities and design checks (strong column/weak beam, bracing of MF beams, etc.) and design of other structural components (horizontal diaphragms, penthouse, basement shear walls, etc.) were similar to those performed for a non-isolated building and found to be within code limits.

Elements below the Isolators and Non-Structural Elements

Per ASCE 7-05 sections 17.5.4.1 and 17.6.4.1, the structural elements below the isolation system (e.g., the concrete pedestals and mat foundation) were designed to withstand a minimum base shear of 90%Vs = 0.9×(kd,max = 0.116)×(D_D =14.0”) = 0.146W. The axial and shear loads applied to the mat foundation were taken directly from the TH model reactions resulting from loads applied at the center of mass as ramp time history cases (to enable the NLLINK elements to uplift). The magnitude and distribution of applied loads was based on the first mode shape of the superstructure in each direction, with the total loads summing up to the minimum base shear of 90%Vs. Orthogonal combinations of +/-100% and +/-30% were used and this resulted in 16 load combinations. The design of the mat foundation with 148 hold-downs was validated using the computer program SAFE and included all loads from the superstructure as well as mat self-weight and buoyancy loads due to ground water.

Benefits of Base Isolation

In the case of SFGH, the base-isolation design produced three significant benefits as compared with a fixed-base design: the hospital will be cheaper and easier to build, interior volume is increased so the layout will be less constrained and coordination of utilities will be easier, and the building will perform better in a major earthquake so its contents will be better protected.

A study was performed in which the isolators were removed from the SFGH model, and member sizes were revised to meet the Code’s 1% limit for allowable drift for a fixed-base steel moment frame hospital building. The resulting steel tonnage was roughly 3000 tons more than for the isolated building model. This savings in steel is significantly greater than the costs of the isolators, moat covers, and flexible utilities connections.

The fixed-base building would require deeper columns and beams. Bigger columns would constrain space programming. Deeper beams would limit the space available for utilities, thus making coordination more difficult. A fixed-base design would also have to meet the requirements of special moment frames including member size limitations, connections (e.g., reduced beam sections) and details (e.g., more bottom flange braces). The floor accelerations for the fixed-base building would be much higher than those for the isolated building, and this would result in significantly greater anchorage and bracing of equipment and non-structural elements.

In a major event, the moment connections from beam to column in the fixed-base building would yield; replacing them would be difficult, expensive and would result in significant downtime. For the same earthquake, the members in the base-isolated building would not yield and the bearings would not exceed their capacity. In fact, the bearings would allow the building to re-center itself after the earthquake, whereas the fixed-base building might exhibit permanent residual drifts (i.e., be out of vertical alignment).

Conclusions

Base isolation can provide the best insurance for mitigating seismic hazards for critical facilities, with lower costs, and without negatively impact the design schedule. The integration of upfront parametric framing system studies and prototype bearing testing with the design and innovative review processes (CPR—Concurrent Plan Review) avoided schedule delays, mitigated traditional risks, and minimized conservative and costly assumptions. The San Francisco General Hospital and Trauma Center Re-build Team was a successful collaboration because everyone—the owner, the architects, the contractor, the isolator provider, the structural designers, and the review teams—all worked together toward common goals that are crucial for the hospital to carry out its mission. The results are stunning—a new base-isolated hospital that will be cheaper to build and that will provide better protection for occupants and contents than a comparable conventional fixed-base hospital.

Acknowledgements

The design team is thankful for the owner's arranging to make prototype testing a reality. The team is also grateful for the wisdom and guidance provided by Dr. Mohammad Karim at OSHPD in establishing the Seismic Design and Testing Criteria. The generosity and commitment of resources by the isolator provider, EPS, throughout the design and review process, are highly treasured. Timely input and support from the Peer Review Team, Badri Prasad at Thornton Tomasetti, Prof. Michael Constantinou of The State University of New York, Buffalo, and Shahriar Vahdani at Fugro, are greatly appreciated. The commitment to the early parametric studies of framing systems by the project architect, FCA, proved invaluable. The CPR process would not have been successful without the dedication from the third party review team led by Dr. Balram Gupta at TMAD. The tireless effort by our technical editor, Dave Wilton, is also acknowledged.

References

ASCE 7-05—American Society of Civil Engineers Standard, "Minimum Design Loads for Buildings and Other Structures", 2005.

Fenz, D.M., and Constantinou, M.C. (2008). "Spherical sliding isolation bearings with adaptive behavior: Theory" *Earthquake Engineering Structural Dynamics*, **37**(2), 163-205.

Morgan, T.A. (2007). "The Use of Innovative Isolation Systems to Achieve Complex Seismic Performance Objectives," Doctoral Dissertation, University of California, Berkeley, USA.

Morgan, T.A. (2009). "Comparison of TP Model and Calibrated Bilinear Model for SFGH Project," January 7, 2009.