R-8

CITY OF LONG BEACH

DEPARTMENT OF PUBLIC WORKS
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February 12, 2013

HONORABLE MAYOR AND CITY COUNCIL
City of Long Beach
California

RECOMMENDATION:

Authorize the City Manager to proceed with the entitlement phase of the proposed Long-Term Belmont Pool Revitalization Project (Project), and with the installation of a temporary outdoor pool or other interim option. (District 3)

DISCUSSION

On January 13, 2013, the City's Belmont Pool Natatorium (Belmont Pool) was temporarily closed due to a Structural and Seismic Evaluation (Evaluation) that concluded the facility (Exhibit A: Location Map) was seismically unsafe in the event of a moderate earthquake. Ensuring life-safety was a key objective for conducting the Evaluation. The Evaluation was performed by Paul KT Yeh, Structural Engineer, of TMAD Taylor Gaines (Exhibit B). The City's Building Official has reviewed the Evaluation, inspected the facility, and determined that the Belmont Pool, in its current condition, is a substandard building that is seismically and structurally unsafe. The City Engineer also reviewed the Evaluation and concurs with the Building Official's determination. The Belmont Pool was constructed in the late 1960's and met all applicable building codes at the time. However, based in part on current building codes, the Belmont Pool is now in need of either a major seismic retrofit and other upgrades, or a complete reconstruction. For over 40 years, the Belmont Pool has fulfilled a critical role in the City's development, providing young children, adolescents, adults and seniors with diverse recreational and competitive swimming opportunities.

Recommended Long-Term Project

In 2008, the Department of Parks, Recreation and Marine hired a consultant to help develop a cost estimate to retrofit and upgrade the existing Belmont Pool and conduct community outreach. The cost for such retrofit and upgrade was estimated at $44 million for construction alone; however, a report was never finalized because of budget constraints. In February 2012, the City Council appropriated funding to reinitiate the planning process for the Project. Since then, staff has carefully considered historical information, held discussions with staff from the California Coastal Commission, reassessed the facility's condition, held several focused meetings with recreational and competitive users, and is now recommending approval to proceed with the entitlement and environmental review of the proposed Project (Exhibit C).

Staff from the California Coastal Commission has notified the City that any replacement facility must provide broad based recreational opportunities, and consequently, expressed their reluctance to recommend approval of a competitively-focused aquatics facility. As such, the proposed Project carefully balances broad based recreation and specialized competitive opportunities in an indoor Natatorium in the approximate location of the existing Belmont Pool, and a new outdoor pool immediately north of the existing facility. The proposed Project is intended to host all competitive swim and water polo events that are currently hosted (local/regional/national), while providing added water space to enhance the experience of all users and potentially attracting additional events. Examples of local, regional, and national (NCAA) swim and water polo competitions that are intended to be accommodated by the proposed Project include: California Interscholastic Federation, Pac-12, Mountain Pacific Sports Federation Conference, and Southern California Intercollegiate Athletic Conference. While the existing Belmont Pool does not meet international standards for major events, the proposed Project will meet such standards for water polo and 50-meter, 25-meter and 25-yard swimming.

The preliminary cost estimate for the proposed Project is $54 million, inclusive of construction and soft costs, including recreational diving boards. Additional water space to accommodate taller competitive diving platforms (5, 7.5 and 10 meters) would increase the cost by $8.1 million. If City Council supports the proposed Project, the next step is to initiate the proposed Project's entitlement phase, which includes California Environmental Quality Act (CEQA) clearance and any required local discretionary approvals such as Site Plan Review.

Interim Options to Provide Pool Facilities

In light of the results of the TMAD Taylor Gaines Evaluation, the following interim options were considered to help address the closure of the Belmont Pool:

1. Install a temporary outdoor pool in the adjacent parking lot;
2. Conduct selective demolition of the Natatorium to remove the existing roof and strengthen the support columns, and potentially install a new roof;
3. Conduct an emergency seismic retrofit of the existing columns using a fiber wrap method; or
4. Accommodate existing user groups at other City and local pools, if feasible.

Staff recommends Option 1 (Exhibit D) because it is the most cost-effective and flexible option that will provide needed pool space during Project entitlement. Option 1 would cost approximately $4.2 million, and take 5 to 8 months for approval by the California Coastal Commission as well as construction. As a reference, the City previously installed a temporary pool in downtown Long Beach during the 2004 Olympic swim trials. In this option, the pool could be reused as part of the permanent design.

Option 2 would involve surgical demolition that is difficult, time-consuming, and would cost between $4.2 million and $5.5 million, while taking 6 to 9 months to complete. Option 2 would also require that the existing pool facilities be protected. If Option 2 is preferred, it is highly recommended that the support columns be strengthened and a horizontal truss system be installed without a roof deck (open ceiling), or install a new lightweight steel construction roof deck instead. As part of this option, a complete conversion to an outdoor pool was considered, but is not recommended by the project's Structural Engineer, due to: a) regular strong winds that carry beach sand, which may interfere with pool components (e.g. filtration and electrical systems) that were not designed for such conditions; b) another wall system will be necessary to protect against winds and beach sand, thus increasing costs; c) additional engineering will be necessary to ensure that the Special Events and Office/Locker Room Building will not be negatively affected; and, d) the improvements would be temporary until a new facility is constructed.

Option 3 involves strengthening the support columns using a fiber wrap method, costing approximately $3 million and taking 3 to 5 months to complete. While the least costly, this option will not provide any interim accommodations should the proposed Project proceed to construction, and will result in the lack of pool facilities for 9 to 12 months once construction commences.

Option 4 would have minimal cost but also limits the accommodations of existing activity. Not all recreation and competitive uses could be relocated to existing pools.

This matter was reviewed by Deputy City Attorney Linda Vu on February 1, 2013 and by Budget Management Officer Victoria Bell on February 1, 2013.

TIMING CONSIDERATIONS

City Council approval of this item is requested on February 12, 2013 to allow staff to expeditiously proceed with the next steps of this important project.

FISCAL IMPACT

The preliminary, estimated cost for the long-term Project is at least $50 million, and funding will be requested in phases as the Project moves forward. The additional, estimated cost for the Option 1 recommendation of a temporary outdoor pool is $4.2 million, and is budgeted for FY 13 in the Tidelands Operations Fund (TF 401) in the City Manager Department (CM).

SUGGESTED ACTION:

Approve recommendation.

Respectfully submitted,

Respectfully submitted,

MICHAEL P. CONWAY
DIRECTOR OF PUBLIC WORKS

GEORGE CHAPJIAN
DIRECTOR OF PARKS, RECREATION AND MARINE

ATTACHMENTS

MPC:GC:EOL

APPROVED:

PATRICK H. WEST
CITY MANAGER

EXHIBIT A

Belmont Plaza Pool

Location Map

EXHIBIT B

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Inland Empire

Los Angeles

Pasadena

San Diego

San Francisco

Thousand Oaks

COLORADO

Lone Tree

TEXAS

Austin

Dallas

San Antonio

WASHINGTON

Bellevue

SEISMIC EVALUATION FOR

COLLAPSE PROBABILITY ASSESSMENT

FOR

Belmont Plaza Pool Buildings

CITY OF LONG BEACH

LONG BEACH, CALIFORNIA

Paul KT YEH, S.E.
Structural Engineer S2872
TMAD TAYLOR & GAINES
February 4, 2013
TTG # 0112041.00

TMAD TAYLOR & GAINES

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EXECUTIVE SUMMARY

In late 2008, the City of Long Beach began assessing the existing conditions of the Belmont Plaza Pool Buildings and developing preliminary cost estimates to rehabilitate or replace the Buildings. Unfortunately, those efforts were postponed because of funding constraints. The City recently reinitiated its effort to develop a long-term plan for Belmont Plaza Pool and hired TTG to evaluate its structural conditions. Based on the City's prior assessments of Belmont Pool and their desire to evaluate whether the buildings would be safe for use by the public while plans for a rehabilitation of new aquatics facility have been developed, TTG recommended a collapse probability assessment based on an Office of Statewide Health Planning & Development (OSHPD) Hazards U.S. (HAZUS) evaluation for the Belmont Plaza Pool Buildings at 4000 East Olympic Plaza in Long Beach, California. A collapse probability assessment for the existing buildings will help to determine their current and long-term operational status.

OSHPD developed a version of the HAZUS method to calculate the probability of a structural collapse of an individual building in the event of an earthquake. Developed by FEMA and the National Institute of Building Science, HAZUS is a methodology that evaluates potential structural damage, death, and economic loss due to natural disasters like earthquakes, hurricanes, or floods. While only California hospital buildings are required by law to be assessed using HAZUS, it currently is the most advanced method for assessing any individual building's collapse probability during an earthquake. Where other methods for evaluating a building's seismic performance, specifically the old Tier 1 evaluations per FEMA 310 (now known as ASCE 31-03), may only reveal potential structural deficiencies of the building during an earthquake, HAZUS produces solid quantitative results illustrating the relative severity of structural deficiencies. Therefore, the HAZUS method is the most appropriate one to use for evaluation of the Belmont Plaza Pool Buildings at this stage.

The goal of this evaluation is to utilize OSHPD's version of HAZUS to calculate the probability of a structural collapse in the three existing Belmont Plaza buildings. Currently, buildings with an OSHPD HAZUS collapse probability threshold of 1.2% or less are considered not at risk of complete collapse, but may not be reparable or functional following a Design Earthquake A (moderate earthquake with a magnitude of about 5.0, having a 10% probability to be exceeded in 5 years).

Both the Belmont Private Event Building and Locker Building have an acceptable collapse probability of less than 1.2%. However, the Natatorium Building, in its current condition, has a 1.5% probability of collapse, which exceeds OSHPD's HAZUS collapse threshold. This building poses a significant risk of collapse in the event of a Design Earthquake A.

Per the HAZUS evaluation, TTG recommends that the Natatorium concrete columns be strengthened to reduce the risk of building collapse in the event of a Design Earthquake A, allowing the Belmont Plaza Pool Complex to remain operational through the next 5 years. This, however, is a limited structural retrofit which can only achieve a short-term collapse prevention objective. Long-term collapse prevention of the site will either be achieved through a comprehensive structural upgrade of the existing buildings to mitigate earthquake risk, or the removal of the buildings from service.

1. INTRODUCTION

1.1 KEY TERM DEFINITIONS

Building Type

Building structural systems are to be classified as to their Model Building Type per the Table below. For buildings with multiple types, all types will be listed. The building type resulting in the maximum collapse probability will be utilized to determine collapse probability.

Model Building Type (MBT)	Description
W1	Wood, Light Frame (≤5,000 sq. ft.)
W2	Wood, Greater than 5,000 sq. ft.
S1	Steel Moment Frame
S2	Steel Braced Frame
S3	Steel Light Frame
S4	Steel Frame with Cast-In-Place Concrete Shear Walls
S5	Steel Frame with Un-reinforced Masonry Infill Walls
C1	Concrete Moment Frame
C2	Concrete Shear Walls
C3	Concrete Frame with Un-Reinforced Masonry Infill Walls
PC1	Pre-cast Concrete Tilt-Up Walls
PC2	Precast Concrete Frames with Concrete Shear Walls
RM1	Reinforced-masonry Bearing Walls with Wood or Metal Deck Diaphragms
RM2	Reinforced-masonry Bearing Walls with Concrete Diaphragms
URM	Unreinforced masonry Bearing Walls
MH	Manufactured Housing

Design Earthquake A

The site-specific response spectra of a moderate earthquake with magnitude of about 5~5.9, having a 10% probability of being exceeded in 5 years, selected for a three-dimensional linear dynamic analysis (refer to *Site Specific Response Spectra* in this section for further explanation).

Design Earthquake B

The site-specific response spectra of a strong earthquake with magnitude of about 6~6.9, having a 10% probability of being exceeded by an earthquake over the next 50 years, typically used for new building designs (refer to *Site Specific Response Spectra* in this section for further explanation).

Dynamic Analysis

Analysis of structures subjected to dynamic loads (earthquake, wind) involves consideration of time-dependent forces. The resistance (responses) to displacement exhibited by a structure may include forces which are functions of the displacement and inertial forces.

Earthquake Classification

Class	Magnitude
Great	8 or more
Major	7 - 7.9
Strong	6 - 6.9
Moderate	5 - 5.9
Light	4 - 4.9
Minor	3 - 3.9

HAZUS
Developed by FEMA and the and National Institute of Building Science, HAZUS stands for "Hazards U.S.", and is now considered a standardized, nationally applicable loss estimation methodology used to evaluate potential structural damage, death, and economic loss due to natural disasters like earthquakes, hurricanes, or floods.

HAZUS as adopted by the Office of Statewide Health Planning & Development (OSHPD)
California Building Standards Commission approved the implementation of HAZUS on November 14, 2007 to reexamine the seismic risk Structural Performance Category 1 (SPC-1) hospital buildings (refer to *Structural Performance Categories* in this section for further explanation). These buildings are considered at risk of collapse in the event of an earthquake or other natural disaster. This reassessment allows OSHPD to reprioritize SPC-1 hospital buildings based on their level of seismic risk, and if they meet specified criteria, they can be reclassified to the SPC-2 category. If reclassified, these buildings move from a 2008/2013 seismic retrofit/replacement deadline to a 2030 deadline. Per OSHPD's estimate, 50% to 60% of the 1100 SPC-1 hospital building retrofits and replacements qualified for the reclassification under the new HAZUS methodology. The goal of OSHPD's version of HAZUS is to calculate the probability of collapse of an individual hospital building in an earthquake. For this project, TTG is applying the methodology of OSHPD's HAZUS for evaluation of the collapse probability of the buildings in Belmont Plaza Pool complex.
More information about the OSHPD HAZUS can be found at http://www.oshpd.ca.gov.

Lateral Force Resisting System (LFRS)
A part of the structural system designed to resist the Design Seismic/Wind Forces.

Liquefaction
The process by which saturated, unconsolidated sediments are transformed into a substance that acts like a liquid.

Peak Ground Acceleration (PGA)

Peak ground acceleration is a measure of earthquake acceleration on the ground surface, meaning it is a measure of how hard the earth shakes in a given geographic area (otherwise known as the earthquake's intensity). In an earthquake, damage to buildings and infrastructure is related more closely to ground motion rather than the magnitude of the earthquake. For moderate earthquakes, PGA is the more accurate determinate of damage. In major earthquakes, damage is more often correlated with peak ground velocity.

P[COL]

The probability of collapse is equal to the multiplication of HAZUS collapse factor and the probability of complete structural damage. The Office of Statewide Health Planning & Development (OSHPD) determines that a hospital building is at high risk of collapse when its probability of collapse is greater than 1.2%.

Seismic Base Shear Coefficient

The coefficient to calculate the total design seismic force or shear at the base of a structure. The design seismic shear is equal to the multiplication of this coefficient and the building effective seismic weight.

Significant Structural Deficiencies (SSD)

An attribute of the structure considered to be significant with respect to the probability of collapse. Following is a list of the significant structural deficiencies to be considered in the HAZUS analyses.

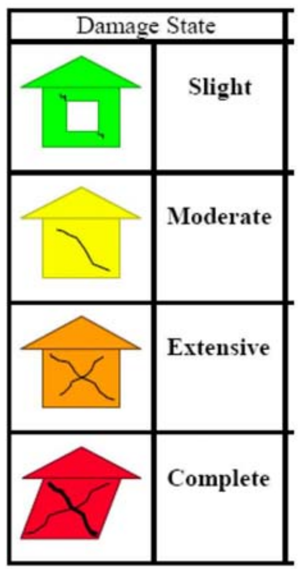
1. Age – Pre 1993
2. Material Testing
3. Load Path
4. Mass Irregularity
5. Vertical Discontinuity
6. Short Captive Column
7. Material Deterioration
8. Weak Columns
9. Wall Anchorage
10. Redundancy
11. Weak Story Irregularity
12. Soft Story Irregularity
13. Torsional Irregularity
14. Deflection Incompatibility
15. Cripple Walls
16. Openings (in diaphragm) at Shear Walls
17. Topping Slab Missing or Building Type is of Lift Slab Construction

Site Specific Response Spectra

A response spectrum is simply a plot of the peak or steady-state response (displacement, velocity, or acceleration) of a series of oscillators of varying natural frequency that are forced into motion by the same base vibration or shock. The resulting plot can then be used to assess the peak response of buildings to earthquakes. A site-specific elastic design response spectrum based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site. The Geotechnical consultant, Marshall Lew of AMEC, integrated the effects of all the earthquakes of different sizes, occurring at different sources at different probabilities of occurrence, to provide an estimate of probability of exceeding different levels of ground motion at a site during a specified period of time. Based on the estimated remaining life of 5 to 10 years for the facility, site-specific response spectra were developed for ground motions having a 10% probability of being exceeded in 5 years.

Structural Damage States

The extent and severity of structural damage to a building is described by one of the four damage states.



Structural Performance Categories (SPC)

This is the structural performance of a building in relation to the structural provisions of the Alquist Hospital Facilities Seismic Safety Act.

SPC	DESCRIPTION
SPC1	Buildings posing a significant risk of collapse and a danger to the public. Buildings with a Probability of Collapse greater than 1.20% shall be placed in this category.
SPC2	Buildings in compliance with the pre-1973 *California Building Standards Code* or other applicable standards, but are not in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act. These buildings do not significantly jeopardize life, but may not be repairable or functional following strong ground motion. Buildings with a Probability of Collapse less than or equal to 1.20% shall be placed in this category.
SPC3	Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, utilizing steel moment-resisting frames in regions of high seismicity and constructed under a permit issued prior to October 25, 1994. These buildings may experience structural damage which does not significantly jeopardize life, but may not be repairable or functional following strong ground motion.
SPC 4	Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, may not experience structural damage which may inhibit the ability to provide services to the public following strong ground motion.
SPC5	Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, and is reasonably capable of providing services to the public following strong ground motion.

1.2 BUILDING DESCRIPTION

The Belmont Plaza Pool Buildings, constructed circa 1968, are likely designed based on the requirements of the 1964 edition of the Uniform Building Code. From the information provided by the City including original blueprints, these three buildings were built to the "codes of the time". The existing building complex is composed of three separate structures: the main one-story Natatorium Building, the two-story Private Event Building, and the one-story Locker Room Building. The Natatorium Building is situated in the middle of the site, and it is flanked by the Private Event Building to the west, and the Locker Building to the east (see Appendix A, Figure 1 to 15). These three buildings are separated by 2" wide joints. The two-story Private Event Building shares a partial basement with the Natatorium Building. The one-story Natatorium Building is built on a 4'-0" high podium, and the one-story Locker Building raises 4'-0" above the adjacent grade to share the same ground floor elevation as the Natatorium. All three structures are concrete frame buildings with a mixture of cast-in-place concrete and pre-cast concrete construction.

The Natatorium Building has an approximate footprint of 224'x148'. The Natatorium is a very tall, one-story, sloped roof structure with a height that varies between 48' and 50' above the floor deck. The structure's Lateral Force Resisting System consists of a shear wall-frame interactive system with full-height, cast-in-place concrete columns supporting discontinuous 25'-0" high precast concrete shear walls above 23'-0" high glass curtain wall (see Appendix A, Figure 1, 5, 9 and 10).

The Private Event Building has an approximate footprint of 127'x78'. The Lateral Force Resisting System of the Private Event Facility consists of perimeter precast concrete shear walls above the 2nd floor, and cast-in-place concrete walls below. The east side of the 2nd floor is mostly open to the Natatorium with only two 10' wide precast concrete panels located 25' from the east end. All of the above mentioned walls are not continuous to the foundation, and are supported by cast-in-place overhang concrete beams on the 2nd floor. The 1st floor concrete shear walls are set back 26'-0" and 11'-0" from the 2nd floor on the west side and the north/south direction, respectively. The north exterior ramp and south exterior stair provide access to the 2nd floor (see Appendix A, Figure 2, 3, 6 and 7).

The Locker Building has an approximate footprint of 148'x85'. It is a one-story building similar in construction to the Private Event Building. Its lateral force-resisting system consists of cast-in-place and precast concrete shear walls (see Figure 4, 5 and 9). For its relatively small size, the Locker Building is composed of a relatively large number of shear walls.

The main characteristics of the buildings are summarized below.

The Private Event Building is constructed as follows:

Roof:	Precast concrete double –T slab with 2.5" thick cast-in-place concrete topping, which is supported by concrete beams and columns.
Floor:	4.5" thick cast-in-place concrete slabs supported by concrete joists, beams, and columns.
Foundation:	Timber piles with concrete pile caps supporting columns and walls.
Lateral System:	Concrete diaphragm with concrete shear walls for the both the East-West direction and the North-South direction.
Ceiling:	Finished ceiling
Floor-to-Floor Heights:	Basement Level: 9'-0" 1st Level: 15'-0"; 2nd Level: 16'-0"

The Natatorium Building is constructed as follows:

Roof:	4.5" thick cast-in-place roof concrete slabs supported by precast concrete beams, prestressed concrete girders and cast-in-place concrete columns.
Foundation:	Timber piles with concrete pile caps supporting columns and walls.
Lateral System:	Concrete roof diaphragm supported by a shear wall-frame interactive system with full-height, cast-in-place concrete columns, which support discontinuous precast concrete shear walls above 23'-0" high glass curtain wall in both the East-West direction and the North-South direction.
Ceiling:	Exposed Concrete Soffit
Floor-to-Roof Height:	48'-0' ~ 50'-0"

The Locker Building is constructed as follows:

Roof:	4.5" thick cast-in-place roof concrete slabs supported by concrete joists and beams.
Foundation:	Timber piles with concrete pile caps supporting columns and walls.
Lateral System:	Concrete roof diaphragm supported by precast concrete frames with shear walls infill in both the East-West directions, and the North-South directions.
Ceiling:	Finished ceiling

Floor-to-Roof Height: 11'-6"

1.3 DOCUMENTS REVIEWED

1. Review of the original structural drawings of the Belmont Plaza Beach Center by Bole & Wilson Structural Engineers, dated 14 February 1967.
2. Review of the original architectural drawings of the Belmont Plaza Beach Center by Heusel, Homolka & Associates, Dated 14 February 1967.
3. Review of the Report of Geotechnical Consultation, prepared by AMEC, dated October 10, 2012.

1.4 SCOPE OF WORK

1. Perform a short-term collapse probability assessment using the Office of State Health Planning and Development (OSHPD) version of the HAZUS evaluation method, as adopted by Chapter 6 of 2010 California Administration Code for seismic evaluation of hospital buildings, for the existing Belmont Plaza Pool Buildings and pool structure. The seismic parameters are based on a site specific response spectrum of Design Earthquake A. The site-specific response spectra for Design Earthquake A are selected for the investigation in order to assist the City of Long Beach in its decision making process involving the current and long-term operational status of the Belmont Pool facilities.
2. Coordinate with the geotechnical sub-consultant to study the liquefaction probability of the site and to develop site specific response parameters for an earthquake event having a 10% probability of being exceeded in 5 years, as well as for an earthquake event having a 10% probability of being exceeded in 50 years.
3. Prepare an evaluation report summarizing TTG's findings.

2. BUILDING MATERIAL PROPERTIES

The properties listed below are indicated on the existing structural drawings and have not been verified by tests or existing test reports. The building's lateral components and their material strengths as indicated on the original construction documents are as follows:

- Concrete: f'c = 3,000 psi normal weight concrete for foundations, slabs, Beams, columns and walls;
  f'c = 5,000 psi Light Weight concrete for precast double-T slabs in Private Event Building roof construction.
  f'c = 5,000 psi Normal Weight concrete for columns in Natatorium Building
- Reinforcement: ASTM A 615, Intermediate Grade, 40 ksi typical reinforcement.
  ASTM A16 Hard Grade 50 ksi for column longitudinal reinforcement in Natatorium Building

Note: Per standard policy and general practice of the Building Safety Agency, original test and inspection reports, material certifications, project specifications, and structural calculations are only kept for 7 years after the completion of construction, meaning these documents are not available for review.

3. SITE & SOIL PARAMETERS

The geotechnical investigation reports by AMEC Inc. dated October 10, 2012 are used as a reference by TTG during the evaluation.

Available geotechnical reports state that the site is underlain by artificial fill, consisting of silty sand, placed for the existing development. The fill is underlain by beach and estuary deposits consisting of poorly graded sand with silty sand, sandy silt and silty clay.

The following SB 1953 seismic coefficients are developed per Chapter 6, Seismic Evaluation Procedure for Hospital Buildings, of the 2010 California Building Standards Administrative Code (2010 CBSAC), and are provided in the geotechnical report.

Acceleration Coefficient:	Aa = 0.4
Velocity Coefficient:	Av = 0.4
Soil Profile Type:	S3
Site Coefficient:	S = 1.5

The geotechnical consultant for the project, AMEC Inc., also provides site specific response parameters for a Design Earthquake A, as well as parameters for a Design Earthquake B. The seismic coefficients for the Design Earthquake A are listed below.

Acceleration Coefficient:	Aa = 0.11
Velocity Coefficient:	Av = 0.13
Soil Profile Type:	S2
Site Coefficient:	S = 1.2

Site specific response spectrum for a Design Earthquake A in the geotechnical reports are used for the dynamic analysis of the structures. The response spectrum is scaled to SB-1953 static base shear levels where applicable. The peak ground acceleration (PGA) for the Design Earthquake A is 0.13g, and the PGA for the Design Earthquake B is 0.37g.

Based on the available geologic data, active or potentially active faults with the potential for surface fault ruptures are not known to be located beneath, or projecting towards, the Belmont Plaza site. The potential for surface ruptures at the site due to fault plane displacement propagating to the ground's surface during the design life of the project is considered low. Liquefaction potential is greatest where the groundwater level is shallow, and submerged, loose, fine sands occur at a depth of around 15 meters (50 feet) or less. The groundwater level for liquefaction analysis is assumed to be 7' below the existing grade based on measurements of the groundwater level. The results of AMEC's site-specific liquefaction evaluation indicate that the medium dense granular soils encountered in AMEC's exploratory borings are susceptible to liquefaction and seismically-induced settlement. The total seismically-induced settlements for the different targeted risk levels are shown in the following table.

Type	Risk Level	Total Seismically-Induced Settlement
Design Earthquake A	10% in 5 years	0 (inch)
Design Earthquake B	10% in 50 years	0.5 to 1.8 (inch)

Some, but not all, liquefiable soils are susceptible to lateral spreading. Up to 5' of lateral movement may occur at the site in the event of ground motion during a Design Earthquake B. However, the potential for lateral spreading at the site in the event of ground motions during a Design Earthquake A is considered low.

4. BUILDING TYPE

The Private Event Building is a reinforced concrete shear wall building with concrete diaphragm at all levels. The structure is classified as **Building Type 9** (Concrete Shear Walls) per Section 2.2.3 of Chapter 6 of 2010 CBSAC.

The building seismic response coefficients are obtained from Chapter 6 of CBSAC 2010:

Response Modification Coefficient: $R = 4.5$
Deformation Modification Coefficient: $C_d = 4$

The Natatorium Building is an ordinary concrete moment frame and shear wall interactive system. The structure can be classified as **Building Type 8** (Concrete moment frame) in combination with **Building Type 9** (Concrete Shear Walls) per Section 2.2.3 of Chapter 6 of 2010 CBSAC. The Natatorium structure may also be classified as **Building Type 12** (Precast Concrete Frame with Shear Walls), because of the presence of precast concrete roof girders, precast beams and precast shears wall tied together by cast-in-place concrete construction, which in many ways fits the description of Building Type 12. For structures with multiple building types, the building type resulting in the maximum collapse probability is utilized to determine the probability of collapse.

The building seismic response coefficients are obtained from Chapter 6 of CBSAC 2010:

Response Modification Coefficient: R=2 for concrete moment frame
Deformation Modification Coefficient: Cd = 2 for concrete moment frame

The Locker Building is a precast concrete frame with concrete shear wall building with concrete diaphragm at roof level. The structure is classified as **Building Type 12** (Precast Concrete Frames with Concrete Shear Walls) per Section 2.2.3 of Chapter 6 of 2010 CBSAC.

The building seismic response coefficients are obtained from Chapter 6 of CBSAC 2010:

Response Modification Coefficient: R = 4.5
Deformation Modification Coefficient: C_d = 4

5. SITE EVALUATION

Site visits were conducted on July 10 and October 16, 2012 to meet with the Belmont Plaza Pool personnel, obtain site data, visually observe the site's physical condition, and corroborate the type and nature of the structures with the drawings. No finishes were removed. No field measurements were taken. The buildings relate to the drawings in the extent indicated above, and appear to be in fair condition with no signs of major structural distress. There are, however, a large number of cracks observable in the concrete slabs as well as deterioration in the concrete beams and columns. Flooding is also reported in the basement level due to past water leakage at the existing pool wall.

6. PAST BUILDING PERFORMANCE

There are no reports of significant damage due to past seismic activities in the Belmont Plaza Buildings. Past performance is discussed with facilities personnel and, to their knowledge, no major structural or non-structural damage has occurred at the Belmont Plaza Pool Buildings as a result of past earthquakes. Damage to the concrete structures was observed during the site visit; however, no written documentation was available for TTG's review (see Figure 16 to Figure 30 of Appendix A for more information). There are a number of cracks in the concrete slab on grade in the Natatorium Building area as well as in the Private Event Building area. Concrete deteriorations is observed in the basement beams and slabs, as well as hairline cracks and minor concrete spalling on some exterior columns near the bottom of precast panel in the Natatorium Building. This is possibly due to stress concentration in the area where the precast panel restrains the columns because of past earthquake events.

Concrete discoloration and streaking are observed in the roof framing of the Natatorium Building, and similar conditions can be seen around the skylights (see Figure 25 of Appendix A). It is not clear whether this is due to roof leakage or condensation of pool water. The interior and exterior environments surrounding the buildings are very conducive to the corrosion of steel reinforcement in the concrete, possibly because of the chemicals used in pool water maintenance, and also the Belmont Plaza Buildings' proximity to the ocean. Cracks and stains can be seen on the roof-overhang of the Natatorium Building, indicating possible corrosion of the reinforcement in the concrete (see Figure 26 of Appendix A). Cracks in the concrete roof slabs overhangs are also present in the Locker Building (see Figure 27 of Appendix A). There are also concrete cracks in the Private Event Building's basement area (see Figure 28 to 30 of Appendix A). Very large, horizontal cracks and signs of heavy rebar corrosion can be seen in the wall separating the pool from the basement of the Private Event Building (see Figure 30 of Appendix A). The electrical distribution boxes have also been damaged by corrosion (see Figure 29 of Appendix A). As seen, the basement environment is clearly corrosive to steel. There are signs of heavy water leaks, which are most likely being caused by faulty pipe connections in the water treatment system. The prolonged exposure of reinforcing steel to this corrosive water and moisture is of great structural safety concern.

7. EVALUATION OF STRUCTURAL DEFICIENCIES AND COLLAPSE PROBABILITY

The evaluations of the probability of collapse of the subject buildings in an earthquake are based on the buildings' structural Lateral Force Resistance System (LFRS) properties and Significant Structural Deficiencies (SSD) per Section 1.4.5.1.2.2 Sub-section 2.2.2 of Chapter 6, 2010 CAC (all section numbers referenced from here on refer to Chapter 6 of 2010 California Administration Code).

In order to determine the existence or absence of torsion irregularity and deformation incompatibility, etc., the Belmont Plaza Buildings are simulated in a 3-D ETABS computer model, which is based on the existing structural drawings of the buildings (see Appendix C for graphics of the computer models and deformed building shapes under seismic loads). All elements believed to be contributing to seismic resistance of the buildings are modeled. Most of the structural members for gravity loads are included in the model, as well as the basement of the Private Event Building. The Belmont Plaza buildings' weight/mass is distributed in the simulation as realistically as possible (see details of the structural calculations provided in Appendix D). The main characteristics of the seismic analysis of these buildings are briefly summarized below.

The seismic base shear coefficient for the Private Event Building and the Locker Building based on CAC 2010 Chapter 6 is 0.061 for a Design Earthquake A, and 0.188 for a Design Earthquake B, where the magnitude of the seismic force for the Design Earthquake B is 3 times that of the Design Earthquake A. For the Natatorium Building, the seismic base shear coefficient is 0.10 for a Design

Earthquake A, and 0.352 for a Design Earthquake B, where the magnitude of the seismic force for the Design Earthquake B is 3.5 times that of the Design Earthquake A.

Building diaphragm displacements and drifts under seismic force corresponding to a Design Earthquake A are summarized in Table 1.

Table 1 - Diaphragm Displacement and Drift under Seismic Force (Multiplied by Cd factor)

	Dispt. at E-W Force (X Dir.)	Dispt. at N-S Force (Y Dir.)	Story Height (ft)	Drift Ratio E-W Force (X Dir.)	Drift Ratio N-S Force (Y Dir.)
Private Event Building	Rf: 0.103” 2nd Flr: 0.041”	Rf: 0.353” 2nd Flr: 0.036”	Rf: 16’ 2nd Flr: 15’	Rf: 0.0003 2nd Flr: 0.0002	Rf:0.0017 2nd Flr:0.0002
Natatorium Building	Rf: 1.13”	Rf: 1.56”	Rf: 49’	Rf: 0.0019	Rf: 0.0027
Locker Building	Rf: 0.008“	Rf: 0.005”	Rf: 12.5’	Rf: 0.00005	Rf: 0.00003

The evaluation of the Significant Structural Deficiencies (SSD) in accordance with Chapter 6 of 2010 CAC, subsection 1.4.5.1.2.2 for the Belmont Plaza Buildings and their results are summarized below for each building (more details of the HAZUS-critical SSD evaluation for the buildings are shown in Appendix D).

Private Event Building – Significant Structural Deficiencies

Material Test – No material testing information is available. The unconfirmed structural material quality is identified as a deficiency (Material Test deficiencies exist).

Weak Story – There are significant strength discontinuities in many of the vertical elements composing the Lateral Force Resisting System (the story strength at any story shall not be less than 80% of the strength of the story above it). The 2nd floor shear walls do not continue to the foundation. The story strength of the 1st floor is not less than 80% of the story strength of the 2nd floor. For HAZUS calculations, this deficiency is considered to exist due to the discontinuity of shear walls.

Soft Story – There are significant stiffness discontinuities in many of the vertical elements in the Lateral Force Resisting System (the lateral stiffness of a story shall not be less than 70% of that of the story above or less than 80% of the average stiffness of the three stories above it). For this building, the 2nd floor shear walls do not continue to foundation. The story stiffness of the 1st floor is not less than 70% of the story stiffness of the 2nd floor. For HAZUS calculations, this deficiency is considered to exist due to the discontinuity of shear walls.

Vertical Discontinuity – The shear walls do not continue to foundation, causing vertical discontinuity deficiency to exist.

Concrete Deterioration – Cracks in concrete members and corrosion of the reinforcement are observed in beams and walls, causing concrete deterioration deficiencies to exist.

Weak Column – The concrete columns supporting the discontinuous walls appear to be weak. Weak column deficiencies exist.

Natatorium Building – Significant Structural Deficiencies

Material Test – No material testing information is available. The unconfirmed structural material quality is identified as a deficiency (Material Test deficiencies exist).

Weak Story – There are significant strength discontinuities in many of the vertical elements in the Lateral Force Resisting System (the story strength at any story shall not be less than 80% of the strength of the story above it). The upper precast shear walls do not continue to the foundation. The strength of the columns below the shear walls are less than 80% of the strength of the shear walls. For HAZUS calculations, this deficiency is considered to exist.

Soft Story – There are significant stiffness discontinuities in many of the vertical elements in the Lateral Force Resisting System (the lateral stiffness of a story shall not be less than 70% of that of the story above or less than 80% of the average stiffness of the three stories above it). For this building, the precast shear walls stop 25 feet above the 1st floor. The stiffness of columns below the shear walls is less than 70% of the stiffness of the shear walls. For HAZUS calculations, this deficiency is considered to exist.

Vertical Discontinuity – The shear walls do not continue to foundation. Vertical discontinuity deficiencies exist.

Deflection Incompatibility – Column and beam assemblies that are not part of the Lateral Force Resisting System (gravity load carrying frames) are not capable of accommodating imposed building drifts, including amplified drift, caused by diaphragm deflections. All columns in this building serve both as the only gravity load and lateral load carrying elements. The capacity of the columns for amplified loads (multiplied by Cd factor) is not sufficient. Lack of ductility in the columns and the high expected deformations right below the precast concrete panel will cause the yielding of the gravity/lateral columns. Once any one column fails, the Vertical Load Resisting System will not fully function, and the building will likely suffer partial or complete collapse. Therefore, deflection incompatibility deficiencies exist.

Concrete Deterioration – Cracks in the concrete members and corrosion of steel reinforcement are observed in the columns, beams and walls; concrete deterioration deficiencies exist.

Weak Column – The concrete columns supporting discontinuous wall appear to be weak. Weak column deficiencies exist.

Locker Building – Significant Structural Deficiencies

Material Test – No material testing information is available. The unconfirmed structural material quality is identified as a deficiency (Material Test deficiencies exist).

Deflection Incompatibility – Column and beam assemblies that are not part of the Lateral Force Resisting System (gravity load carrying frames) are not capable of accommodating imposed building drifts, including amplified drift caused by diaphragm deflections. There are a large number of shear walls in this building and the building drift will be very small. Deflection incompatibility deficiencies do not exist in this HAZUS calculation.

Concrete Deterioration – Cracks in the concrete members and corrosion of steel reinforcement is observed in the beams and walls. Deterioration deficiencies exist.

TTG calculated the collapse probability of the Belmont Plaza Pool buildings $\rho_{[col]}$ using the method adopted in Chapter 6 of 2010 CAC for a Design Earthquake A. The results of the collapse probability for each building are summarized in Table 2 below (details of the calculations are provided in Appendix D).

TTG's calculations of the collapse probability $\rho_{[col]}$ for the Private Event Facility Building and Locker Building are less than 1.2%, with SSD as identified in Table 2 below. It is TTG's opinion that these two buildings can be classified as SPC-2 for earthquake events having a 10% probability of being exceeded in 5 years (these buildings do not significantly jeopardize life, but may not be reparable or functional following the design ground motion). The collapse probability $\rho_{[col]}$ for the Natatorium Building in its current "as-is" condition is greater than 1.2%, which indicates the building falls under the SPC-1 category, and poses a significant risk of collapse and a danger to the public in the event of a Design Earthquake A (moderate earthquake of magnitude 5~5.9, having a 10% probability of being exceeded in 5 years). After reviewing the condition of the diving platforms and pool walls areas, TTG believes the collapse probabilities for these areas are very low, and they are not at risk from earthquake event having a 10% probability of being exceeded in 5 years.

Table 2 – (Buildings in "as-is" condition) Collapse Probability for Earthquake Event Having a 10% Probability of being Exceeded in 5 Years (Design Earthquake A)

Building Name	Collapse Probability $\rho_{[col]}$	Remark
Private Event Building	0.68%	Deficiencies included in calculation: Material Test, Weak Story, Soft Story, Vertical Discontinuity, Concrete Deterioration, Weak Column.
		< 1.2% Considered not at risk of collapse
Natatorium Building	1.51%	Deficiencies included in calculation: Material Test, Weak Story, Soft Story, Vertical Discontinuity, Deflection Incompatibility, Concrete Deterioration, Weak Column
		> 1.2% Considered at risk of collapse
Locker Building	0.08%	Deficiencies: Material Test, Concrete Deterioration
		< 1.2% Considered not at risk of collapse

8. STRUCTURAL UPGRADE RECOMMENDATIONS FOR NATATORIUM BUILDING

Based on TTG's evaluation of the collapse probability of the Natatorium Building, the existing structure poses a significant risk of collapse from a Design Earthquake A event, and the collapse hazard is a danger to public safety. The architectural configuration of the deep concrete spandrels (25' deep precast concrete panels) spanning between the 48' high concrete columns has an adversely dominant influence on the earthquake-resisting performance of the Natatorium Building. All seismic movements/energies of the very rigid spandrels are transmitted to the slender concrete columns, allowing overstressed damages to be concentrated in the limited column areas rather than being distributed equally. Evidence of existing concrete column cracks in these areas from the previous minor earthquakes can be found in Figure 16 through Figure 24 of Appendix A. The last moderate earthquake, a magnitude 5.4, occurred near the project site in 1933. In the event of a strong-to-major earthquake, magnitude 6.0 to 7.9, the existing #3 column rebar ties at 18" on center will be widely splayed. After losing the confinement of column rebar ties, the concrete core will shatter and vertical rebars will buckle similar to that in the photos of the Imperial County Service Building in Appendix E. In this example, the Imperial County Service Building did not fully collapse due to the seismic loads transferred to its interior concrete columns and shear walls, which prevented its total structural failure. The Natatorium Building does not have this interior structure, which puts it in serious risk of total structural collapse if and when the exterior concrete columns fail during a Design Earthquake A. However, it is TTG's opinion that if the concrete columns of the Natatorium Building are strengthened, the collapse probability of the building can be reduced to less than 1.2%, greatly enhancing the survivability of the Natatorium. Thus, the goal of collapse prevention for the Design Earthquake A can be achieved. See Table 3 below for the summary of collapse probability results after the columns have been strengthened (details of the collapse probability $\rho_{[col]}$ calculations are shown in Appendix D).

Table 3 - Collapse Probability for the Design Earthquake A
[After strengthening columns of the Natatorium Building]

Building Name	Collapse Probability $\rho_{[col]}$	Remark
Natatorium Building	0.03%	Deficiencies included in calculation: Material Test, Concrete Deterioration, Weak Column (Please see Note 1)
		< 1.2% Considered not at risk of collapse

Note 1: *The structural deficiencies of weak story, soft story, and vertical discontinuity are considered to be remedied in a HAZUS evaluation when concrete columns are strengthened.*

The proposed strengthening of the columns in the Natatorium Building can be accomplished by using a fiber-wrap strengthening method. Cracks and deterioration of the concrete already occurring in the columns shall be filled with pressurized epoxy injection. Then the existing concrete columns surfaces are wrapped with the carbon reinforcing fabric and bonded with epoxy. The proposed strengthening will enhance the capacity of columns in the regions of extremely high stress concentrations. Once the columns are strengthened, the collapse probability of the building will be less than 1.2%, which is considered not at risk of complete collapse, but may not be reparable or functional following a Design Earthquake A. Other major concrete cracks with potentially corroded reinforcing steel may require additional remedies to maintain the short-term structural integrity of the building. The fiber wrap strengthening will only provide a short-term solution; major structural upgrades will still be required.

9. CONCLUSIONS

TTG's evaluation concludes that all three buildings of the Belmont Plaza fall short of achieving a "collapse prevention performance objective" for a Design Earthquake B. There are many major deficiencies that require seismic risk mitigations. The combination of a high collapse risk structure, no anchorage between concrete pile cap and timber piles, insufficient strength of timber piles, and geological-seismic hazards would make the cost of a complete facility rehabilitation extremely high, and the cost-benefit of the rehabilitation fairly low (approximately $44 million for construction alone).

TTG's evaluation of the collapse probabilities of the buildings at the Belmont Plaza Pool complex show that for a Design Earthquake A, the Private Event Building and the Locker Building have a collapse probability of less than 1.2% and are considered not at risk of complete collapse, but may not be reparable or functional following a Design Earthquake A.

The collapse probability for the Natatorium Building in its current condition is greater than 1.2% for a Design Earthquake A, and is considered to pose a danger to the public due to its significant risk of collapse. The Natatorium Building has many major structural deficiencies, of which, its weak and non-ductile concrete columns contribute the most important factors to its risk of collapse. Unlike the Imperial County Service Building in the photos of Appendix E, the Natatorium Building does not have an interior lateral force resisting system, which puts the Natatorium Building in serious risk of total structural collapse if the its columns ever suffer similar damage.

Limited structural retrofit can achieve only a short-term collapse prevention objective for the Natatorium Building. Long-term solutions for the Natatorium Building must be either comprehensively upgraded or removed from service to mitigate the earthquake risk to the building and the occupants.

Appendix A –Figures

Figure 1 – Belmont Plaza Olympic Pool Structures, View from South
[Community Facility to the West (left), Natatorium at Center, Locker Room to the East (right)]

Figure 2 – Community Facility Building, View from Southwest
[with Natatorium Building in the background]

Figure 3 – Community Facility Building, View from West
[with Natatorium Building in the background]

Figure 4 –Locker Building, View from Northeast
[with Natatorium Building in the background]

Figure 5 – Locker Building, View from Southeast
[with Natatorium Building in the background]

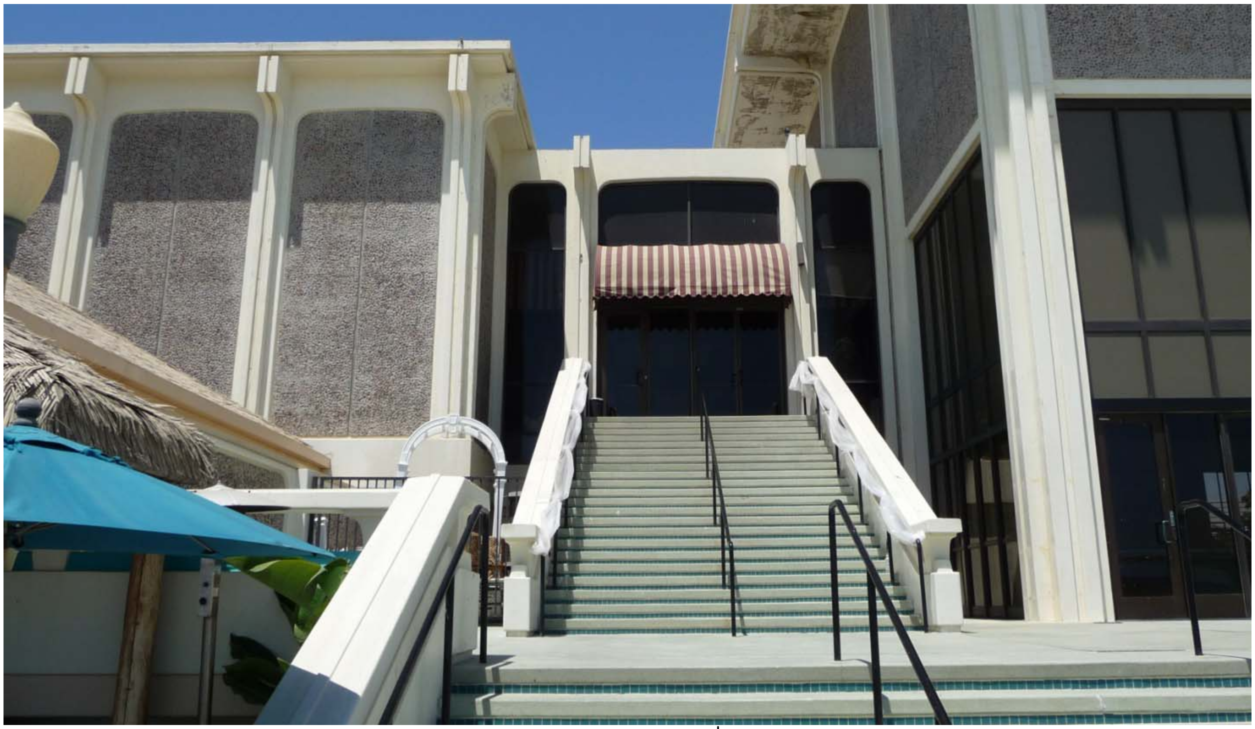


Figure 6 – Stair at Community Building 2nd Floor South Entrance

Figure 7 – Ramp at Community Building 2nd Floor North Entrance

Figure 8 – Locker Building to Natatorium Connection Corridor, North View

Figure 9 – Locker Building to Natatorium Connection Corridor, South View

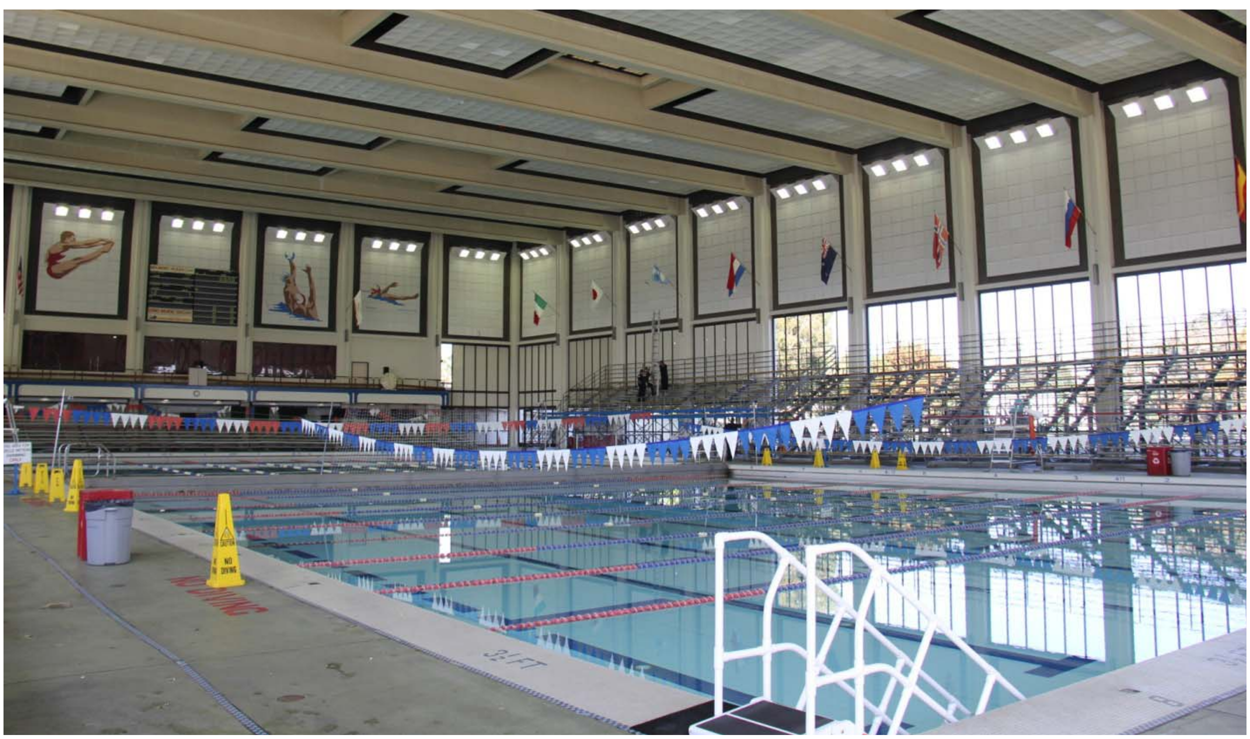


Figure 10 –Natatorium Building, Interior View from Southeast

Figure 11 –Natatorium, Balcony at West End (next to Community Building 2nd Floor)

Figure 12 –Natatorium, Balcony/Entrance at East End (next to Locker Corridor)

Figure 13 –Natatorium, Diving Platforms

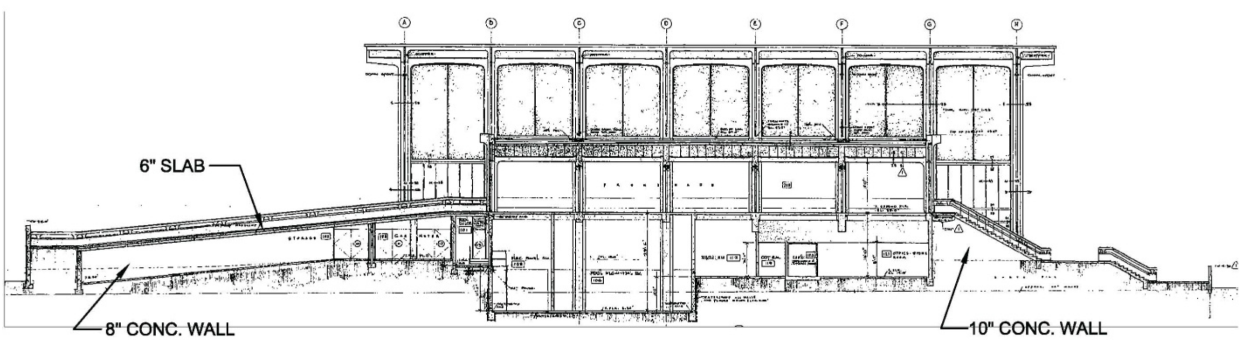


Figure 14 – Community Building Section North-South

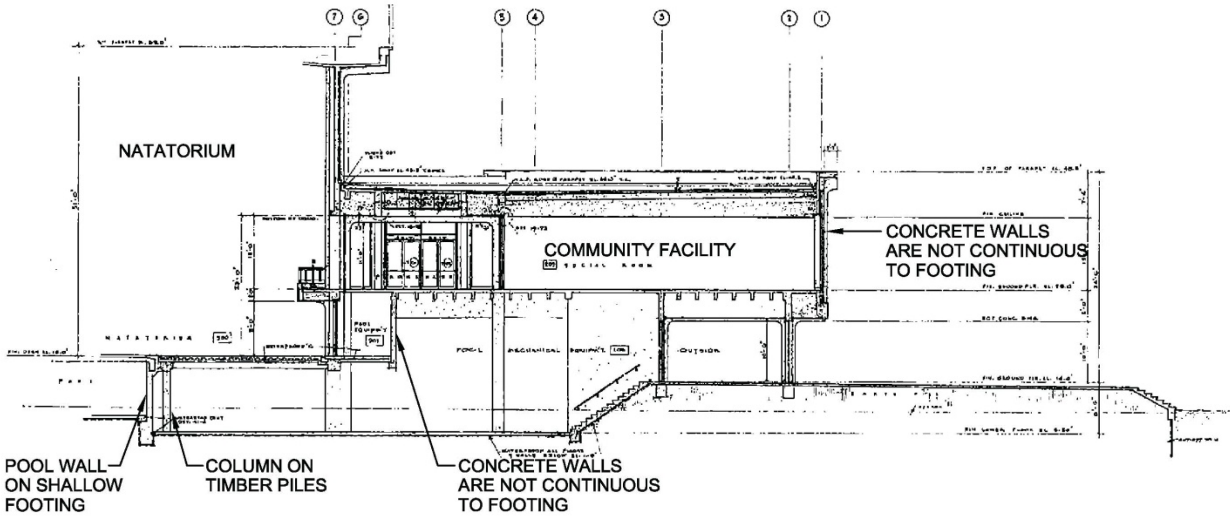


Figure 15 – Community Building Section East-West

Figure 16 – Natatorium Building, Cracks in Column at Northeast Corner (Grid A/7)

Figure 17 – Natatorium Building, Cracks in Column at North Elevation (Grid A/11)

Figure 18 – Natatorium Building, Cracks in Column at Northeast Corner (Grid A/18)

Figure 19 – Natatorium Building, Cracks in Column at East Elevation (Grid B/18)

Figure 20 – Natatorium Building, Cracks in Column at East Elevation (Grid C/18)

Figure 21 – Natatorium Building, Cracks in Column at East Elevation (Grid C/18)

Figure 22 – Natatorium Building, Cracks in Column at Southeast Corner (Grid H/18)

Figure 23 – Natatorium Building, Cracks in Column at South Elevation (Grid H/17)

Figure 24 – Natatorium Building, Cracks in Column at Southwest Corner (Grid H/7)

Figure 25 – Natatorium Building, Stains in Roof Slab/Beam at Interior South End

Figure 26 – Natatorium Building, Cracks in Concrete Wall/Roof at North Elevation

Figure 27 – Locker Building, Cracks in Concrete Roof Slab at West Elevation

Figure 28 – Community Building, Cracks in Concrete Beams in Basement

Figure 29 – Community Building, Cracks in Concrete Beams in Basement

Figure 30 – Community Building, Cracks in Concrete Wall in Basement

Appendix B – HAZUS Method Overview (as Adopted by OSHPD)

HAZUS Overview

A Brief Introduction of the HAZUS Methodology as Adopted by Office of Statewide Health Planning and Development (OSHPD)

What is HAZUS?

HAZUS (Stands for Hazards U.S.) is a state-of-the art technology for risk assessment.
HAZUS has been developed by FEMA beginning in the early 1990s.
HAZUS is a methodology and tools/software.
The goal of HAZUS is to calculate potential losses from an earthquake, hurricane, or flood.
Losses in the form of structural damage, deaths, or economic losses can be evaluated.

HAZUS Adopted by OSHPD

California Building Standards Commission approved the implementation of HAZUS on November 14, 2007 to reexamine the seismic risk Structural Performance Category 1 (SPC-1) hospital buildings. These buildings are considered at risk of collapse in the event of an earthquake or other natural disaster. This reassessment allows the OSHPD to reprioritize SPC-1 hospital buildings based on their level of seismic risk and if they meet specified criteria, move the buildings to SPC-2 category. If reclassified to SPC-2, these hospital buildings would move from a 2008/2013 seismic deadline to a 2030 deadline. Per OSHPD estimate, 50% to 60% of the 1100 SPC-1 hospital building would qualify for the reclassification under the new HAZUS methodology. Current version of the OSHPD adopted HAZUS method is in Chapter 6 of 2010 California Administrative Code, "Seismic Evaluation Procedures for Hospital Buildings" (Administrative Regulations for the OSHPD); more information about the OSHPD HAZUS can be found in its website at http://www.oshpd.ca.gov.

The goal of OSHPD's version of HAZUS is to calculate the probability of collapse of an individual hospital building in an earthquake.

For this project, we applied the methodology of OSHPD's HAZUS for evaluation of the collapse probability of the buildings in Belmont Plaza Pool complex.

SPC-1 Building

Buildings posing a significant risk of collapse and a danger to the public. Where OSHPD has performed a collapse probability assessment, buildings with Probability of Collapse greater than 1.20% shall be placed in this category.

SPC-2 Building

Buildings in compliance with the pre-1973 California Building Standards Code or other applicable standards, but not in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act. These buildings do not significantly jeopardize life, but may not be repairable or functional following strong ground motion. These buildings must be brought into compliance with the structural provisions of the Alquist Hospital Facilities Seismic

Safety Act, its regulations or its retrofit provisions by January 1, 2030, or be removed from acute care service.
Where OSHPD has performed a collapse probability assessment, buildings with Probability of Collapse less than or equal to 1.20% shall be placed in this category.

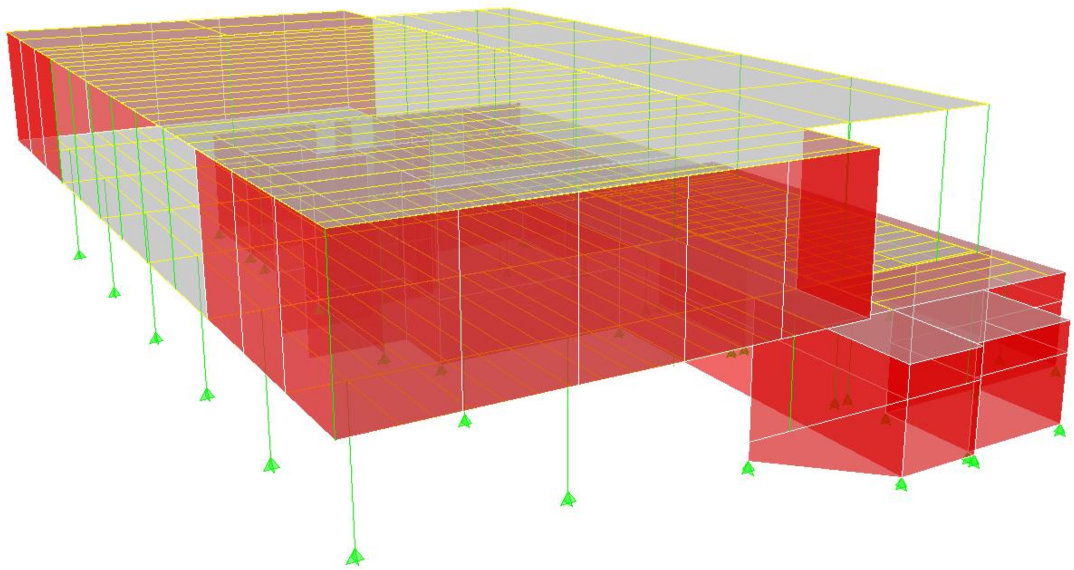
Alquist Hospital Facilities Seismic Safety Act
Following the 1994 Northridge Earthquake, which resulted in $3 billion damage to hospitals, the Alfred E. Alquist Hospital Facilities Seismic Safety Act was amended under SB 1953 (Senate Bill 1953, Chapter 740, Statutes of 1994), Seismic Mandate.

The seismic mandate established five structural and five non-structural classifications of hospital building seismic-safety levels, as well as deadlines for some classification upgrades.

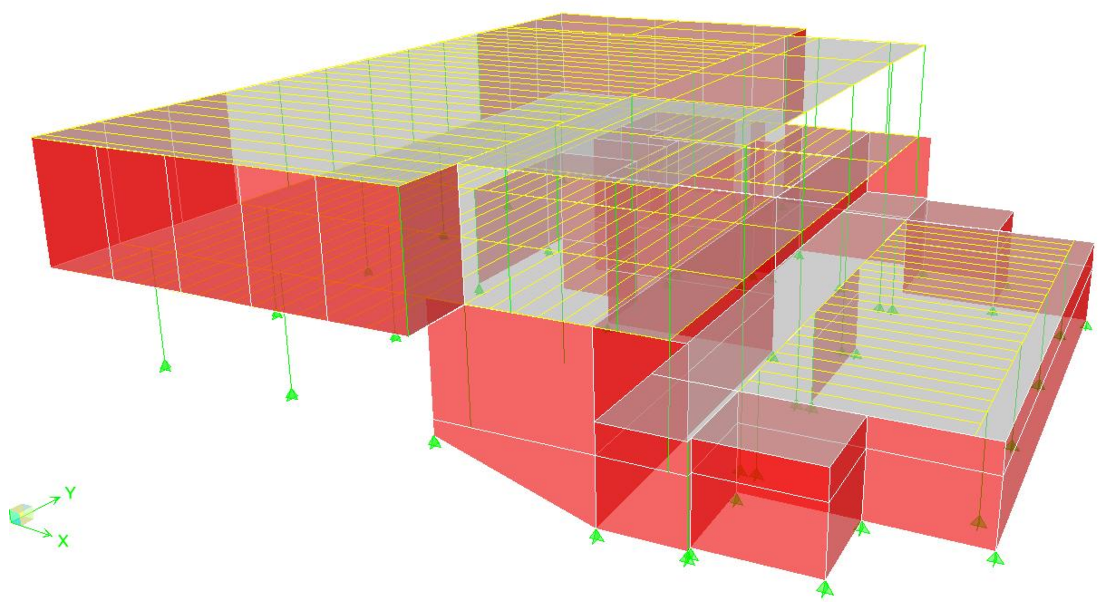
The mandate has been amended a number of times since its enactment. To view all legislation that has amended the seismic mandate, go to website of OSHPD Facilities Development Division (FDD) Recent Legislation (http://www.oshpd.ca.gov).

Appendix – C: Computer Models and Deformed Building Shapes

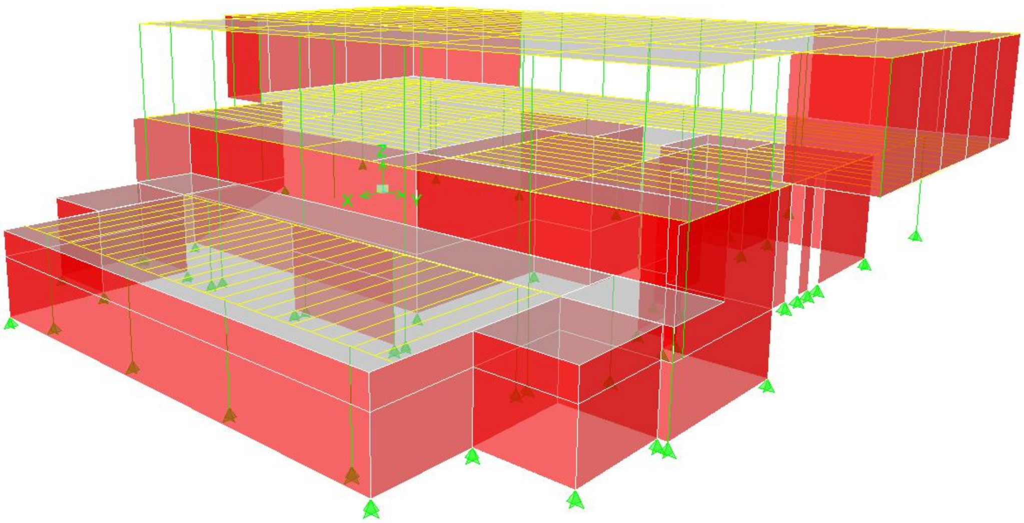
(Selected Snapshots of ETABS Computer Model Views for the Buildings)



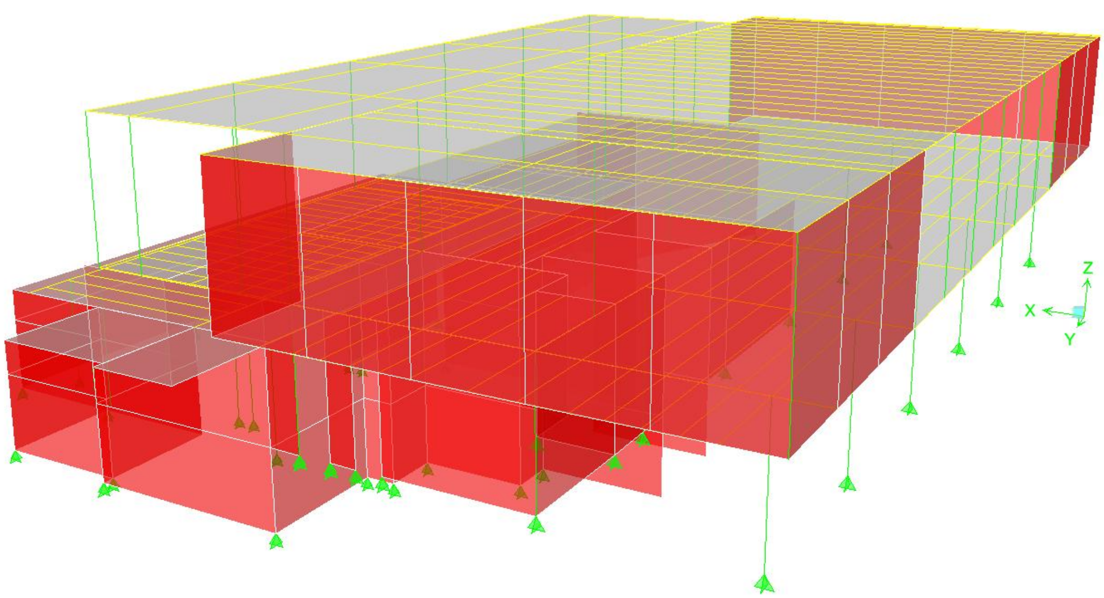
Community Building – View from South-West



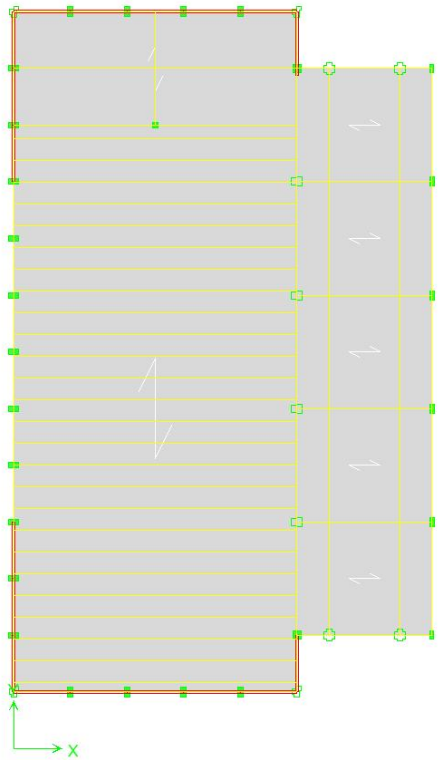
Community Building – View from South-East



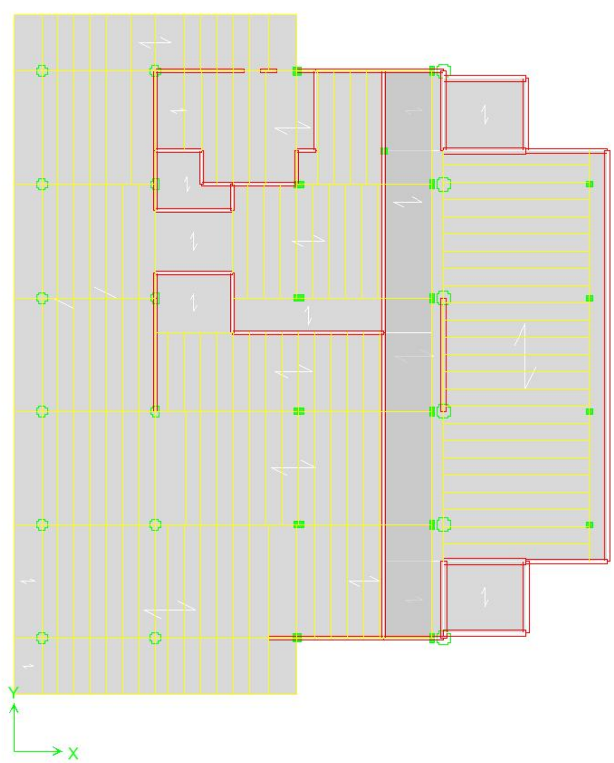
Community Building – View from North-East



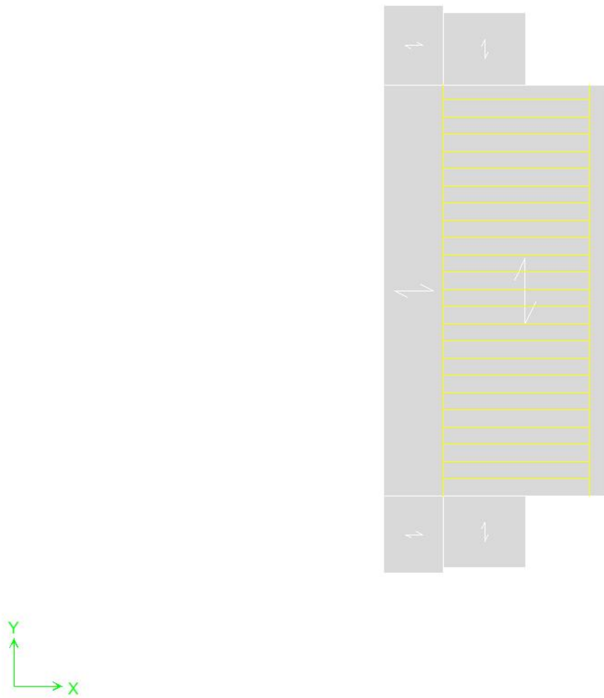
Community Building – View from North-West



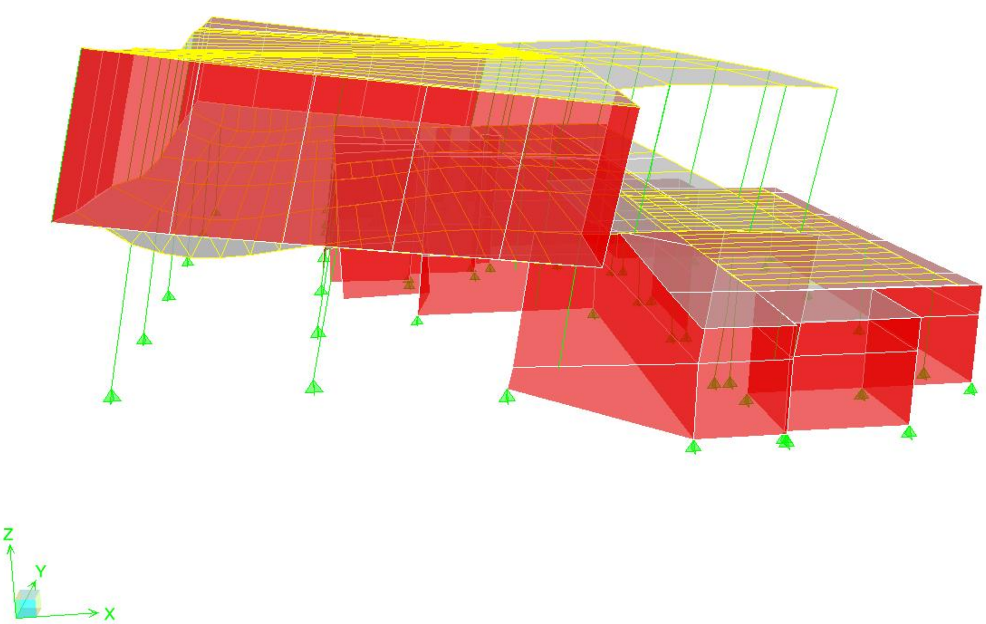
Community Building – Roof Plan View



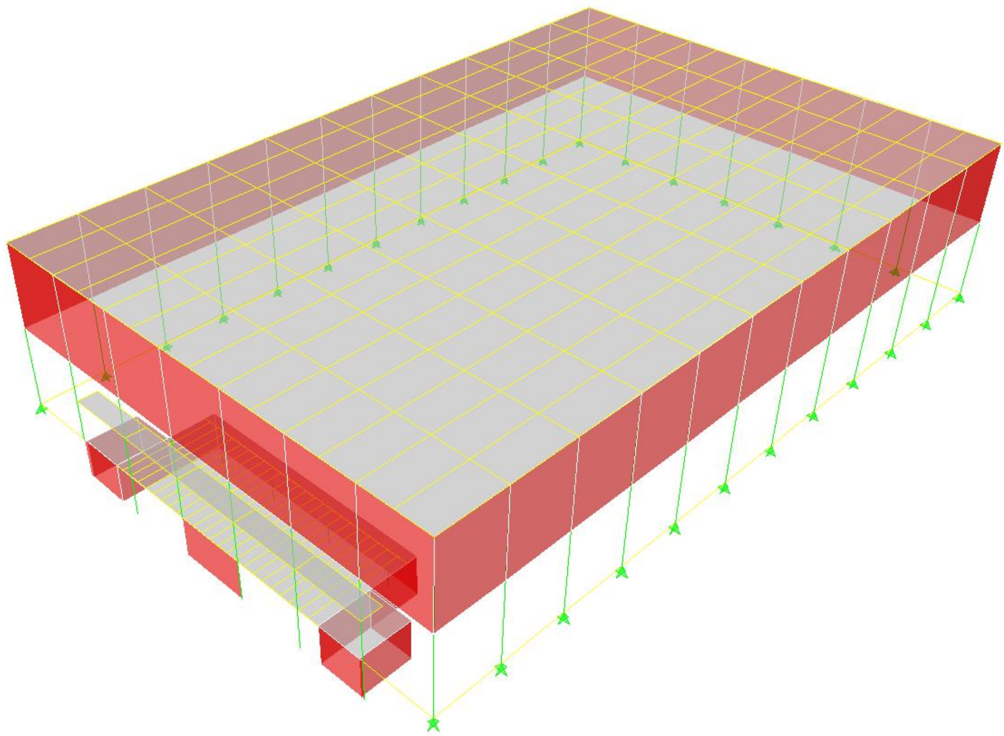
Community Building – 2nd Floor Plan View



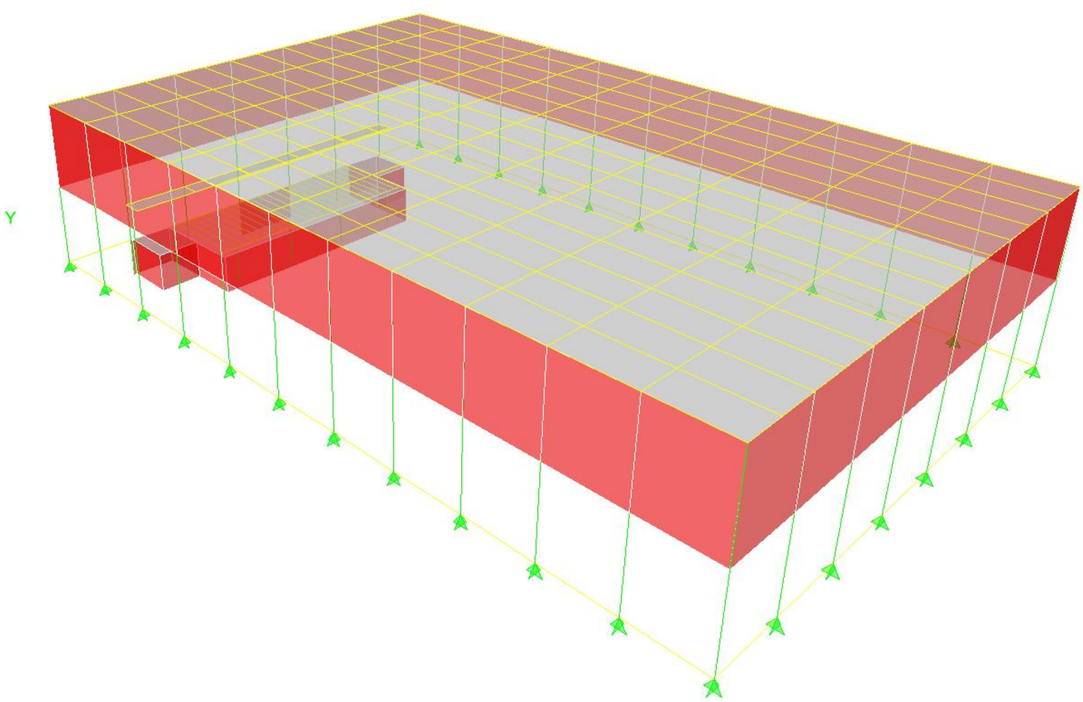
Community Building – Pool Deck Level Plan View



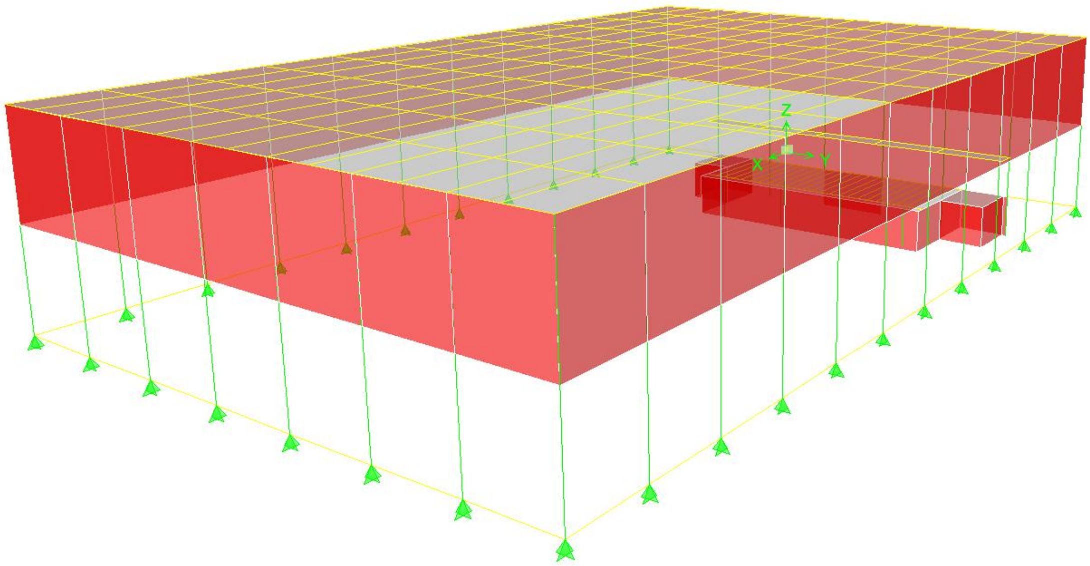
Community Building – Deformed Building Shape under E-W Direction Lateral Force



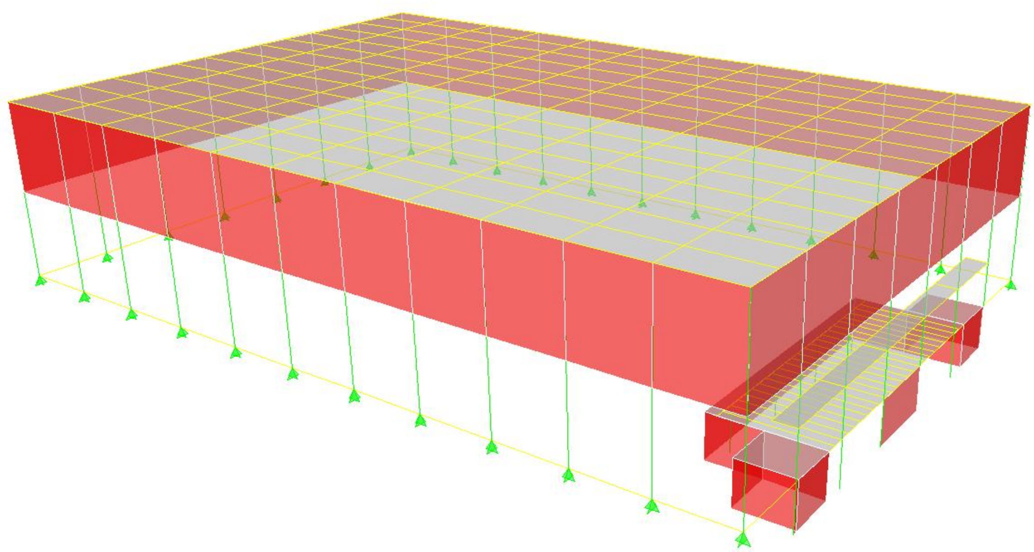
Natatorium Building –View from South-West



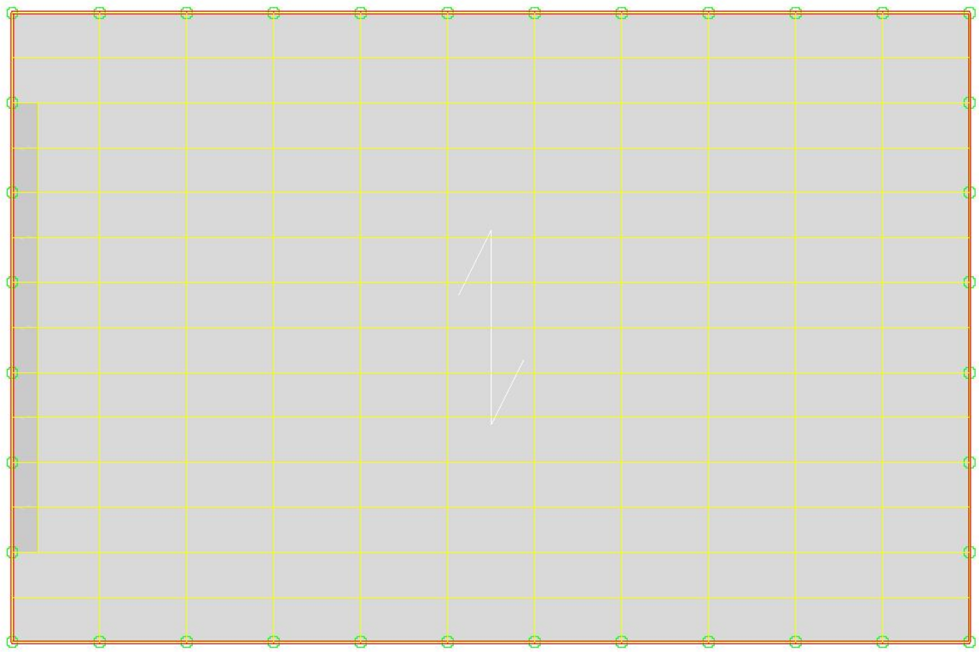
Natatorium Building –View from South-East



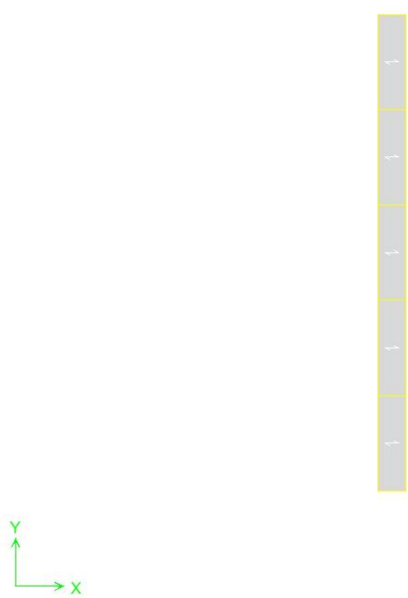
Natatorium Building –View from North-East



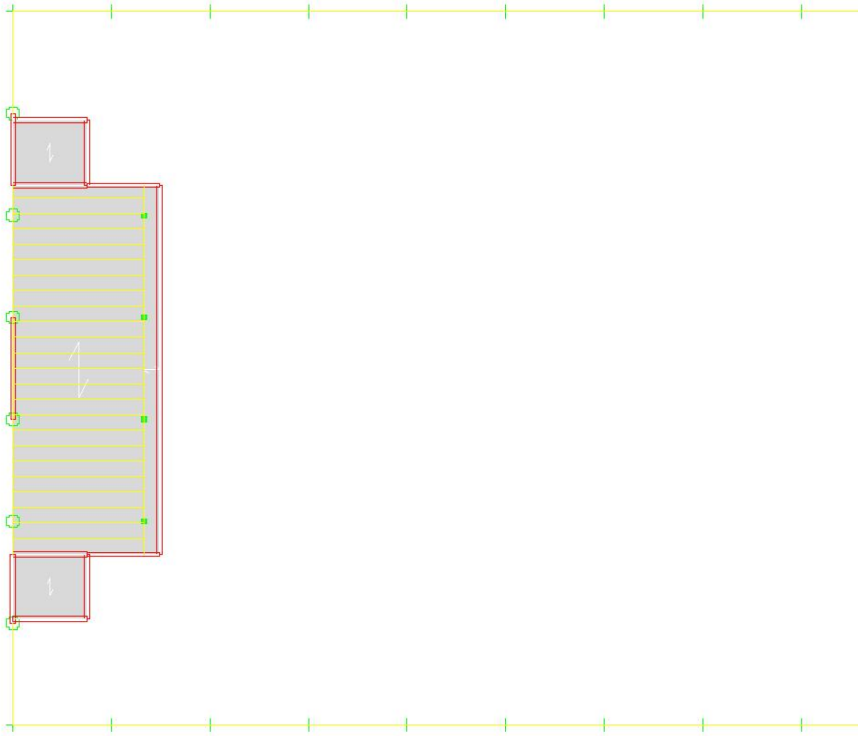
Natatorium Building –View from North-West



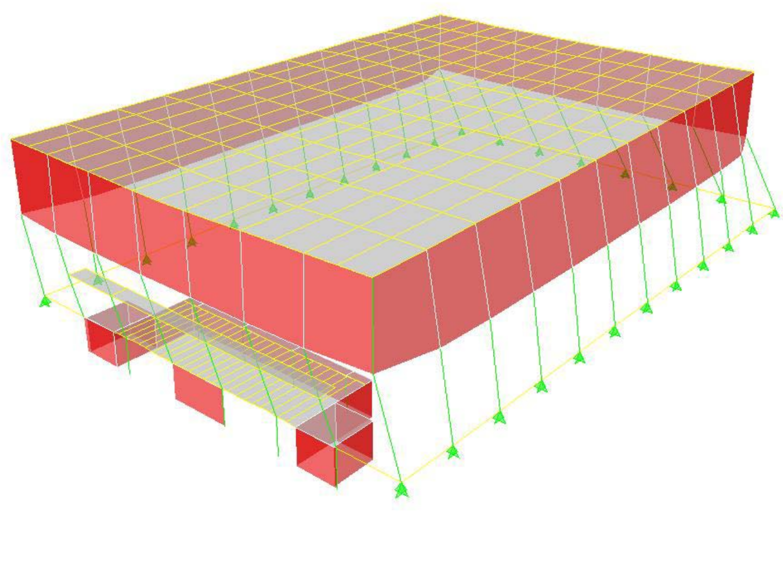
Natatorium Building –Roof Plan View



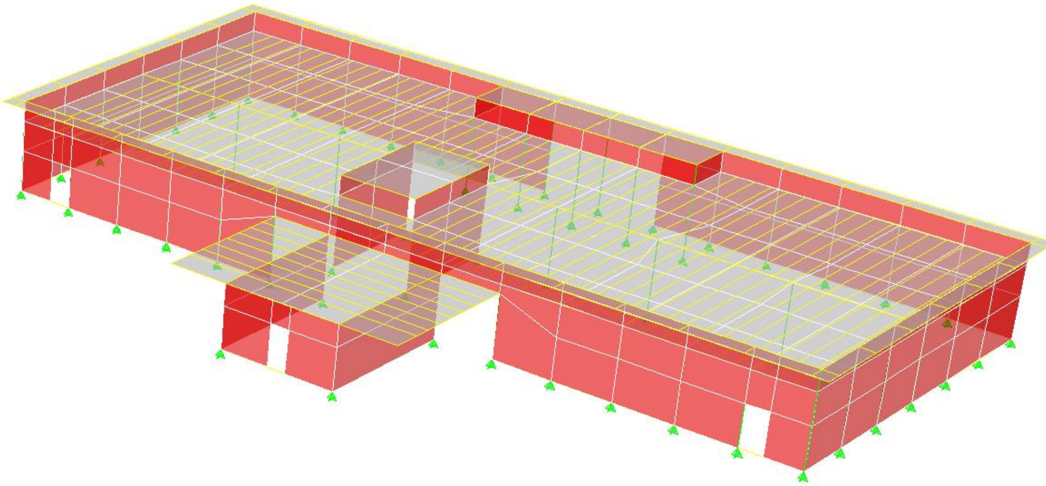
Natatorium Building –Balcony Level Plan View



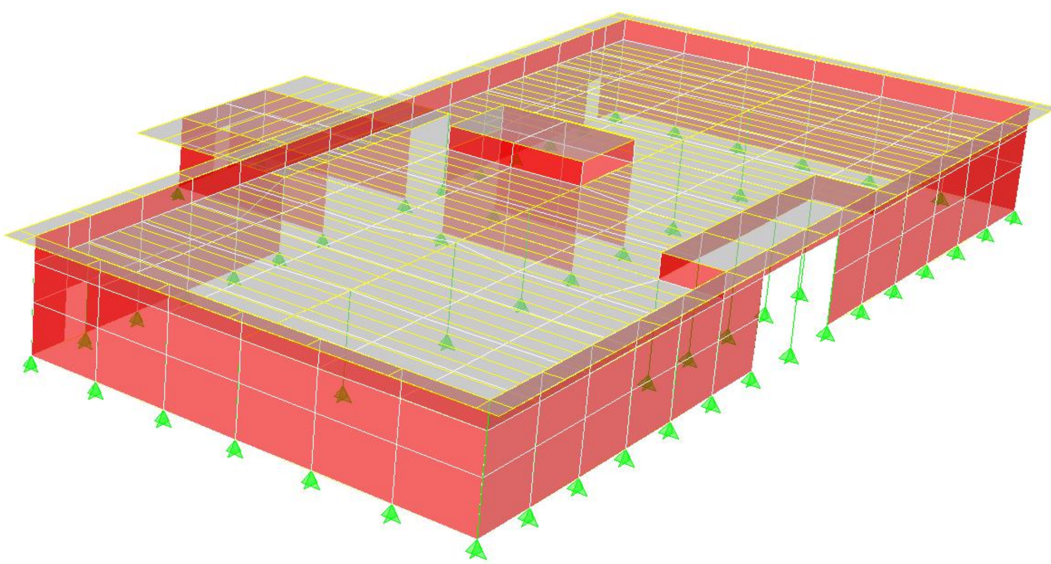
Natatorium Building –Pool Deck Level Plan View



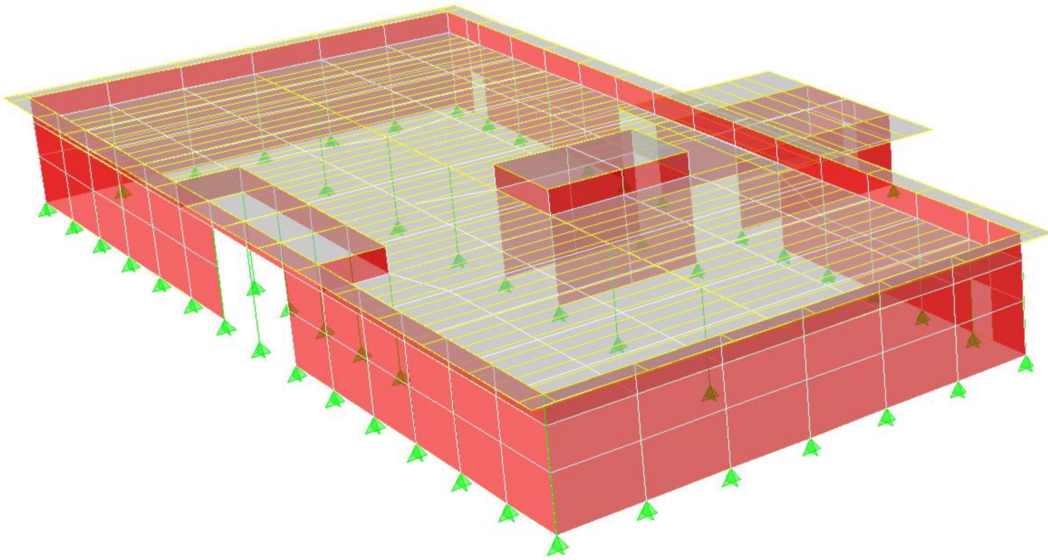
Natatorium Building –Deformed Building Shape under N-S Direction Lateral Force



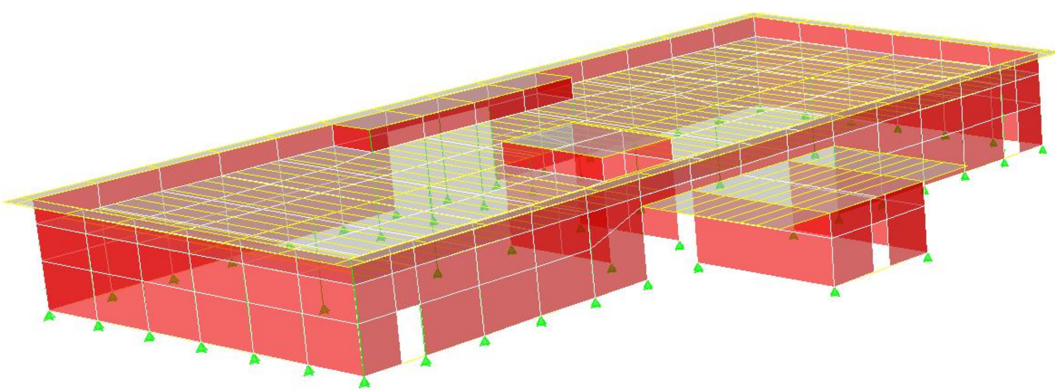
Locker Building – View from South-West



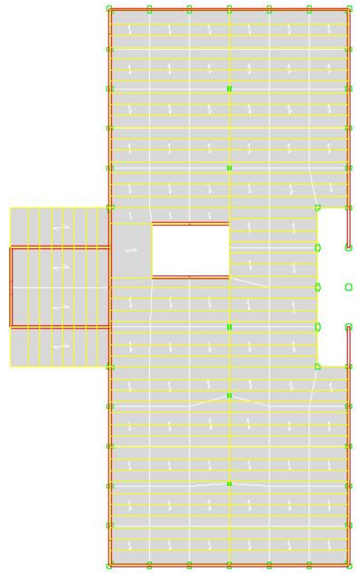
Locker Building – View from South-East



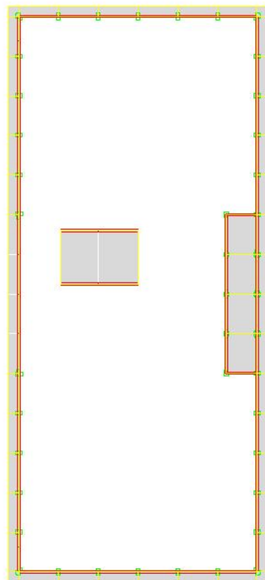
Locker Building – View from North-East



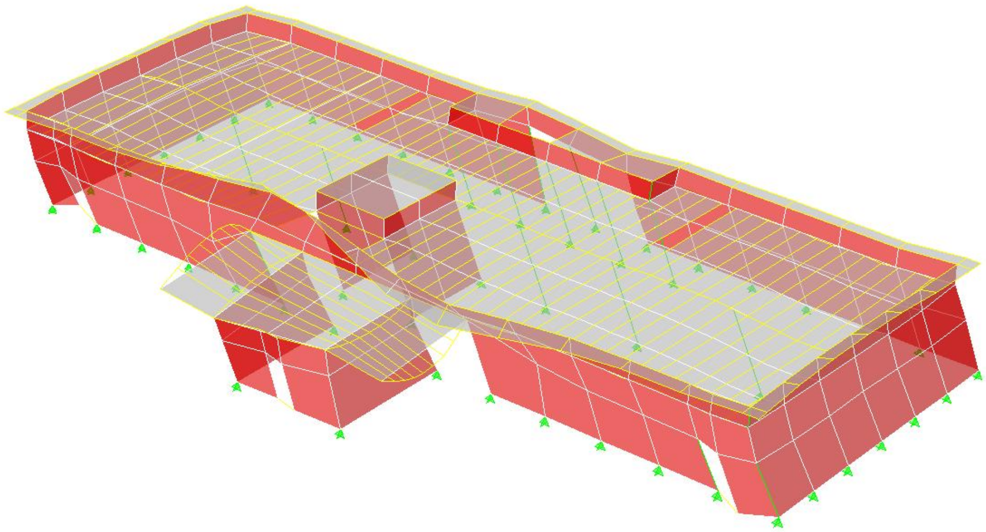
Locker Building – View from North-West



Locker Building – Roof Plan View



Locker Building – High Roof Level Plan View



Locker Building – Deformed Building Shape under N-S Direction Force

Appendix D – Structural Calculations

(a) Loading Criteria

BELMONT PLAZA, COMMUNITY CENTER BLDG, LOADING CRITERIA

ROOF & FLOOR, DEAD LOAD AND SEISMIC DEAD LOAD (Additional to Self Wt & Mass)

Roof (at metal deck without concrete fill).

		Gravity	PSF		Seismic
Roofing and Insulation		8.0			8.0
Ceiling		3.0			3.0
MEP		3.0			3.0
Misc		2.0			2.0
DL & Mass, additioinal to self	Σ =	**16.0**		Σ =	**16.0**
				Add partition 1/2 of 15 psf =	**7.5**
	Use	16.0		Use	**23.5**
				Mass (Wt/32.2) =	**0.730**

Note: Concrete slab, beam and column wt and mass calculated by ETABS

Floor Typical, Cast-in-place concret slab/beam system (Normal Wt Conc 150 lbs/ft3).

		Gravity	PSF		Seismic
Floor Finishes		2.0			2.0
Ceiling		3.0			3.0
MEP		3.0			3.0
Misc		2.0			2.0
DL & Mass, additioinal to self	Σ =	**10.0**		Σ =	**10.0**
				Add for partition	**15**
	Use	10		Use	**25**
				Mass (Wt/32.2) =	**0.776**

Note: Concrete slab, beam and column wt and mass calculated by ETABS

WALL LOAD

	(PSF)
8" Concrete Wall or Slab	100
10" Concrete Wall	125
12" Concrete Wall	150
16" Concrete Wall	200

OTHER LOADS

Roof live load:	**RLL** =	**20.0**	PSF	
Roof live load:	**RLL =**	**50.0**	PSF	Mechanincal & Electrical Equipment Areas
Floor Live Loads:	**LL** =	**60.0**	PSF	
Floor Live Loads:	**LL =**	**80.0**	PSF	Include 20 psf partition weight
Floor Live Loads:	**LL** =	**100.0**	PSF	at Exit Corridors
Mech Room Flr Live	**LL** =	**100.0**	PSF	

(b) MATERIAL PROPERTIES USED IN ETABS MODEL

Concrete Properties

Assumtions used in ETABS Analysis								
R	**C_d**	**Orthogonal Effects**	**Beams**	**Columns**	**Walls In-plane / Out-of-plane**	**Slab In-plane / Out-of-plane**	**f'_c**	**f_y**
4.5	4	N/A	0.3EI	0.3EI	0.5EI/0.35EI	EI/0.25EI	3	40

Notes:
1. Typical concrete properties for reinforced concrete shear walls structure only.
2. R=2, Cd=2 for Natatorium Building as a Ordinary Concrete Moment Frame structure

LOAD COMBINATIONS USED IN ANALYSIS

Load Combination per Section 2.4.2, Chapter 6 of CAC 2010

$1.1D+0.25L + Q_E$	(Eq 2-1) Load Comination
$0.9D + Q_E$	(Eq 2-2) Load Comination
$0.9D - Q_E$	(Eq 2-2) Load Comination

Cd= 4

Q_E = Cd/2 E = 2*E For Vertical Discontinuity Check

Q_E = Cd E = 4*E for Deformation Incompatibility Check

(c) Seismic parameters and Response Spectrum
based on Geotech report

kilometers based on the available geologic data. To account for the uncertainty in the ground motion attenuation relationships, each relationship was integrated to three standard deviations beyond the median.

The PSHA analysis integrates the effects of all the earthquakes of different sizes, occurring at different sources at different probabilities of occurrence, to provide an estimate of probability of exceeding different levels of ground motion at a site during a specified period of time. Based on the estimated remaining life of 5 to 10 years for the facility, site-specific response spectra were developed for ground motions having a 10% probability of being exceeded in 5 years and a 10% probability of being exceeded in 50 years. The 2, 5, and 10% damped site-specific response spectra for the ground motion having a 10% probability of being exceeded in 5 years are shown on Figure 2.1. The 2, 5, and 10% damped site-specific response spectra for the ground motion having a 10% probability of being exceeded in 50 years are shown on Figure 2.2. The site-specific response spectra for both risk levels in digitized form are shown on Tables 1 and 2.

SEISMIC DESIGN PARAMETERS

We understand that the proposed HAZUS evaluation is being performed using the requirements of Senate Bill 1953 (SB 1953). The soil profile type for the site may be classified as S_2 per SB 1953 when considering the community building or when considering the ground motion having a 10% probability of being exceeded in 5 years. Due to the liquefaction potential when considering the ground motion having a 10% probability of being exceeded in 50 years, additional analyses may be required to assess the site response beneath the pool enclosing structure.

The code-based values of the Effective Peak Acceleration and Velocity Coefficients (A_a and A_v), as defined in Article 2 of SB 1953, may both be taken as 0.4g; however, the site-specific values of A_a and A_v are presented in the following table for the two risk levels currently under consideration for the project:

Risk Level	A_a (g)	A_v (g)
10% in 5 years	0.11	0.13
10% in 50 years	0.30	0.38

Values used in this evaluation, typ.

LIQUEFACTION POTENTIAL

Liquefaction potential is greatest where the groundwater level is shallow, and submerged loose, fine sands occur within a depth of about 15 meters (50 feet) or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

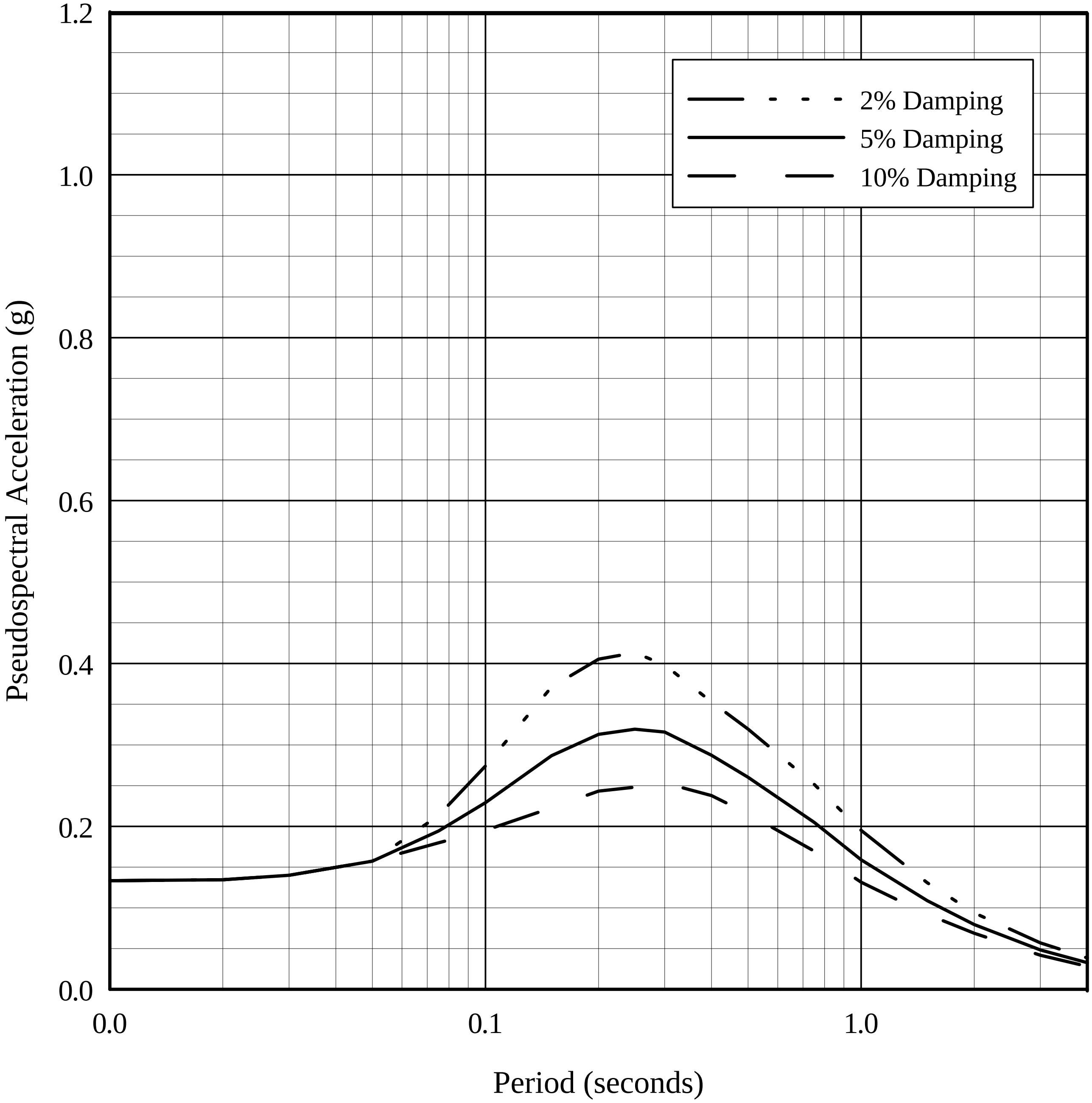
The site is located within a State of California designated Liquefaction Hazard Zone based on maps published by the California Geological Survey. However, we have performed a

Table 2, Site-Specific Response Spectra
Pseudospectral acceleration in g

	2% Damping		5% Damping		10% Damping	
Period in Seconds	10% in 50 Years	10% in 5 Years	10% in 50 Years	10% in 5 Years	10% in 50 Years	10% in 5 Years
0.01	0.37	0.13	0.37	0.13	0.37	0.13
0.02	0.37	0.13	0.37	0.13	0.37	0.13
0.03	0.39	0.14	0.39	0.14	0.39	0.14
0.05	0.44	0.16	0.44	0.16	0.44	0.16
0.075	0.59	0.21	0.53	0.19	0.49	0.18
0.10	0.75	0.27	0.63	0.23	0.53	0.20
0.15	0.98	0.37	0.76	0.29	0.59	0.22
0.20	1.05	0.41	0.81	0.31	0.63	0.24
0.25	1.07	0.41	0.82	0.32	0.64	0.25
0.30	1.03	0.40	0.82	0.32	0.66	0.25
0.40	0.94	0.35	0.77	0.29	0.63	0.24
0.50	0.87	0.32	0.71	0.26	0.59	0.22
0.75	0.71	0.25	0.58	0.21	0.48	0.17
1.00	0.58	0.20	0.47	0.16	0.39	0.13
1.50	0.40	0.13	0.33	0.11	0.28	0.09
2.00	0.30	0.09	0.25	0.08	0.22	0.07
3.00	0.19	0.06	0.16	0.05	0.14	0.04
4.00	0.14	0.04	0.12	0.03	0.10	0.03

By: ET 9/17/12
Chkd: MM 9/19/12

Site specific response spectrum used in ETABS model analysis



NOTES: Probabilistic spectrum was computed for a ground motion level with a 10% probability of being exceeded in 5 years.

Prepared/Date: ET 9/18/12
Checked/Date: MM 9/18/12

Proposed HAZUS Evaluation Existing Belmont Plaza Olmpic Pool Long Beach, California	amec	HORIZONTAL RESPONSE SPECTRA Site-Specific Probabilistic Response Spectra Project No. 4953-12-0931 Figure 2.1

(d) Seismic Coefficients

BELMONT PLAZA BEACH CENTER - Community Center Building (10% in 5 Yr Earthquake)
Seismic Coefficient per Section 2.4.3 of Chapter 6, CBC 2010

Community Center Building
Base Shear V=Cs*W

$$C_s = \frac{0.8 A_v S}{R T^{2/3}}$$ (Equation 2-4)

$$C_s \le 2.12 \frac{A_a}{R}$$ (Equation 2-5)

Location of Facility is in Long Beach City, County of LA

Av=	0.11	Per Geotech Report
Aa=	0.13	Per Geotech Report
S=	1.2	Per Geotech Report

Ta= $$\frac{0.05\, h_n}{\sqrt{L}}$$

hn=	31.5	feet
L=	130	feet
Ta=	0.138	second
R=	4.5	Reinforced concrete shear wall, Table 2.4.3.1 of Chapter 6, CBC 2010
Cd=	4	
Cs=	0.088	
Cs<=	0.061	Control,

BELMONT PLAZA BEACH CENTER - Community Center Building (10% in 50 Yr Earthquake)
Seismic Coefficient per Section 2.4.3 of Chapter 6, CBC 2010

Community Center Building

Base Shear V=Cs*W

$$C_s = \frac{0.8 A_v S}{R T^{2/3}}$$ (Equation 2-4)

$$C_s \leq 2.12 \frac{A_a}{R}$$ (Equation 2-5)

Location of Facility is in Long Beach City, County of LA

Av=	0.4	From Figure 2.1
Aa=	0.4	From Figure 2.1
S=	1.5	Site Coefficient from Table 2-1

Ta= $$\frac{0.05\, h_n}{\sqrt{L}}$$

hn=	31.5	feet
L=	130	feet
Ta=	0.138	second
R=	4.5	Reinforced concrete shear wall, Table 2.4.3.1 of Chapter 6, CBC 2010
Cd=	4	
Cs=	0.399	
Cs<=	0.188	Control,

BELMONT PLAZA BEACH CENTER - Natatorium Building (10% in 5 Yr Earthquake)
Seismic Coefficient per Section 2.4.3 of Chapter 6, CBC 2010

Community Center Building
Base Shear V=Cs*W

$$\boxed{C_s = \frac{0.8 A_v S}{RT^{2/3}}}$$ (Equation 2-4)

$$\boxed{C_s \leq 2.12 \frac{A_a}{R}}$$ (Equation 2-5)

Location of Facility is in Long Beach City, County of LA

Av=	0.11	Per Geotech Report
Aa=	0.13	Per Geotech Report
S=	1.2	Per Geotech Report

Ta= $C_t h_n^{(3/4)}$ = $0.03 h_n^{(3/4)}$ = 0.56

hn=	50	feet
L=	146	feet
Ta=	0.56	second
R=	2	Ordinary concrete moment frame, Table 2.4.3.1 of Chapter 6, CBC 2010
Cd=	2	
Cs=	0.077	Control,
Cs<=	0.138	

BELMONT PLAZA BEACH CENTER - Natatorium Building (10% in 50 Yr Earthquake)
Seismic Coefficient per Section 2.4.3 of Chapter 6, CBC 2010

Community Center Building

Base Shear V=Cs*W

$$\boxed{C_s = \frac{0.8 A_v S}{RT^{2/3}}}$$ (Equation 2-4)

$$\boxed{C_s \leq 2.12 \frac{A_a}{R}}$$ (Equation 2-5)

Location of Facility is in Long Beach City, County of LA

Av=	0.4	From Figure 2.1
Aa=	0.4	From Figure 2.1
S=	1.5	Site Coefficient from Table 2-1

Ta= $C_t h_n^{(3/4)}$ = $0.03 h_n^{(3/4)}$ = 0.56

hn=	50	feet
L=	146	feet
Ta=	0.56	second
R=	2	Ordinary concrete moment frame, Table 2.4.3.1 of Chapter 6, CBC 2010
Cd=	2	
Cs=	0.352	Control,
Cs<=	0.424	

BELMONT PLAZA BEACH CENTER - Locker Building (10% in 5 Yr Earthquake)
Seismic Coefficient per Section 2.4.3 of Chapter 6, CBC 2010

Community Center Building

Base Shear V=Cs*W

$$C_s = \frac{0.8 A_v S}{R T^{2/3}}$$ (Equation 2-4)

$$C_s \leq 2.12 \frac{A_a}{R}$$ (Equation 2-5)

Location of Facility is in Long Beach City, County of LA

Av=	0.11	Per Geotech Report
Aa=	0.13	Per Geotech Report
S=	1.2	Per Geotech Report

$$T_a = \frac{0.05 h_n}{\sqrt{L}}$$

hn=	12.5	feet
L=	146	feet
Ta=	0.052	second
R=	4.5	Reinforced concrete shear wall, Table 2.4.3.1 of Chapter 6, CBC 2010
Cd=	4	
Cs=	0.169	
Cs<=	0.061	Control,

BELMONT PLAZA BEACH CENTER - Locker Building (10% in 50 Yr Earthquake)
Seismic Coefficient per Section 2.4.3 of Chapter 6, CBC 2010

Community Center Building

Base Shear V=Cs*W

$$C_s = \frac{0.8 A_v S}{R T^{2/3}}$$ (Equation 2-4)

$$C_s \leq 2.12 \frac{A_a}{R}$$ (Equation 2-5)

Location of Facility is in Long Beach City, County of LA

Av=	0.4	From Figure 2.1
Aa=	0.4	From Figure 2.1
S=	1.5	Site Coefficient from Table 2-1

Ta= $$\frac{0.05\, h_n}{\sqrt{L}}$$

hn=	12.5	feet
L=	146	feet
Ta=	0.052	second
R=	4.5	Reinforced concrete shear wall, Table 2.4.3.1 of Chapter 6, CBC 2010
Cd=	4	
Cs=	0.768	
Cs<=	0.188	Control,

S T O R Y D A T A

STORY	SIMILAR TO	HEIGHT	ELEVATION
RF	None	16.000	45.000
2ND	None	15.000	29.000
1ST	None	9.000	14.000
BASE	None		5.000

S T A T I C   L O A D   C A S E S

STATIC CASE	CASE TYPE	AUTO LAT LOAD	SELF WT MULTIPLIER	NOTIONAL FACTOR	NOTIONAL DIRECTION
D	DEAD	N/A	1.0000		
L	LIVE	N/A	0.0000		
EX	QUAKE	USER_COEFF	0.0000		
EY	QUAKE	USER_COEFF	0.0000		

R E S P O N S E   S P E C T R U M   C A S E S

RESP SPEC CASE: QXUS

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	SS50	32.2000
U2	----	N/A
UZ	----	N/A

RESP SPEC CASE: QYUS

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	----	N/A
U2	SS50	32.2000
UZ	----	N/A

RESP SPEC CASE: QX

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	SS50	28.2540
U2	----	N/A
UZ	----	N/A

RESP SPEC CASE: QY

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	----	N/A
U2	SS50	34.7300
UZ	----	N/A

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EX

AUTO SEISMIC INPUT DATA

Direction: X
Typical Eccentricity = 5%
Eccentricity Overrides: No

Period Calculation: Program Calculated
Ct = 0.035 (in feet units)

Top Story: RF
Bottom Story: 1ST

C = 0.188
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 4240.72

V Used = 0.1880W = 797.26

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
RF	461.61	0.00	0.00	0.000	0.000	0.000
2ND	335.64	0.00	0.00	0.000	-502.449	0.000
1ST	0.00	0.00	0.00	0.000	0.000	0.000

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EY

AUTO SEISMIC INPUT DATA

Direction: Y
Typical Eccentricity = 5%
Eccentricity Overrides: No

Period Calculation: Program Calculated
Ct = 0.035 (in feet units)

Top Story: RF
Bottom Story: 1ST

C = 0.188
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 4240.72

V Used = 0.1880W = 797.26

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
RF	0.00	461.61	0.00	0.000	0.000	0.000
2ND	0.00	335.64	0.00	502.449	0.000	0.000
1ST	0.00	0.00	0.00	0.000	0.000	0.000

M A S S   S O U R C E   D A T A

MASS FROM	LATERAL MASS ONLY	LUMP MASS AT STORIES
Masses	Yes	Yes

D I A P H R A G M   M A S S   D A T A

STORY	DIAPHRAGM	MASS-X	MASS-Y	MMI	X-M	Y-M
RF	D1	5.267E+01	5.267E+01	1.136E+05	37.597	72.103
2ND	D1	6.837E+01	6.837E+01	1.462E+05	34.965	74.511
2ND	D2	1.077E+01	1.077E+01	7.397E+03	91.209	71.745

A S S E M B L E D   P O I N T   M A S S E S

STORY	UX	UY	UZ	RX	RY	RZ
RF	5.267E+01	5.267E+01	0.000E+00	0.000E+00	0.000E+00	1.136E+05
2ND	7.914E+01	7.914E+01	0.000E+00	0.000E+00	0.000E+00	1.536E+05
1ST	3.052E+01	3.052E+01	0.000E+00	0.000E+00	0.000E+00	0.000E+00
BASE	9.106E+00	9.106E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
Totals	1.714E+02	1.714E+02	0.000E+00	0.000E+00	0.000E+00	2.671E+05

C E N T E R S   O F   C U M U L A T I V E   M A S S   &   C E N T E R S   O F   R I G I D I T Y

STORY LEVEL	DIAPHRAGM NAME	CENTER OF MASS			CENTER OF RIGIDITY	
		MASS	ORDINATE-X	ORDINATE-Y	ORDINATE-X	ORDINATE-Y
RF	D1	5.267E+01	37.597	72.103	16.635	79.508
2ND	D1	1.210E+02	36.110	73.464	54.871	86.664
2ND	D2	1.077E+01	91.209	71.745	91.760	63.569

M O D A L   P E R I O D S   A N D   F R E Q U E N C I E S

MODE NUMBER	PERIOD (TIME)	FREQUENCY (CYCLES/TIME)	CIRCULAR FREQ (RADIANS/TIME)
Mode 1	0.27835	3.59254	22.57261
Mode 2	0.27165	3.68122	23.12976
Mode 3	0.25833	3.87096	24.32197
Mode 4	0.25264	3.95820	24.87009
Mode 5	0.25059	3.99058	25.07353
Mode 6	0.22391	4.46602	28.06086
Mode 7	0.16290	6.13877	38.57106
Mode 8	0.16279	6.14277	38.59614
Mode 9	0.16079	6.21924	39.07662
Mode 10	0.15607	6.40757	40.25995
Mode 11	0.15289	6.54083	41.09727
Mode 12	0.15234	6.56417	41.24391
Mode 13	0.13976	7.15495	44.95585
Mode 14	0.13811	7.24078	45.49518
Mode 15	0.12608	7.93161	49.83576
Mode 16	0.11274	8.86992	55.73135
Mode 17	0.11126	8.98828	56.47505
Mode 18	0.10736	9.31428	58.52332
Mode 19	0.10706	9.34093	58.69081
Mode 20	0.08792	11.37421	71.46629
Mode 21	0.08704	11.48836	72.18350
Mode 22	0.08676	11.52637	72.42229
Mode 23	0.08594	11.63645	73.11399
Mode 24	0.08003	12.49541	78.51097
Mode 25	0.07880	12.69115	79.74085
Mode 26	0.07303	13.69321	86.03696
Mode 27	0.07245	13.80174	86.71890
Mode 28	0.06990	14.30624	89.88876
Mode 29	0.06250	16.00091	100.53667
Mode 30	0.05760	17.35981	109.07488
Mode 31	0.05697	17.55173	110.28079
Mode 32	0.05687	17.58337	110.47959
Mode 33	0.05308	18.84107	118.38192
Mode 34	0.04955	20.18203	126.80742
Mode 35	0.04363	22.92112	144.01766
Mode 36	0.04358	22.94659	144.17770
Mode 37	0.03725	26.84632	168.68037
Mode 38	0.03446	29.01634	182.31504
Mode 39	0.02982	33.53126	210.68314
Mode 40	0.02951	33.88590	212.91139
Mode 41	0.02924	34.20119	214.89239
Mode 42	0.02830	35.34144	222.05679
Mode 43	0.02784	35.91513	225.66140
Mode 44	0.02768	36.12944	227.00796
Mode 45	0.02586	38.67137	242.97941
Mode 46	0.02533	39.47306	248.01655
Mode 47	0.02485	40.23832	252.82480
Mode 48	0.02121	47.15461	296.28115
Mode 49	0.02082	48.02218	301.73224
Mode 50	0.02029	49.28353	309.65758

M O D A L   P A R T I C I P A T I N G   M A S S   R A T I O S

MODE NUMBER	X-TRANS %MASS <SUM>	Y-TRANS %MASS <SUM>	Z-TRANS %MASS <SUM>	RX-ROTN %MASS <SUM>	RY-ROTN %MASS <SUM>	RZ-ROTN %MASS <SUM>
Mode 1	0.00 < 0>	1.05 < 1>	0.00 < 0>	1.33 < 1>	0.00 < 0>	0.02 < 0>
Mode 2	0.02 < 0>	37.24 < 38>	0.00 < 0>	69.13 < 70>	0.02 < 0>	1.41 < 1>
Mode 3	0.00 < 0>	2.65 < 41>	0.00 < 0>	3.83 < 74>	0.00 < 0>	0.08 < 2>
Mode 4	0.00 < 0>	0.34 < 41>	0.00 < 0>	0.29 < 75>	0.00 < 0>	0.00 < 2>
Mode 5	0.00 < 0>	0.01 < 41>	0.00 < 0>	0.01 < 75>	0.00 < 0>	0.00 < 2>
Mode 6	0.09 < 0>	0.01 < 41>	0.00 < 0>	0.01 < 75>	0.04 < 0>	0.01 < 2>
Mode 7	55.74 < 56>	0.38 < 42>	0.00 < 0>	0.25 < 75>	73.08 < 73>	8.26 < 10>
Mode 8	1.04 < 57>	0.01 < 42>	0.00 < 0>	0.01 < 75>	1.80 < 75>	0.28 < 10>
Mode 9	2.96 < 60>	0.01 < 42>	0.00 < 0>	0.01 < 75>	4.66 < 80>	0.18 < 10>
Mode 10	0.06 < 60>	0.00 < 42>	0.00 < 0>	0.00 < 75>	0.20 < 80>	0.21 < 10>
Mode 11	0.00 < 60>	0.01 < 42>	0.00 < 0>	0.00 < 75>	0.03 < 80>	0.14 < 11>
Mode 12	0.20 < 60>	0.00 < 42>	0.00 < 0>	0.00 < 75>	0.49 < 80>	0.00 < 11>
Mode 13	0.03 < 60>	0.03 < 42>	0.00 < 0>	0.00 < 75>	0.04 < 80>	0.00 < 11>
Mode 14	0.02 < 60>	0.02 < 42>	0.00 < 0>	0.01 < 75>	0.02 < 80>	0.04 < 11>
Mode 15	5.03 < 65>	5.43 < 47>	0.00 < 0>	3.87 < 79>	8.17 < 89>	59.21 < 70>
Mode 16	0.63 < 66>	0.00 < 47>	0.00 < 0>	0.00 < 79>	0.10 < 89>	0.01 < 70>
Mode 17	0.01 < 66>	0.00 < 47>	0.00 < 0>	0.00 < 79>	0.00 < 89>	0.00 < 70>
Mode 18	0.00 < 66>	0.21 < 47>	0.00 < 0>	0.03 < 79>	0.00 < 89>	0.11 < 70>
Mode 19	0.00 < 66>	0.64 < 48>	0.00 < 0>	0.09 < 79>	0.00 < 89>	0.65 < 71>
Mode 20	0.27 < 66>	0.04 < 48>	0.00 < 0>	0.01 < 79>	0.06 < 89>	0.11 < 71>
Mode 21	0.03 < 66>	0.50 < 49>	0.00 < 0>	0.09 < 79>	0.01 < 89>	0.39 < 71>
Mode 22	0.24 < 66>	0.00 < 49>	0.00 < 0>	0.00 < 79>	0.05 < 89>	0.11 < 71>
Mode 23	0.02 < 66>	0.54 < 49>	0.00 < 0>	0.11 < 79>	0.00 < 89>	0.29 < 72>
Mode 24	0.00 < 66>	0.67 < 50>	0.00 < 0>	0.26 < 79>	0.00 < 89>	0.07 < 72>
Mode 25	0.00 < 66>	0.66 < 50>	0.00 < 0>	0.26 < 80>	0.00 < 89>	0.06 < 72>
Mode 26	18.51 < 85>	0.11 < 51>	0.00 < 0>	0.02 < 80>	7.33 < 96>	0.95 < 73>
Mode 27	0.66 < 86>	31.75 < 82>	0.00 < 0>	16.46 < 96>	0.20 < 96>	1.26 < 74>
Mode 28	0.13 < 86>	2.70 < 85>	0.00 < 0>	1.40 < 97>	0.03 < 96>	0.56 < 74>
Mode 29	0.01 < 86>	0.00 < 85>	0.00 < 0>	0.04 < 98>	0.00 < 96>	0.04 < 74>
Mode 30	0.00 < 86>	0.05 < 85>	0.00 < 0>	0.00 < 98>	0.00 < 96>	0.00 < 74>
Mode 31	0.17 < 86>	0.00 < 85>	0.00 < 0>	0.00 < 98>	0.04 < 96>	0.00 < 74>
Mode 32	0.01 < 86>	0.00 < 85>	0.00 < 0>	0.00 < 98>	0.00 < 96>	0.01 < 74>
Mode 33	0.00 < 86>	0.00 < 85>	0.00 < 0>	0.00 < 98>	0.00 < 96>	0.00 < 74>
Mode 34	0.21 < 86>	2.40 < 87>	0.00 < 0>	1.46 < 99>	0.09 < 96>	5.85 < 80>
Mode 35	0.08 < 86>	0.05 < 88>	0.00 < 0>	0.02 < 99>	0.09 < 97>	0.06 < 80>
Mode 36	0.74 < 87>	0.44 < 88>	0.00 < 0>	0.21 < 99>	0.86 < 97>	0.51 < 81>
Mode 37	6.72 < 94>	0.08 < 88>	0.00 < 0>	0.05 < 99>	1.30 < 99>	0.06 < 81>
Mode 38	0.00 < 94>	0.01 < 88>	0.00 < 0>	0.00 < 99>	0.00 < 99>	0.21 < 81>
Mode 39	4.08 < 98>	0.02 < 88>	0.00 < 0>	0.04 < 99>	0.93 <100>	0.16 < 81>
Mode 40	0.08 < 98>	0.88 < 89>	0.00 < 0>	0.04 < 99>	0.02 <100>	0.68 < 82>
Mode 41	0.00 < 98>	0.02 < 89>	0.00 < 0>	0.00 < 99>	0.00 <100>	0.01 < 82>
Mode 42	0.03 < 98>	0.19 < 89>	0.00 < 0>	0.00 < 99>	0.00 <100>	0.02 < 82>
Mode 43	0.14 < 98>	0.02 < 89>	0.00 < 0>	0.00 < 99>	0.01 <100>	0.08 < 82>
Mode 44	0.00 < 98>	0.15 < 89>	0.00 < 0>	0.00 < 99>	0.00 <100>	0.02 < 82>
Mode 45	0.07 < 98>	8.07 < 97>	0.00 < 0>	0.45 <100>	0.02 <100>	12.19 < 94>
Mode 46	0.30 < 98>	0.10 < 97>	0.00 < 0>	0.03 <100>	0.07 <100>	2.62 < 97>
Mode 47	0.04 < 98>	0.77 < 98>	0.00 < 0>	0.01 <100>	0.01 <100>	0.01 < 97>
Mode 48	0.00 < 98>	0.01 < 98>	0.00 < 0>	0.01 <100>	0.00 <100>	0.24 < 97>
Mode 49	0.00 < 98>	0.00 < 98>	0.00 < 0>	0.00 <100>	0.00 <100>	0.02 < 97>
Mode 50	0.00 < 98>	0.01 < 98>	0.00 < 0>	0.00 <100>	0.00 <100>	0.05 < 97>

M O D A L   L O A D   P A R T I C I P A T I O N   R A T I O S
(STATIC AND DYNAMIC RATIOS ARE IN PERCENT)

TYPE	NAME	STATIC	DYNAMIC
Load	D	0.0105	0.0000
Load	L	0.0007	0.0000
Load	EX	100.0076	97.9873
Load	EY	99.9977	97.9783
Load	EXP	100.0164	97.9037
Load	EXM	99.9978	97.9086
Load	EYP	99.9970	97.9698
Load	EYM	99.9981	97.9774
Accel	UX	100.0528	98.3679
Accel	UY	99.9939	98.2771
Accel	UZ	0.0000	0.0000
Accel	RX	88.1448	99.8670
Accel	RY	113.2229	99.7770
Accel	RZ	3764.3553	97.2820

TOTAL REACTIVE FORCES (RECOVERED LOADS) AT ORIGIN

LOAD	FX	FY	FZ	MX	MY	MZ
D	-9.087E-12	1.696E-11	5.358E+03	3.959E+05	-2.615E+05	8.738E-07
L	-1.650E-12	2.880E-12	1.215E+03	8.864E+04	-6.037E+04	-4.337E-07
EX	-7.973E+02	-1.432E-11	2.098E-14	3.303E-10	-3.000E+04	5.817E+04
EY	-2.885E-12	-7.973E+02	-2.203E-13	3.000E+04	1.902E-08	-3.166E+04
EXP	-7.973E+02	-1.777E-11	2.291E-13	-1.016E-09	-3.000E+04	6.310E+04
EXM	-7.973E+02	-2.631E-11	-1.298E-13	-4.229E-10	-3.000E+04	5.323E+04
EYP	-1.915E-12	-7.973E+02	1.403E-13	3.000E+04	-1.926E-08	-3.468E+04
EYM	-3.753E-12	-7.973E+02	3.091E-13	3.000E+04	-2.354E-08	-2.864E+04
QXUS	9.471E+02	1.011E+02	2.770E-13	2.911E+03	3.574E+04	6.586E+04
QYUS	1.011E+02	7.626E+02	3.393E-13	3.023E+04	3.944E+03	3.994E+04
QX	8.310E+02	8.874E+01	2.461E-13	2.554E+03	3.136E+04	5.779E+04
QY	1.091E+02	8.225E+02	3.658E-13	3.260E+04	4.254E+03	4.308E+04

S T O R Y   F O R C E S

STORY	LOAD	P	VX	VY	T	MX	MY
RF	EX	6.088E-09	-4.619E+02	6.829E-03	3.330E+04	-1.093E-01	-7.390E+03
2ND	EX	-7.290E-06	-7.977E+02	2.683E-02	5.820E+04	-5.292E-01	-1.885E+04
1ST	EX	3.571E+02	-5.337E+02	-7.542E+01	3.140E+04	2.826E+04	-3.060E+04
RF	EY	6.334E-09	6.213E-03	-4.623E+02	-1.738E+04	7.397E+03	9.941E-02
2ND	EY	8.987E-06	2.870E-02	-7.982E+02	-3.170E+04	1.887E+04	5.375E-01
1ST	EY	-1.128E+02	1.532E+01	-4.618E+02	-2.829E+04	8.368E+02	5.816E+03
RF	EXP	5.610E-09	-4.619E+02	1.167E-02	3.619E+04	-1.866E-01	-7.390E+03
2ND	EXP	-7.619E-06	-7.977E+02	2.196E-02	6.313E+04	-5.281E-01	-1.885E+04
1ST	EXP	3.558E+02	-5.199E+02	-5.010E+01	3.439E+04	2.922E+04	-3.019E+04
RF	EXM	6.566E-09	-4.619E+02	1.993E-03	3.041E+04	-3.188E-02	-7.390E+03
2ND	EXM	-6.961E-06	-7.977E+02	3.170E-02	5.326E+04	-5.304E-01	-1.885E+04
1ST	EXM	3.583E+02	-5.474E+02	-1.007E+02	2.842E+04	2.730E+04	-3.101E+04
RF	EYP	6.627E-09	3.993E-03	-4.623E+02	-1.916E+04	7.397E+03	6.389E-02
2ND	EYP	9.192E-06	2.237E-02	-7.982E+02	-3.471E+04	1.887E+04	4.116E-01
1ST	EYP	-1.120E+02	6.730E+00	-4.775E+02	-3.010E+04	2.354E+02	5.565E+03
RF	EYM	6.039E-09	8.433E-03	-4.623E+02	-1.561E+04	7.397E+03	1.349E-01
2ND	EYM	8.781E-06	3.503E-02	-7.982E+02	-2.868E+04	1.887E+04	6.634E-01
1ST	EYM	-1.135E+02	2.390E+01	-4.462E+02	-2.649E+04	1.438E+03	6.066E+03
RF	QXUS	9.619E-09	5.916E+02	1.492E+01	4.032E+04	2.387E+02	9.466E+03
2ND	QXUS	8.430E-06	9.094E+02	1.024E+02	6.317E+04	1.521E+03	2.266E+04
1ST	QXUS	4.341E+02	6.866E+02	1.618E+02	3.577E+04	3.212E+04	3.819E+04
RF	QYUS	8.766E-09	7.426E+01	5.943E+02	2.948E+04	9.509E+03	1.188E+03
2ND	QYUS	9.727E-06	9.687E+01	7.395E+02	3.825E+04	1.989E+04	2.569E+03
1ST	QYUS	1.277E+02	7.555E+01	4.788E+02	3.151E+04	6.732E+03	7.284E+03
RF	QX	8.440E-09	5.191E+02	1.309E+01	3.537E+04	2.094E+02	8.306E+03
2ND	QX	7.397E-06	7.980E+02	8.989E+01	5.543E+04	1.335E+03	1.989E+04
1ST	QX	3.809E+02	6.025E+02	1.419E+02	3.138E+04	2.819E+04	3.351E+04
RF	QY	9.455E-09	8.009E+01	6.410E+02	3.179E+04	1.026E+04	1.281E+03
2ND	QY	1.049E-05	1.045E+02	7.976E+02	4.125E+04	2.145E+04	2.770E+03
1ST	QY	1.377E+02	8.149E+01	5.165E+02	3.399E+04	7.261E+03	7.856E+03

STORY DRIFTS

STORY	DIRECTION	LOAD	MAX DRIFT
RF	X	EX	1/4460
2ND	X	EX	1/5172
1ST	X	EX	1/5998
RF	Y	EY	1/988
2ND	Y	EY	1/1487
1ST	Y	EY	1/8004
RF	X	EXP	1/4317
2ND	X	EXP	1/5907
1ST	X	EXP	1/6713
RF	X	EXM	1/4164
2ND	X	EXM	1/4600
1ST	X	EXM	1/5420
RF	Y	EYP	1/975
2ND	Y	EYP	1/1472
1ST	Y	EYP	1/8301
RF	Y	EYM	1/1003
2ND	Y	EYM	1/1502
1ST	Y	EYM	1/7727
RF	X	QXUS	1/2889
2ND	X	QXUS	1/864
1ST	X	QXUS	1/656
RF	Y	QYUS	1/753
2ND	Y	QYUS	1/192
1ST	Y	QYUS	1/2332
RF	X	QX	1/3293
2ND	X	QX	1/985
1ST	X	QX	1/748
RF	Y	QY	1/698
2ND	Y	QY	1/178
1ST	Y	QY	1/2162

DISPLACEMENTS AT DIAPHRAGM CENTER OF MASS

STORY	DIAPHRAGM	LOAD	UX	UY	RZ
RF	D1	EX	0.0058	-0.0002	0.00001
2ND	D1	EX	0.0023	-0.0002	0.00001
2ND	D2	EX	0.0006	0.0001	0.00000
RF	D1	EY	-0.0001	0.0169	0.00002
2ND	D1	EY	-0.0003	0.0021	-0.00002
2ND	D2	EY	0.0000	0.0004	-0.00001
RF	D1	EXP	0.0057	-0.0003	0.00000
2ND	D1	EXP	0.0022	-0.0001	0.00001
2ND	D2	EXP	0.0005	0.0000	0.00000
RF	D1	EXM	0.0059	0.0000	0.00002
2ND	D1	EXM	0.0023	-0.0003	0.00002
2ND	D2	EXM	0.0006	0.0001	0.00000
RF	D1	EYP	-0.0001	0.0170	0.00002
2ND	D1	EYP	-0.0002	0.0021	-0.00001
2ND	D2	EYP	0.0000	0.0004	-0.00001
RF	D1	EYM	-0.0002	0.0168	0.00001
2ND	D1	EYM	-0.0003	0.0022	-0.00002
2ND	D2	EYM	0.0000	0.0004	-0.00001
RF	D1	QXUS	0.0075	0.0006	0.00006
2ND	D1	QXUS	0.0030	0.0010	0.00004
2ND	D2	QXUS	0.0008	0.0001	0.00001
RF	D1	QYUS	0.0008	0.0210	0.00005
2ND	D1	QYUS	0.0004	0.0021	0.00002
2ND	D2	QYUS	0.0001	0.0004	0.00001
RF	D1	QX	0.0066	0.0005	0.00005
2ND	D1	QX	0.0026	0.0009	0.00004
2ND	D2	QX	0.0007	0.0001	0.00001
RF	D1	QY	0.0009	0.0227	0.00005
2ND	D1	QY	0.0004	0.0023	0.00003
2ND	D2	QY	0.0001	0.0004	0.00001

Diaphragm Displacement Roof under Qx load = 0.0066*12"*4=0.317"
Qy load = 0.0227*12"*4=1.089"

Cd=4

Diaphragm Displacement 2nd floor under Qx load = 0.0026*12"*4=0.125"
Qy load = 0.0023*12"*4=0.110"

Above displacements are based on seismic coefficient of 0.188 (10% in 50 year earthquake);
for 10% in 5 year earthquake, seismic coefficient is 0.061, 0.061/0.188=0.324, modify displacement

For 10% in 5 year earthquake;
Diaphragm Displacement Roof under Qx load = 0.317"*0.324=0.103"
Qy load = 1.089"*0.324=0.353"

Diaphragm Displacement 2nd floor under Qx load = 0.125"*0.324=0.041"
Qy load = 0.110"*0.324=0.036"

S T O R Y D A T A

STORY	SIMILAR TO	HEIGHT	ELEVATION
RF	None	49.000	66.000
1ST	None	12.000	17.000
BASE	None		5.000

S T A T I C   L O A D   C A S E S

STATIC CASE	CASE TYPE	AUTO LAT LOAD	SELF WT MULTIPLIER	NOTIONAL FACTOR	NOTIONAL DIRECTION
EX	QUAKE	USER_COEFF	0.0000		
EY	QUAKE	USER_COEFF	0.0000		
EXP	QUAKE	USER_COEFF	0.0000		
EXM	QUAKE	USER_COEFF	0.0000		
EYP	QUAKE	USER_COEFF	0.0000		
EYM	QUAKE	USER_COEFF	0.0000		

R E S P O N S E   S P E C T R U M   C A S E S

RESP SPEC CASE: QX

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	SS50	15.5680
U2	----	N/A
UZ	----	N/A

RESP SPEC CASE: QY

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	----	N/A
U2	SS50	18.2570
UZ	----	N/A

RESP SPEC CASE: QXUNSCALED

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	SS50	32.2000
U2	----	N/A
UZ	----	N/A

RESP SPEC CASE: QYUNSCALED

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	----	N/A
U2	SS50	32.2000
UZ	----	N/A

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EX

AUTO SEISMIC INPUT DATA

Direction: X
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: 1ST

C = 0.1
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 7137.48

V Used = 0.1000W = 713.75

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
RF	713.75	0.00	0.00	0.000	-179.759	0.000
1ST	0.00	0.00	0.00	0.000	0.000	0.000

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EY

AUTO SEISMIC INPUT DATA

Direction: Y
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: 1ST

C = 0.1
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 7137.48

V Used = 0.1000W = 713.75

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
RF	0.00	713.75	0.00	179.759	0.000	0.000
1ST	0.00	0.00	0.00	0.000	0.000	0.000

A U T O S E I S M I C U S E R C O E F F I C I E N T
Case: EXP

AUTO SEISMIC INPUT DATA

Direction: X + EccY
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: 1ST

C = 0.1
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 7137.48

V Used = 0.1000W = 713.75

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
RF	713.75	0.00	0.00	0.000	-179.759	-5194.291
1ST	0.00	0.00	0.00	0.000	0.000	0.000

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EXM

AUTO SEISMIC INPUT DATA

Direction: X - EccY
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: 1ST

C = 0.1
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 7137.48

V Used = 0.1000W = 713.75

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
RF	713.75	0.00	0.00	0.000	-179.759	5194.291
1ST	0.00	0.00	0.00	0.000	0.000	0.000

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EYP

AUTO SEISMIC INPUT DATA

Direction: Y + EccX
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: 1ST

C = 0.1
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 7137.48

V Used = 0.1000W = 713.75

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
RF	0.00	713.75	0.00	179.759	0.000	7864.227
1ST	0.00	0.00	0.00	0.000	0.000	0.000

A U T O S E I S M I C U S E R C O E F F I C I E N T
Case: EYM

AUTO SEISMIC INPUT DATA

Direction: Y - EccX
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: 1ST

C = 0.1
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 7137.48

V Used = 0.1000W = 713.75

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
RF	0.00	713.75	0.00	179.759	0.000	-7864.227
1ST	0.00	0.00	0.00	0.000	0.000	0.000

M A S S   S O U R C E   D A T A

MASS FROM	LATERAL MASS ONLY	LUMP MASS AT STORIES
Masses	Yes	Yes

D I A P H R A G M   M A S S   D A T A

STORY	DIAPHRAGM	MASS-X	MASS-Y	MMI	X-M	Y-M
RF	D1	2.203E+02	2.203E+02	1.911E+06	189.747	72.917
RF	D2	1.510E+00	1.510E+00	1.555E+03	85.083	72.917
1ST	D1	3.038E+01	3.038E+01	3.788E+04	89.159	72.610

A S S E M B L E D   P O I N T   M A S S E S

STORY	UX	UY	UZ	RX	RY	RZ
RF	2.218E+02	2.218E+02	0.000E+00	0.000E+00	0.000E+00	1.912E+06
1ST	7.410E+01	7.410E+01	0.000E+00	0.000E+00	0.000E+00	3.788E+04
BASE	8.801E+00	8.801E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
Totals	3.047E+02	3.047E+02	0.000E+00	0.000E+00	0.000E+00	1.950E+06

C E N T E R S   O F   C U M U L A T I V E   M A S S   &   C E N T E R S   O F   R I G I D I T Y

STORY	DIAPHRAGM	/----------CENTER OF MASS----------			/--CENTER OF RIGIDITY--/	
LEVEL	NAME	MASS	ORDINATE-X	ORDINATE-Y	ORDINATE-X	ORDINATE-Y
RF	D1	2.203E+02	189.747	72.917	170.948	72.913
RF	D2	1.510E+00	85.083	72.917	80.469	72.830
1ST	D1	2.507E+02	177.557	72.879	94.546	72.417

M O D A L   P E R I O D S   A N D   F R E Q U E N C I E S

MODE NUMBER	PERIOD (TIME)	FREQUENCY (CYCLES/TIME)	CIRCULAR FREQ (RADIANS/TIME)
Mode 1	0.87786	1.13914	7.15741
Mode 2	0.75564	1.32338	8.31507
Mode 3	0.57365	1.74321	10.95293
Mode 4	0.06156	16.24471	102.06855
Mode 5	0.05094	19.63045	123.34174
Mode 6	0.04314	23.18254	145.66022
Mode 7	0.03619	27.63205	173.61732
Mode 8	0.03052	32.76788	205.88667
Mode 9	0.02635	37.94407	238.40960
Mode 10	0.00360	278.04815	1747.02807
Mode 11	0.00296	337.30718	2119.36351
Mode 12	0.00243	412.11455	2589.39211
Mode 13	0.00242	412.52000	2591.93959
Mode 14	0.00223	449.03049	2821.34179
Mode 15	0.00221	452.19499	2841.22492
Mode 16	0.00124	804.56792	5055.24931
Mode 17	0.00119	843.76542	5301.53450

M O D A L   P A R T I C I P A T I N G   M A S S   R A T I O S

MODE NUMBER	X-TRANS %MASS <SUM>	Y-TRANS %MASS <SUM>	Z-TRANS %MASS <SUM>	RX-ROTN %MASS <SUM>	RY-ROTN %MASS <SUM>	RZ-ROTN %MASS <SUM>
Mode 1	0.00 < 0>	85.73 < 86>	0.00 < 0>	97.10 < 97>	0.00 < 0>	5.71 < 6>
Mode 2	87.46 < 87>	0.00 < 86>	0.00 < 0>	0.00 < 97>	98.61 < 99>	0.00 < 6>
Mode 3	0.00 < 87>	2.22 < 88>	0.00 < 0>	2.40 <100>	0.00 < 99>	81.51 < 87>
Mode 4	3.45 < 91>	0.00 < 88>	0.00 < 0>	0.00 <100>	0.46 < 99>	0.00 < 87>
Mode 5	0.00 < 91>	0.00 < 88>	0.00 < 0>	0.00 <100>	0.00 < 99>	0.25 < 87>
Mode 6	9.09 <100>	0.00 < 88>	0.00 < 0>	0.00 <100>	0.93 <100>	0.00 < 87>
Mode 7	0.00 <100>	5.37 < 93>	0.00 < 0>	0.16 <100>	0.00 <100>	9.57 < 97>
Mode 8	0.00 <100>	5.83 < 99>	0.00 < 0>	0.22 <100>	0.00 <100>	2.11 < 99>
Mode 9	0.00 <100>	0.84 <100>	0.00 < 0>	0.12 <100>	0.00 <100>	0.84 <100>
Mode 10	0.00 <100>	0.00 <100>	0.00 < 0>	0.00 <100>	0.00 <100>	0.00 <100>
Mode 11	0.00 <100>	0.00 <100>	0.00 < 0>	0.00 <100>	0.00 <100>	0.00 <100>
Mode 12	0.00 <100>	0.00 <100>	0.00 < 0>	0.00 <100>	0.00 <100>	0.00 <100>
Mode 13	0.00 <100>	0.00 <100>	0.00 < 0>	0.00 <100>	0.00 <100>	0.00 <100>
Mode 14	0.00 <100>	0.00 <100>	0.00 < 0>	0.00 <100>	0.00 <100>	0.00 <100>
Mode 15	0.00 <100>	0.00 <100>	0.00 < 0>	0.00 <100>	0.00 <100>	0.00 <100>
Mode 16	0.00 <100>	0.00 <100>	0.00 < 0>	0.00 <100>	0.00 <100>	0.00 <100>
Mode 17	0.00 <100>	0.00 <100>	0.00 < 0>	0.00 <100>	0.00 <100>	0.00 <100>

M O D A L   L O A D   P A R T I C I P A T I O N   R A T I O S
(STATIC AND DYNAMIC RATIOS ARE IN PERCENT)

TYPE	NAME	STATIC	DYNAMIC
Load	D	0.0008	0.0000
Load	L	0.0009	0.0000
Load	EX	100.0000	100.0000
Load	EY	100.0000	100.0000
Load	EXP	100.0000	100.0000
Load	EXM	100.0000	100.0000
Load	EYP	100.0000	100.0000
Load	EYM	100.0000	100.0000
Accel	UX	100.0000	100.0000
Accel	UY	100.0000	100.0000
Accel	UZ	0.0000	0.0000
Accel	RX	92.4047	100.0000
Accel	RY	107.5815	100.0000
Accel	RZ	56.4616	100.0000

TOTAL REACTIVE FORCES (RECOVERED LOADS) AT ORIGIN

LOAD	FX	FY	FZ	MX	MY	MZ
D	2.128E-14	3.645E-13	9.849E+03	7.177E+05	-1.769E+06	6.708E-11
L	3.873E-13	2.279E-13	2.385E+03	1.738E+05	-4.241E+05	-2.237E-10
EX	-7.137E+02	4.274E-11	2.593E-13	-2.818E-09	-4.693E+04	5.204E+04
EY	5.241E-11	-7.137E+02	-3.755E-12	4.693E+04	4.305E-09	-1.349E+05
EXP	-7.137E+02	4.075E-11	-9.921E-13	-2.605E-09	-4.693E+04	5.724E+04
EXM	-7.137E+02	3.917E-11	8.300E-13	-2.561E-09	-4.693E+04	4.685E+04
EYP	5.177E-11	-7.137E+02	1.538E-12	4.693E+04	3.230E-09	-1.428E+05
EYM	5.518E-11	-7.137E+02	-8.775E-13	4.693E+04	3.911E-09	-1.271E+05
QX	7.205E+02	2.844E-01	1.287E-12	5.563E+00	4.736E+04	5.815E+04
QY	3.335E-01	7.329E+02	1.227E-12	4.810E+04	6.775E+00	1.567E+05
QXUNSCALED	1.490E+03	5.882E-01	2.693E-12	1.151E+01	9.796E+04	1.203E+05
QYUNSCALED	5.882E-01	1.293E+03	2.062E-12	8.483E+04	1.195E+01	2.764E+05

S T O R Y   F O R C E S

STORY	LOAD	P	VX	VY	T	MX	MY
RF	EX	2.487E-13	-7.192E+02	-1.611E-04	5.244E+04	7.844E-03	-3.506E+04
1ST	EX	-1.844E+01	-1.402E+01	1.221E-01	1.031E+03	-1.345E+03	7.437E+02
RF	EY	-2.231E-12	-1.866E-04	-7.210E+02	-1.364E+05	3.515E+04	-7.538E-03
1ST	EY	-5.646E-01	2.108E-02	-2.135E+02	-1.699E+04	6.330E+03	4.588E+01
RF	EXP	-2.913E-13	-7.192E+02	4.837E-02	5.767E+04	-2.451E+00	-3.506E+04
1ST	EXP	-1.846E+01	-1.402E+01	-7.828E+00	4.331E+02	-1.152E+03	7.455E+02
RF	EXM	-1.421E-14	-7.192E+02	-4.869E-02	4.721E+04	2.466E+00	-3.506E+04
1ST	EXM	-1.842E+01	-1.402E+01	8.072E+00	1.630E+03	-1.538E+03	7.420E+02
RF	EYP	1.918E-12	-1.686E-04	-7.211E+02	-1.443E+05	3.515E+04	-6.779E-03
1ST	EYP	-5.329E-01	1.934E-02	-2.013E+02	-1.604E+04	6.036E+03	4.326E+01
RF	EYM	-2.032E-12	-2.047E-04	-7.209E+02	-1.284E+05	3.514E+04	-8.296E-03
1ST	EYM	-5.963E-01	2.281E-02	-2.257E+02	-1.795E+04	6.625E+03	4.850E+01
RF	QX	3.822E-13	7.232E+02	1.074E-01	5.806E+04	5.389E+00	3.541E+04
1ST	QX	1.872E+01	6.198E+01	7.329E+00	4.699E+03	1.550E+03	1.126E+03
RF	QY	1.456E-12	5.900E-02	7.372E+02	1.580E+05	3.604E+04	2.812E+00
1ST	QY	5.753E-01	3.291E-01	2.226E+02	1.786E+04	6.474E+03	4.674E+01
RF	QXUNSCALED	6.791E-13	1.496E+03	2.221E-01	1.201E+05	1.115E+01	7.325E+04
1ST	QXUNSCALED	3.873E+01	1.282E+02	1.516E+01	9.718E+03	3.206E+03	2.329E+03
RF	QYUNSCALED	2.416E-12	1.041E-01	1.300E+03	2.787E+05	6.357E+04	4.959E+00
1ST	QYUNSCALED	1.015E+00	5.804E-01	3.926E+02	3.150E+04	1.142E+04	8.243E+01

STORY DRIFTS

STORY	DIRECTION	LOAD	MAX DRIFT
RF	X	EX	1/1052
1ST	X	EX	1/128788
RF	Y	EY	1/714
1ST	X	EY	1/32191
1ST	Y	EY	1/21598
RF	X	EXP	1/1015
1ST	X	EXP	1/108257
RF	X	EXM	1/1015
1ST	X	EXM	1/112120
RF	Y	EYP	1/670
1ST	X	EYP	1/34492
1ST	Y	EYP	1/22973
RF	Y	EYM	1/766
1ST	X	EYM	1/30178
1ST	Y	EYM	1/20378
RF	X	QX	1/1004
1ST	X	QX	1/71001
RF	Y	QY	1/605
1ST	Y	QY	1/21052
RF	X	QXUNSCALED	1/486
1ST	X	QXUNSCALED	1/34327
RF	Y	QYUNSCALED	1/343
1ST	Y	QYUNSCALED	1/11936

DISPLACEMENTS AT DIAPHRAGM CENTER OF MASS

STORY	DIAPHRAGM	LOAD	UX	UY	RZ
RF	D1	EX	0.0466	0.0000	0.00000
RF	D2	EX	0.0056	0.0000	0.00000
1ST	D1	EX	0.0001	0.0000	0.00000
RF	D1	EY	0.0000	0.0620	0.00006
RF	D2	EY	0.0000	0.0285	-0.00001
1ST	D1	EY	0.0000	0.0005	-0.00001
RF	D1	EXP	0.0466	-0.0004	-0.00002
RF	D2	EXP	0.0056	0.0011	0.00000
1ST	D1	EXP	0.0001	0.0000	0.00000
RF	D1	EXM	0.0466	0.0004	0.00002
RF	D2	EXM	0.0056	-0.0011	0.00000
1ST	D1	EXM	0.0001	0.0000	0.00000
RF	D1	EYP	0.0000	0.0627	0.00009
RF	D2	EYP	0.0000	0.0268	0.00000
1ST	D1	EYP	0.0000	0.0005	-0.00001
RF	D1	EYM	0.0000	0.0614	0.00002
RF	D2	EYM	0.0000	0.0301	-0.00001
1ST	D1	EYM	0.0000	0.0005	-0.00001
RF	D1	QX	0.0471	0.0004	0.00002
RF	D2	QX	0.0054	0.0011	0.00001
1ST	D1	QX	0.0001	0.0000	0.00000
RF	D1	QY	0.0000	0.0650	0.00016
RF	D2	QY	0.0000	0.0290	0.00002
1ST	D1	QY	0.0000	0.0005	0.00001
RF	D1	QXUNSCALED	0.0974	0.0009	0.00005
RF	D2	QXUNSCALED	0.0112	0.0022	0.00001
1ST	D1	QXUNSCALED	0.0003	0.0000	0.00000
RF	D1	QYUNSCALED	0.0000	0.1147	0.00028
RF	D2	QYUNSCALED	0.0000	0.0511	0.00003
1ST	D1	QYUNSCALED	0.0000	0.0009	0.00001

Diaphragm Displacement Roof under Qx load = 0.0471*12"*2=1.13"
Qy load = 0.0650*12"*2=1.56"

Cd=2

Above displacements are based on seismic coefficient of 0.1 (for 10% in 5 year earthquake)

S T O R Y   D A T A

STORY	SIMILAR TO	HEIGHT	ELEVATION
HIGH RF	None	3.500	34.000
RF	None	12.500	30.500
BASE	None		18.000

S T A T I C   L O A D   C A S E S

STATIC CASE	CASE TYPE	AUTO LAT LOAD	SELF WT MULTIPLIER	NOTIONAL FACTOR	NOTIONAL DIRECTION
EX	QUAKE	USER_COEFF	0.0000		
EY	QUAKE	USER_COEFF	0.0000		
EXP	QUAKE	USER_COEFF	0.0000		
EXM	QUAKE	USER_COEFF	0.0000		
EYP	QUAKE	USER_COEFF	0.0000		
EYM	QUAKE	USER_COEFF	0.0000		

R E S P O N S E   S P E C T R U M   C A S E S

RESP SPEC CASE: QX

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	IBC2006	6.9270
U2	----	N/A
UZ	----	N/A

RESP SPEC CASE: QY

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	----	N/A
U2	IBC2006	8.2192
UZ	----	N/A

RESP SPEC CASE: QXUNSCALED

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	IBC2006	32.2000
U2	----	N/A
UZ	----	N/A

RESP SPEC CASE: QYUNSCALED

BASIC RESPONSE SPECTRUM DATA

MODAL COMBO	DIRECTION COMBO	MODAL DAMPING	SPECTRUM ANGLE	TYPICAL ECCEN
CQC	SRSS	0.0500	0.0000	0.0500

RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA

DIRECTION	FUNCTION	SCALE FACT
U1	----	N/A
U2	IBC2006	32.2000
UZ	----	N/A

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T

Case: EX

AUTO SEISMIC INPUT DATA

Direction: X
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: BASE

C = 0.188
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 1504.55

V Used = 0.1880W = 282.86

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
HIGH RF	0.00	0.00	0.00	0.000	0.000	0.000
RF	282.86	0.00	0.00	0.000	-15.462	-43.627

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EY

AUTO SEISMIC INPUT DATA

Direction: Y
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: BASE

C = 0.188
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 1504.55

V Used = 0.1880W = 282.86

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
HIGH RF	0.00	0.00	0.00	0.000	0.000	0.000
RF	0.00	282.86	0.00	15.462	0.000	-4.615

A U T O S E I S M I C U S E R C O E F F I C I E N T
Case: EXP

AUTO SEISMIC INPUT DATA

Direction: X + EccY
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: BASE

C = 0.188
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 1504.55

V Used = 0.1880W = 282.86

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
HIGH RF	0.00	0.00	0.00	0.000	0.000	0.000
RF	282.86	0.00	0.00	0.000	-15.462	-1999.225

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EXM

AUTO SEISMIC INPUT DATA

Direction: X - EccY
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: BASE

C = 0.188
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 1504.55

V Used = 0.1880W = 282.86

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
HIGH RF	0.00	0.00	0.00	0.000	0.000	0.000
RF	282.86	0.00	0.00	0.000	-15.462	1911.971

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EYP

AUTO SEISMIC INPUT DATA

Direction: Y + EccX
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: BASE

C = 0.188
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 1504.55

V Used = 0.1880W = 282.86

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
HIGH RF	0.00	0.00	0.00	0.000	0.000	0.000
RF	0.00	282.86	0.00	15.462	0.000	837.695

A U T O   S E I S M I C   U S E R   C O E F F I C I E N T
Case: EYM

AUTO SEISMIC INPUT DATA

Direction: Y - EccX
Typical Eccentricity = 5%
Eccentricity Overrides: No

Top Story: RF
Bottom Story: BASE

C = 0.188
K = 1

AUTO SEISMIC CALCULATION FORMULAS

V = C W

AUTO SEISMIC CALCULATION RESULTS

W Used = 1504.55

V Used = 0.1880W = 282.86

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
HIGH RF	0.00	0.00	0.00	0.000	0.000	0.000
RF	0.00	282.86	0.00	15.462	0.000	-846.926

M A S S   S O U R C E   D A T A

MASS FROM	LATERAL MASS ONLY	LUMP MASS AT STORIES
Masses	Yes	Yes

D I A P H R A G M   M A S S   D A T A

STORY	DIAPHRAGM	MASS-X	MASS-Y	MMI	X-M	Y-M
RF	D1	4.385E+01	4.385E+01	1.188E+05	363.100	72.726
RF	D2	1.704E+00	1.704E+00	5.232E+02	314.361	72.917

A S S E M B L E D   P O I N T   M A S S E S

STORY	UX	UY	UZ	RX	RY	RZ
HIGH RF	1.309E+01	1.309E+01	0.000E+00	0.000E+00	0.000E+00	0.000E+00
RF	4.676E+01	4.676E+01	0.000E+00	0.000E+00	0.000E+00	1.193E+05
BASE	2.275E+01	2.275E+01	0.000E+00	0.000E+00	0.000E+00	0.000E+00
Totals	8.261E+01	8.261E+01	0.000E+00	0.000E+00	0.000E+00	1.193E+05

C E N T E R S   O F   C U M U L A T I V E   M A S S   &   C E N T E R S   O F   R I G I D I T Y

STORY	DIAPHRAGM	/----------CENTER OF MASS----------			//--CENTER OF RIGIDITY--/	
LEVEL	NAME	MASS	ORDINATE-X	ORDINATE-Y	ORDINATE-X	ORDINATE-Y
RF	D1	4.385E+01	363.100	72.726	365.842	74.905
RF	D2	1.704E+00	314.361	72.917	329.381	73.133

M O D A L   P E R I O D S   A N D   F R E Q U E N C I E S

MODE NUMBER	PERIOD (TIME)	FREQUENCY (CYCLES/TIME)	CIRCULAR FREQ (RADIANS/TIME)
Mode 1	0.08135	12.29238	77.23528
Mode 2	0.07728	12.94058	81.30809
Mode 3	0.07376	13.55746	85.18400
Mode 4	0.06519	15.34057	96.38767
Mode 5	0.06420	15.57605	97.86721
Mode 6	0.05794	17.25960	108.44530
Mode 7	0.05524	18.10234	113.74038
Mode 8	0.05085	19.66648	123.56813
Mode 9	0.04975	20.10077	126.29689
Mode 10	0.04930	20.28473	127.45269
Mode 11	0.04787	20.89089	131.26131
Mode 12	0.04675	21.39074	134.40197
Mode 13	0.04516	22.14467	139.13909
Mode 14	0.04264	23.45184	147.35228
Mode 15	0.04151	24.08781	151.34820
Mode 16	0.04120	24.27334	152.51392
Mode 17	0.03751	26.66018	167.51085
Mode 18	0.03713	26.93528	169.23933
Mode 19	0.03470	28.81972	181.07964
Mode 20	0.03082	32.44214	203.83998
Mode 21	0.03017	33.14325	208.24518
Mode 22	0.02979	33.57391	210.95112
Mode 23	0.02928	34.15150	214.58020
Mode 24	0.02650	37.74010	237.12806
Mode 25	0.02437	41.03064	257.80312
Mode 26	0.02368	42.22665	265.31790
Mode 27	0.02328	42.95171	269.87357
Mode 28	0.02249	44.46276	279.36773

M O D A L   P A R T I C I P A T I N G   M A S S   R A T I O S

MODE NUMBER	X-TRANS %MASS <SUM>	Y-TRANS %MASS <SUM>	Z-TRANS %MASS <SUM>	RX-ROTN %MASS <SUM>	RY-ROTN %MASS <SUM>	RZ-ROTN %MASS <SUM>
Mode 1	0.01 < 0>	0.00 < 0>	0.00 < 0>	0.00 < 0>	0.01 < 0>	0.00 < 0>
Mode 2	0.00 < 0>	0.21 < 0>	0.00 < 0>	1.71 < 2>	0.00 < 0>	0.01 < 0>
Mode 3	71.23 < 71>	0.02 < 0>	0.00 < 0>	0.02 < 2>	73.05 < 73>	0.13 < 0>
Mode 4	0.04 < 71>	28.05 < 28>	0.00 < 0>	27.59 < 29>	0.04 < 73>	1.11 < 1>
Mode 5	0.26 < 72>	0.00 < 28>	0.00 < 0>	0.00 < 29>	0.09 < 73>	0.00 < 1>
Mode 6	0.13 < 72>	13.93 < 42>	0.00 < 0>	14.26 < 44>	0.13 < 73>	15.44 < 17>
Mode 7	0.01 < 72>	39.12 < 81>	0.00 < 0>	40.74 < 84>	0.02 < 73>	15.73 < 32>
Mode 8	8.93 < 81>	0.01 < 81>	0.00 < 0>	0.01 < 84>	8.94 < 82>	0.04 < 32>
Mode 9	0.00 < 81>	2.83 < 84>	0.00 < 0>	2.63 < 87>	0.00 < 82>	9.03 < 41>
Mode 10	1.06 < 82>	0.01 < 84>	0.00 < 0>	0.01 < 87>	1.09 < 83>	0.00 < 41>
Mode 11	2.30 < 84>	0.00 < 84>	0.00 < 0>	0.00 < 87>	2.21 < 86>	0.03 < 42>
Mode 12	0.09 < 84>	0.03 < 84>	0.00 < 0>	0.02 < 87>	0.09 < 86>	0.00 < 42>
Mode 13	0.00 < 84>	7.06 < 91>	0.00 < 0>	5.78 < 93>	0.00 < 86>	1.75 < 43>
Mode 14	0.00 < 84>	7.70 < 99>	0.00 < 0>	6.51 < 99>	0.00 < 86>	0.34 < 44>
Mode 15	0.50 < 85>	0.00 < 99>	0.00 < 0>	0.01 < 99>	0.48 < 86>	52.61 < 96>
Mode 16	13.31 < 98>	0.00 < 99>	0.00 < 0>	0.00 < 99>	12.39 < 99>	0.96 < 97>
Mode 17	0.39 < 98>	0.00 < 99>	0.00 < 0>	0.00 < 99>	0.36 < 99>	0.01 < 97>
Mode 18	0.01 < 98>	0.03 < 99>	0.00 < 0>	0.02 < 99>	0.01 < 99>	0.63 < 98>
Mode 19	0.01 < 98>	0.00 < 99>	0.00 < 0>	0.00 < 99>	0.01 < 99>	0.00 < 98>
Mode 20	0.03 < 98>	0.00 < 99>	0.00 < 0>	0.00 < 99>	0.03 < 99>	0.19 < 98>
Mode 21	0.01 < 98>	0.01 < 99>	0.00 < 0>	0.01 < 99>	0.01 < 99>	0.07 < 98>
Mode 22	0.01 < 98>	0.00 < 99>	0.00 < 0>	0.00 < 99>	0.01 < 99>	0.00 < 98>
Mode 23	0.00 < 98>	0.00 < 99>	0.00 < 0>	0.00 < 99>	0.00 < 99>	0.32 < 98>
Mode 24	0.12 < 98>	0.00 < 99>	0.00 < 0>	0.00 < 99>	0.11 < 99>	0.00 < 98>
Mode 25	0.16 < 99>	0.00 < 99>	0.00 < 0>	0.00 < 99>	0.08 < 99>	0.00 < 98>
Mode 26	0.00 < 99>	0.04 < 99>	0.00 < 0>	0.01 < 99>	0.00 < 99>	0.00 < 98>
Mode 27	0.00 < 99>	0.00 < 99>	0.00 < 0>	0.00 < 99>	0.00 < 99>	0.00 < 98>
Mode 28	0.01 < 99>	0.00 < 99>	0.00 < 0>	0.00 < 99>	0.00 < 99>	0.06 < 98>

M O D A L   L O A D   P A R T I C I P A T I O N   R A T I O S
(STATIC AND DYNAMIC RATIOS ARE IN PERCENT)

TYPE	NAME	STATIC	DYNAMIC
Load	D	0.1979	0.0000
Load	L	0.0754	0.0000
Load	EX	99.2728	86.2738
Load	EY	98.8339	80.1785
Load	EXP	99.2564	85.9149
Load	EXM	99.2526	86.0734
Load	EYP	98.7969	79.8405
Load	EYM	98.8636	80.4404
Accel	UX	99.9709	98.6207
Accel	UY	99.9706	99.0586
Accel	UZ	0.0000	0.0000
Accel	RX	42.7950	99.3443
Accel	RY	157.1055	99.1544
Accel	RZ	133.3504	98.4836

TOTAL REACTIVE FORCES (RECOVERED LOADS) AT ORIGIN

LOAD	FX	FY	FZ	MX	MY	MZ
D	-9.477E-11	-2.188E-15	2.586E+03	1.892E+05	-9.367E+05	1.120E-08
L	-2.817E-12	-4.135E-14	2.194E+02	1.596E+04	-7.894E+04	2.961E-10
EX	-2.829E+02	-1.333E-13	-8.920E-14	-7.369E-12	-8.612E+03	2.062E+04
EY	1.044E-10	-2.829E+02	-5.844E-14	8.612E+03	3.204E-09	-1.022E+05
EXP	-2.829E+02	-3.980E-13	-8.549E-14	5.135E-12	-8.612E+03	2.257E+04
EXM	-2.829E+02	1.999E-13	-1.300E-13	-1.870E-11	-8.612E+03	1.866E+04
EYP	1.809E-11	-2.829E+02	1.639E-14	8.612E+03	5.488E-10	-1.030E+05
EYM	1.151E-10	-2.829E+02	4.193E-14	8.612E+03	3.502E-09	-1.013E+05
QX	2.832E+02	5.925E+00	1.371E-13	1.846E+02	8.946E+03	2.267E+04
QY	7.031E+00	2.852E+02	4.444E-13	8.919E+03	2.231E+02	1.044E+05
QXUNSCALED	1.316E+03	2.754E+01	6.255E-13	8.583E+02	4.159E+04	1.054E+05
QYUNSCALED	2.754E+01	1.117E+03	1.739E-12	3.494E+04	8.740E+02	4.091E+05

S T O R Y   F O R C E S

STORY	LOAD	P	VX	VY	T	MX	MY
HIGH RF	EX	-1.247E-13	-1.491E-02	1.364E-05	1.085E+00	-4.775E-05	-5.217E-02
RF	EX	3.176E-11	-2.829E+02	1.581E-04	2.062E+04	-2.025E-03	-3.521E+03
HIGH RF	EY	-1.033E-13	4.688E-05	-9.833E-03	-3.580E+00	3.441E-02	1.641E-04
RF	EY	1.189E-11	1.910E-04	-2.829E+02	-1.022E+05	3.521E+03	2.556E-03
HIGH RF	EXP	-8.332E-14	-1.488E-02	6.023E-05	1.154E+00	-2.108E-04	-5.207E-02
RF	EXP	3.246E-11	-2.829E+02	-5.307E-05	2.258E+04	2.980E-04	-3.521E+03
HIGH RF	EXM	-1.387E-13	-1.493E-02	-3.295E-05	1.015E+00	1.153E-04	-5.227E-02
RF	EXM	3.106E-11	-2.829E+02	3.692E-04	1.867E+04	-4.347E-03	-3.521E+03
HIGH RF	EYP	1.306E-14	3.490E-05	-9.855E-03	-3.611E+00	3.449E-02	1.221E-04
RF	EYP	1.157E-11	1.173E-04	-2.829E+02	-1.030E+05	3.521E+03	1.594E-03
HIGH RF	EYM	-7.297E-14	5.887E-05	-9.810E-03	-3.550E+00	3.434E-02	2.060E-04
RF	EYM	1.219E-11	2.647E-04	-2.829E+02	-1.014E+05	3.521E+03	3.519E-03
HIGH RF	QX	1.322E-13	9.775E+01	1.648E+00	7.220E+03	5.769E+00	3.421E+02
RF	QX	3.913E-11	2.820E+02	5.810E+00	2.256E+04	7.805E+01	3.852E+03
HIGH RF	QY	1.521E-13	2.562E+00	7.767E+01	2.850E+04	2.718E+02	8.968E+00
RF	QY	9.877E-12	7.028E+00	2.830E+02	1.037E+05	3.787E+03	9.659E+01
HIGH RF	QXUNSCALED	6.168E-13	4.544E+02	7.662E+00	3.356E+04	2.682E+01	1.590E+03
RF	QXUNSCALED	1.819E-10	1.311E+03	2.701E+01	1.049E+05	3.628E+02	1.790E+04
HIGH RF	QYUNSCALED	6.000E-13	1.004E+01	3.043E+02	1.116E+05	1.065E+03	3.513E+01
RF	QYUNSCALED	3.870E-11	2.753E+01	1.109E+03	4.063E+05	1.484E+04	3.784E+02

STORY DRIFTS

STORY	DIRECTION	LOAD	MAX DRIFT
HIGH RF	X	EX	1/5467
RF	X	EX	1/18927
HIGH RF	X	EY	1/17749
RF	Y	EY	1/32461
HIGH RF	X	EXP	1/5627
RF	X	EXP	1/18984
HIGH RF	X	EXM	1/5316
RF	X	EXM	1/18871
HIGH RF	X	EYP	1/18628
RF	Y	EYP	1/31756
HIGH RF	X	EYM	1/16950
RF	Y	EYM	1/32650
HIGH RF	X	QX	1/1356
RF	X	QX	1/9579
HIGH RF	Y	QY	1/1531
RF	Y	QY	1/7354
HIGH RF	X	QXUNSCALED	1/292
RF	X	QXUNSCALED	1/2061
HIGH RF	Y	QYUNSCALED	1/391
RF	Y	QYUNSCALED	1/1877

DISPLACEMENTS AT DIAPHRAGM CENTER OF MASS

STORY	DIAPHRAGM	LOAD	UX	UY	RZ
RF	D1	EX	0.0005	0.0000	0.00000
RF	D2	EX	0.0002	0.0000	0.00000
RF	D1	EY	0.0000	0.0004	0.00000
RF	D2	EY	0.0000	0.0003	0.00000
RF	D1	EXP	0.0005	0.0000	0.00000
RF	D2	EXP	0.0002	0.0000	0.00000
RF	D1	EXM	0.0005	0.0000	0.00000
RF	D2	EXM	0.0002	0.0000	0.00000
RF	D1	EYP	0.0000	0.0004	0.00000
RF	D2	EYP	0.0000	0.0003	0.00000
RF	D1	EYM	0.0000	0.0004	0.00000
RF	D2	EYM	0.0000	0.0003	0.00000
RF	D1	QX	0.0005	0.0000	0.00000
RF	D2	QX	0.0001	0.0000	0.00000
RF	D1	QY	0.0000	0.0004	0.00000
RF	D2	QY	0.0000	0.0003	0.00001
RF	D1	QXUNSCALED	0.0025	0.0001	0.00000
RF	D2	QXUNSCALED	0.0006	0.0001	0.00001
RF	D1	QYUNSCALED	0.0000	0.0015	0.00001
RF	D2	QYUNSCALED	0.0000	0.0011	0.00003

Roof Diaphragm Dipt under Qx load = 0.0005*12"*4=0.024"
Qy load = 0.0003*12"*4=0.014"
Cd=4

Above displacements are based on seismic coefficient of 0.188 (10% in 50 year earthquake);
for 10% in 5 year earthquake, seismic coefficient is 0.061, 0.061/0.188=0.324, modify displacement

For 10% in 5 year earthquake;
Diaphragm Displacement Roof under Qx load = 0.024*0.324=0.008"
Qy load = 0.014*0.324=0.005"

(f) Torsion Irregularity Check

```
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C E N T E R S   O F   C U M U L A T I V E   M A S S   &   C E N T E R S   O F   R I G I D I T Y

STORY       DIAPHRAGM    /----------CENTER OF MASS----------//--CENTER OF RIGIDITY--/
LEVEL       NAME                MASS  ORDINATE-X  ORDINATE-Y  ORDINATE-X  ORDINATE-Y

RF          D1             5.267E+01      37.597      72.103      16.635      79.508
2ND         D1             1.210E+02      36.110      73.464      54.871      86.664
2ND         D2             1.077E+01      91.209      71.745      91.760      63.569
```

Community Building, Torsion Irregularity Check, Distance between center of mass and center of rigidity;

Rf Level, Dx=37.60-16.64=20.96' ; > 20%*77' =15.4'

Dy=79.51-72.10= 7.41' ; < 20%*130' = 26'

2nd Level, Dx=54.87-36.11=18.76' ; > 20%*77' = 15.4'

Dy=86.66-73.46=13.2' : < 20%*130'= 26'

Therefore, Significant Torsion does exist, per Section 3.3.6 definition of CAC 2010 Chapter 6.

C E N T E R S   O F   C U M U L A T I V E   M A S S   &   C E N T E R S   O F   R I G I D I T Y

STORY LEVEL	DIAPHRAGM NAME	CENTER OF MASS MASS	CENTER OF MASS ORDINATE-X	CENTER OF MASS ORDINATE-Y	CENTER OF RIGIDITY ORDINATE-X	CENTER OF RIGIDITY ORDINATE-Y
RF	D1	2.203E+02	189.747	72.917	170.948	72.913
RF	D2	1.510E+00	85.083	72.917	80.469	72.830
1ST	D1	2.507E+02	177.557	72.879	94.546	72.417

Natatorium Building Torsion Irregularity Check, Distance between center of mass and center of rigidity;
Rf Level, Dx=189.75-170.95=18.8' < 20%*233' =46.6'
Dy=72.92-72.91= 0.01' < 20%*146' = 29.2'

Therefore, Significant Torsion does not exist, per Section 3.3.6 definition of CAC 2010 Chapter 6.

C E N T E R S O F C U M U L A T I V E M A S S & C E N T E R S O F R I G I D I T Y

STORY	DIAPHRAGM	/----------CENTER OF MASS----------			//--CENTER OF RIGIDITY--/	
LEVEL	NAME	MASS	ORDINATE-X	ORDINATE-Y	ORDINATE-X	ORDINATE-Y
RF	D1	4.385E+01	363.100	72.726	365.842	74.905
RF	D2	1.704E+00	314.361	72.917	329.381	73.133

Locker Building, Torsion Irregularity Check, Distance between center of mass and center of rigidity;
Rf Level, Dx=365.8-363.1=2.7' < 20%* 88' =17.6'
Dy= 74.9-72.7= 2.2' < 20%*146' = 29.2'

Therefore, Significant Torsion does NOT exist, per Section 3.3.6 definition of CAC 2010 Chapter 6.

(g) Natatorium Buildign, Deflection Incompatibility Check (for Corner column)

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L O A D I N G   C O M B I N A T I O N S

COMBO	COMBO TYPE	CASE	CASE TYPE	SCALE FACTOR
DIQC21A	ADD	D	Static	1.1000
		L	Static	0.2500
		QXENV	Combo	2.0000
DIQC21B	ADD	D	Static	1.1000
		L	Static	0.2500
		QYENV	Combo	2.0000
DIQC22A	ADD	D	Static	0.9000
		QXENV	Combo	2.0000
DIQC22B	ADD	D	Static	0.9000
		QYENV	Combo	2.0000

QXENV = QX+0.3QY
QYENV = QY+0.3QX

Note:

1. For corner columns only, above load combinations are used because the corner columns participate in both X and Y directions lateral force system.
2. For columns not at the corners, the 30% portions need NOT be included.
3. Only corner columns are checked for deflection incompatibility, because they are the most critical.

C O L U M N   F O R C E   E N V E L O P E S

STORY	COLUMN	ITEM	P	V2	V3	T	M2	M3
RF	C1	Min Value	-368.55	-58.12	-53.82	-14.518	-881.022	-837.662
		Min Case	DIQC21B	DIQC21A	DIQC22B	DIQC21A	DIQC22B	DIQC22A
		Max Value	83.46	54.00	53.38	18.125	891.449	887.519
		Max Case	DIQC22B	DIQC22A	DIQC21B	DIQC21A	DIQC21B	DIQC21A

C O L U M N F O R C E S

STORY	COLUMN	LOAD	LOC	P	V2	V3	T	M2	M3
RF	C1	DIQC21A MAX							
			0.0000	-7.05	53.50	18.23	6.121	137.552	452.916
			23.0000	14.04	53.50	18.23	6.121	292.163	829.403
			24.0000	14.95	53.50	18.23	6.121	310.772	887.519
			24.0000	-32.69	0.25	-0.03	18.125	5.681	19.821
			46.0000	-12.53	0.25	-0.03	18.125	10.788	15.743
RF	C1	DIQC21A MIN							
			0.0000	-331.13	-58.12	-18.61	-14.518	-135.854	-507.278
			23.0000	-310.05	-58.12	-18.61	-14.518	-281.760	-777.653
			24.0000	-309.13	-58.12	-18.61	-14.518	-299.991	-831.155
			24.0000	-81.37	-1.18	-0.58	-7.369	-8.754	-22.254
			46.0000	-61.20	-1.18	-0.58	-7.369	-0.544	2.112
RF	C1	DIQC21B MAX							
			0.0000	30.37	25.27	53.38	4.160	400.393	211.392
			23.0000	51.46	25.27	53.38	4.160	837.693	421.496
			24.0000	52.38	25.27	53.38	4.160	891.449	451.375
			24.0000	-23.42	0.12	0.00	17.061	14.395	11.486
			46.0000	-3.25	0.12	0.00	17.061	15.739	13.890
RF	C1	DIQC21B MIN							
			0.0000	-368.55	-29.88	-53.76	-12.558	-398.695	-265.753
			23.0000	-347.47	-29.88	-53.76	-12.558	-827.290	-369.747
			24.0000	-346.55	-29.88	-53.76	-12.558	-880.668	-395.012
			24.0000	-90.65	-1.04	-0.61	-6.305	-17.468	-13.919
			46.0000	-70.48	-1.04	-0.61	-6.305	-5.495	3.965
RF	C1	DIQC22A MAX							
			0.0000	28.04	54.00	18.16	7.191	135.614	458.445
			23.0000	45.29	54.00	18.16	7.191	291.743	823.397
			24.0000	46.04	54.00	18.16	7.191	310.418	881.012
			24.0000	-21.15	0.37	0.05	16.749	6.041	20.162
			46.0000	-4.65	0.37	0.05	16.749	9.584	13.520
RF	C1	DIQC22A MIN							
			0.0000	-296.04	-57.61	-18.68	-13.447	-137.792	-501.750
			23.0000	-278.79	-57.61	-18.68	-13.447	-282.180	-783.658
			24.0000	-278.04	-57.61	-18.68	-13.447	-300.345	-837.662
			24.0000	-69.83	-1.06	-0.51	-8.745	-8.395	-21.914
			46.0000	-53.33	-1.06	-0.51	-8.745	-1.748	-0.111
RF	C1	DIQC22B MAX							
			0.0000	65.46	25.77	53.31	5.231	398.455	216.920
			23.0000	82.71	25.77	53.31	5.231	837.273	415.491
			24.0000	83.46	25.77	53.31	5.231	891.095	444.868
			24.0000	-11.88	0.23	0.07	15.685	14.755	11.827
			46.0000	4.63	0.23	0.07	15.685	14.535	11.667
RF	C1	DIQC22B MIN							
			0.0000	-333.47	-29.38	-53.82	-11.487	-400.633	-260.225
			23.0000	-316.21	-29.38	-53.82	-11.487	-827.710	-375.752
			24.0000	-315.46	-29.38	-53.82	-11.487	-881.022	-401.519
			24.0000	-79.10	-0.92	-0.54	-7.682	-17.109	-13.578
			46.0000	-62.60	-0.92	-0.54	-7.682	-6.699	1.742

General Information:

File Name: C:\PROGRA~1\PCACOL\BMT-C1.COL
Project: Belmont Pool Natatorium Bldg
Column: Corner Col — Engineer:
Code: ACI 318-95 — Units: English

Run Option: Investigation — Slenderness: Not considered
Run Axis: Biaxial — Column Type: Structural

Material Properties:

f'c = 5 ksi — fy = 50 ksi
Ec = 4030.51 ksi — Es = 29000 ksi
fc = 4.25 ksi — Rupture strain = Infinity
Ultimate strain = 0.003 in/in
Beta1 = 0.8

Section:

Exterior Points

No.	X (in)	Y (in)	No.	X (in)	Y (in)	No.	X (in)	Y (in)
1	-10.0	18.0	2	10.0	18.0	3	10.0	10.0
4	18.0	10.0	5	18.0	-10.0	6	10.0	-10.0
7	10.0	-18.0	8	-10.0	-18.0	9	-10.0	-10.0
10	-18.0	-10.0	11	-18.0	10.0	12	-10.0	10.0

Gross section area, Ag = 1040 in^2
Ix = 88426.7 in^4 — Iy = 88426.7 in^4
Xo = 0 in — Yo = 0 in

Reinforcement:

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7

Pattern: Irregular

Total steel area, As = 15.60 in^2 at 1.50%

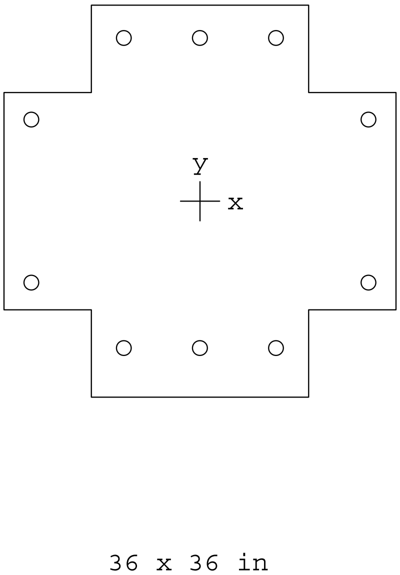
Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)
1.56	-7.0	-13.5	1.56	0.0	15.0	1.56	-7.0	15.0
1.56	7.0	15.0	1.56	7.0	-13.5	1.56	0.0	-13.5
1.56	-15.5	7.5	1.56	-15.5	-7.5	1.56	15.5	-7.5
1.56	15.5	7.5						

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

No.	Pu kip	Mux k-ft	Muy k-ft	fMnx k-ft	fMny k-ft	fMn/Mu
1	-15.0	310.8	887.5	280.8	801.8	0.903
2	309.1	300.0	831.2	349.3	971.1	1.168
3	-52.4	891.5	451.4	715.1	362.5	0.802
4	346.6	880.7	395.0	985.4	441.0	1.119
5	-46.0	310.4	881.0	272.0	770.7	0.875
6	278.0	300.3	837.7	346.1	960.3	1.147
7	-83.5	891.1	444.9	687.1	343.4	0.771
8	315.5	881.0	401.5	969.0	441.4	1.100

*** Program completed as requested! ***

When capacity fMn / Demand Mu ratio is less than 1.0, it indicates that the column capacity is not sufficient. So, DI exists.



Code: ACI 318-95

Units: English

Run axis: Biaxial

Run option: Investigation

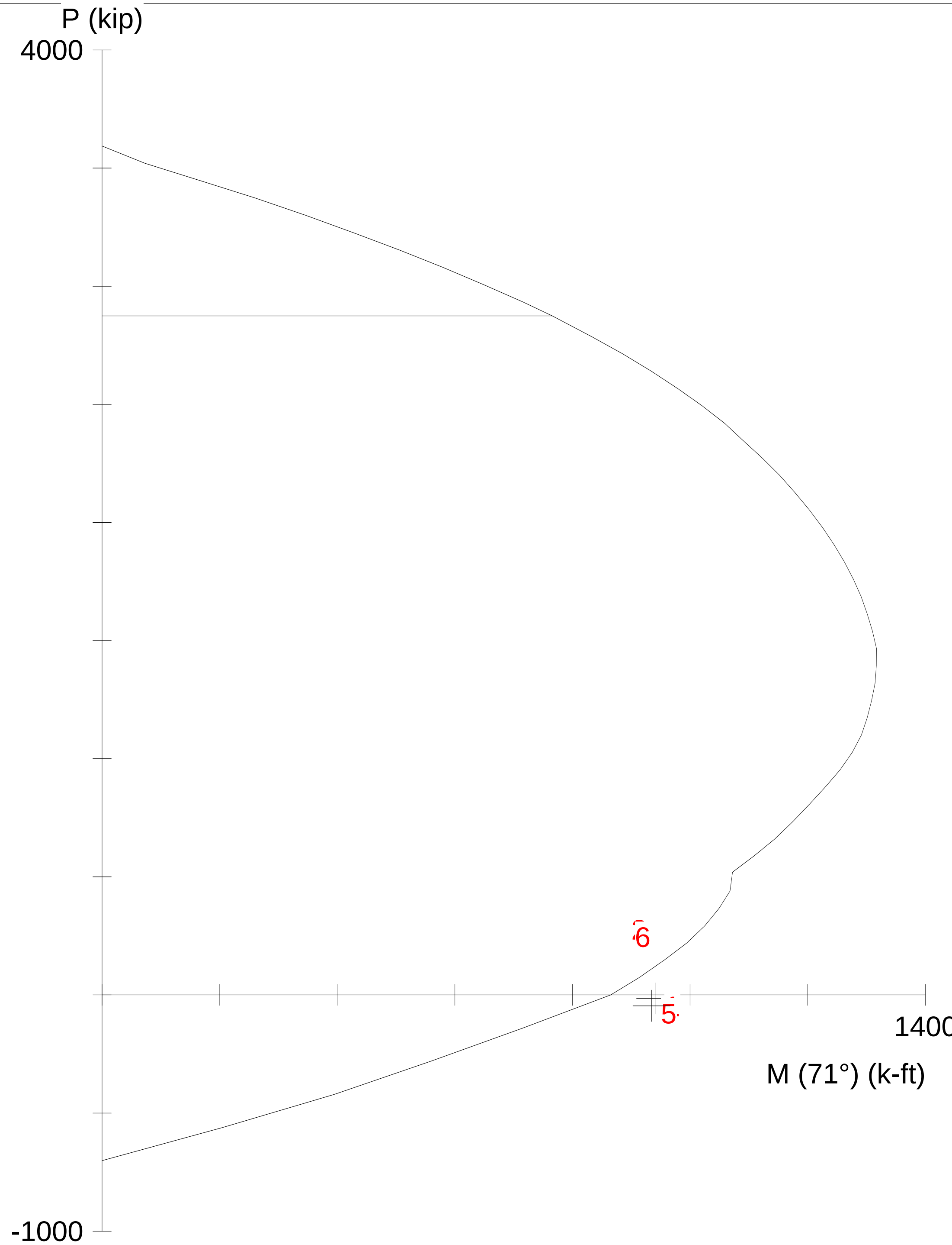
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/15/12

Time: 10:25:45



PCACOL V3.00 (PCA 1999) - Licensed to: TTG, Pasadena, CA

File: C:\PROGRA~1\PCACOL\BMT-C1.COL

Project: Belmont Pool Natatorium Bldg

Column: Corner Col

Engineer:

f'c = 5 ksi	fy = 50 ksi	Ag = 1040 in^2	10 #11 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 15.60 in^2	Rho = 1.50%
fc = 4.25 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 88426.7 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 88426.7 in^4
Beta1 = 0.8		Clear spacing = 5.59 in	Clear cover = N/A

(h) Evaluation of Significant Structural Deficiencies

SIGNIFICANT STRUCTURAL DEFICIENCIES
Community Building

List of HAZUS Related Deficiencies (Per CAC 2010 Chapter 6, Section 1.4.5.1.2.2; Sub Section 2.2)

S/N	Description of Deficiencies	Status	Remark
a	Age–Year of the CBC code used for the original design. Post 1961, drawing dated 1967, may be designed per UBC 1964.	Post-61	
b	Materials Tests (Section 2.1.2) – Present materials properties based on test results for OSHPD approval. Per Exception in Section 2.1.2: material testing is not required for reclassification by the collapse probability assessment option as permitted by Section 1.4.5.1.2, where non-availability of materials test is a deficiency per Section 1.4.5.1.2.2.2.2 (b).	False	MT is considered as a deficiency.
c	Load Path (Section 3.1) – Yes, There is complete load path for seismic force effect from any horizontal direction.	True	
d	Mass Irregularity (Section 3.3.4) – Significant mass irregularity does NOT exist.	True	
e	Vertical Discontinuity (Section 3.3.5) – All shear walls and infill walls continue to foundation.	False	Shear walls do not continue to foundation.
f	Short Captive Column (Section 3.6) – No columns with height-to-depth ratios less than 75% of the nominal height-to-depth ratios of the typical columns at that level.	True	The columns all have same section dimension; so the height-to-depth ratios are all the same. SCC does not exist.
g	Material Deterioration (Section 3.7) – No visible deterioration of concrete or reinforcing steel.	False	There are visible signs of concrete deterioration.
h	Weak Columns (Section 4.2.8 & 4.3.6) – [Section 4.3.6 does not apply; there is no concrete moment frame in the building system]. This deficiency does not exist.	True	It is concrete shear wall system.

i	Wall anchorage (Section 8.2) – Exterior concrete walls are anchored to each of the diaphragm levels for out-of-plane loads	True	Wall reinforcements extend into floor and roof concrete diaphragms.
j	Redundancy (Section 3.2) – There is redundancy present in the lateral system; the structure will remain laterally stable after the failure of any single wall element.	True	
k	Weak story irregularity (Section 3.3.1) – There are NO significant strength discontinuities in a floor of the vertical elements in the lateral force resisting system; the story strength below a floor is not less than 80% of the strength of the story above.	True	
l	Soft Story irregularity (Section 3.3.2) - There are NO significant stiffness discontinuities in a story of the vertical elements in the lateral force resisting system; the story stiffness of a story is NOT less than 70% of that in the story above.	True	
m	Torsional irregularity (Section 3.3.6) - It appears that significant torsion does exist; the distance between the center of rigidity and center of mass appears to be more than 20% of the width of the structure in either major plan dimension.	False	See structural calculations for TI check; TI does exist.
n	Deflection incompatibility (Section 3.5) - Gravity columns and beams need to be checked for being capable of accommodating imposed building drifts including amplified drift, without loss of vertical load-carrying capacity.	False	
o	Cripple walls (Section 5.6.4) – N.A.	NA	This is not a wood frame building; code Section 5.6.4 does not apply.
p	Openings in diaphragm at shear walls (Section 7.1.4) – Does not exist	True	
q	Topping slab missing (Section 7.3 & 7.4) or the building type (structural system) is of lift slab construction – N.A.	True	

Other Information for the building

Code/Year Built	Building Height / (No. of Floors)	SPC Rating	Model Building Type Per Table 1.4.5.1 of Chapter 6, CAC2010
Post-61 (1967) UBC-1964	Above Grade: 31'-0" / (2) Below Grade: 9'-0" / (1)		C2 – Concrete Shear Walls

Note:
(1) Seismic Base is taken at the 1st floor slab on grade elevation.

SIGNIFICANT STRUCTURAL DEFICIENCIES

Natatorium Building (As-is Condition)

List of HAZUS Related Deficiencies (Per CAC 2010 Chapter 6, Section 1.4.5.1.2.2; Sub Section 2.2)

S/N	Description of Deficiencies	Status	Remark
a	Age–Year of the CBC code used for the original design. Post 1961, drawing dated 1967, may be designed per UBC 1964.	Post-61	
b	Materials Tests (Section 2.1.2) – Present materials properties based on test results for OSHPD approval. Per Exception in Section 2.1.2: material testing is not required for reclassification by the collapse probability assessment option as permitted by Section 1.4.5.1.2, where non-availability of materials test is a deficiency per Section 1.4.5.1.2.2.2.2 (b).	False	MT is considered as a deficiency.
c	Load Path (Section 3.1) – Yes, There is complete load path for seismic force effect from any horizontal direction.	True	
d	Mass Irregularity (Section 3.3.4) – Significant mass irregularity does NOT exist.	True	
e	Vertical Discontinuity (Section 3.3.5) – No, it does not exist.	False	
f	Short Captive Column (Section 3.6) – No columns with height-to-depth ratios less than 75% of the nominal height-to-depth ratios of the typical columns at that level.	True	The columns all have same section dimension; so the height-to-depth ratios are all the same. SCC does not exist.
g	Material Deterioration (Section 3.7) – No visible deterioration of concrete or reinforcing steel.	False	Concrete deterioration was observed
h	Weak Columns (Section 4.2.8 & 4.3.6) – Columns are weak compared to infill wall panels	False	Column is considered weak, compared with deep precast wall panel.

i	Wall anchorage (Section 8.2) – Exterior concrete walls are anchored to each of the diaphragm levels for out-of-plane loads	True	Wall reinforcements extend into floor and roof concrete diaphragms.
j	Redundancy (Section 3.2) – There is redundancy present in the lateral system; the structure will remain laterally stable after the failure of any single wall element.	True	
k	Weak story irregularity (Section 3.3.1) – There are significant strength discontinuities in the vertical elements in the lateral force resisting system; the strength below infill wall panels is less than 80% of the strength of the above.	False	
l	Soft Story irregularity (Section 3.3.2) - There are significant stiffness discontinuities in the vertical elements in the lateral force resisting system; the stiffness of the system below the infill wall panels is less than 70% of that in the above.	False	
m	Torsional irregularity (Section 3.3.6) - It appears that significant torsion does NOT exist; the distance between the center of rigidity and center of mass appears to be NOT more than 20% of the width of the structure in either major plan dimension.	True	See structural calculations for TI check; TI does NOT exist.
n	Deflection incompatibility (Section 3.5) - Gravity columns and beams need to be checked for being capable of accommodating imposed building drifts including amplified drift, without loss of vertical load-carrying capacity.	False	
o	Cripple walls (Section 5.6.4) – N.A.	NA	This is not a wood frame building; code Section 5.6.4 does not apply.
p	Openings in diaphragm at shear walls (Section 7.1.4) – Does not exist	True	
q	Topping slab missing (Section 7.3 & 7.4) or the building type (structural system) is of lift slab construction – N.A.	True	

Other Information for the building

Code/Year Built	Building Height / (No. of Floors)	SPC Rating	Model Building Type Per Table 1.4.5.1 of Chapter 6, CAC2010
Post-61 (1967) UBC-1964	Above Grade: 50'-0" / (1) Below Grade: 9'-0" / (1)	SPC-1	C1 – Concrete moment frame C2 – Concrete Shear Walls PC2 – Precast concrete frames w/ shear wall The building is of interaction of C1 and C2 system; due to use of precast wall and roof precast girders and beams; PC2 type is representative of the most severe type.

Note:

(1) Seismic Base is taken at the 1st floor slab on grade elevation.

SIGNIFICANT STRUCTURAL DEFICIENCIES

Natatorium Building (After Column Strengthening Condition)

List of HAZUS Related Deficiencies (Per CAC 2010 Chapter 6, Section 1.4.5.1.2.2; Sub Section 2.2)

S/N	Description of Deficiencies	Status	Remark
a	Age–Year of the CBC code used for the original design. Post 1961, drawing dated 1967, may be designed per UBC 1964.	Post-61	
b	Materials Tests (Section 2.1.2) – Present materials properties based on test results for OSHPD approval. Per Exception in Section 2.1.2: material testing is not required for reclassification by the collapse probability assessment option as permitted by Section 1.4.5.1.2, where non-availability of materials test is a deficiency per Section 1.4.5.1.2.2.2.2 (b).	False	MT is considered as a deficiency.
c	Load Path (Section 3.1) – Yes, There is complete load path for seismic force effect from any horizontal direction.	True	
d	Mass Irregularity (Section 3.3.4) – Significant mass irregularity does NOT exist.	True	
e	Vertical Discontinuity (Section 3.3.5) – No, it does not exist.	True	Deficiency mitigated by calculation.
f	Short Captive Column (Section 3.6) – No columns with height-to-depth ratios less than 75% of the nominal height-to-depth ratios of the typical columns at that level.	True	The columns all have same section dimension; so the height-to-depth ratios are all the same. SCC does not exist.
g	Material Deterioration (Section 3.7) – No visible deterioration of concrete or reinforcing steel.	False	Concrete deterioration was observed
h	Weak Columns (Section 4.2.8 & 4.3.6) – Columns are weak compared to infill wall panels	False	Column is considered weak, compared with deep precast wall panel, even after column strengthening.

i	Wall anchorage (Section 8.2) – Exterior concrete walls are anchored to each of the diaphragm levels for out-of-plane loads	True	Wall reinforcements extend into floor and roof concrete diaphragms.
j	Redundancy (Section 3.2) – There is redundancy present in the lateral system; the structure will remain laterally stable after the failure of any single element.	True	
k	Weak story irregularity (Section 3.3.1) – There are NO significant strength discontinuities in a floor of the vertical elements in the lateral force resisting system; the story strength below a floor is not less than 80% of the strength of the story above.	True	Deficiency mitigated by calculation, after columns are strengthened.
l	Soft Story irregularity (Section 3.3.2) - There are NO significant stiffness discontinuities in a story of the vertical elements in the lateral force resisting system; the story stiffness of a story is NOT less than 70% of that in the story above.	True	Deficiency mitigated by calculation, after columns are strengthened.
m	Torsional irregularity (Section 3.3.6) - It appears that significant torsion does NOT exist; the distance between the center of rigidity and center of mass appears to be NOT more than 20% of the width of the structure in either major plan dimension.	True	See structural calculations for TI check; TI does NOT exist.
n	Deflection incompatibility (Section 3.5) - Gravity columns and beams need to be checked for being capable of accommodating imposed building drifts including amplified drift, without loss of vertical load-carrying capacity.	True	Deficiency mitigated by calculation, after columns are strengthened.
o	Cripple walls (Section 5.6.4) – N.A.	NA	This is not a wood frame building; code Section 5.6.4 does not apply.
p	Openings in diaphragm at shear walls (Section 7.1.4) – Does not exist	True	
q	Topping slab missing (Section 7.3 & 7.4) or the building type (structural system) is of lift slab construction – N.A.	True	

Other Information for the building

Code/Year Built	Building Height / (No. of Floors)	SPC Rating	Model Building Type Per Table 1.4.5.1 of Chapter 6, CAC2010
Post-61 (1967) UBC-1964	Above Grade: 50'-0" / (1) Below Grade: 9'-0" / (1)	SPC-1	C1 – Concrete moment frame C2 – Concrete Shear Walls PC2 – Precast concrete frames w/ shear wall The building is of interaction of C1 and C2 system; due to use of precast wall and roof precast girders and beams; PC2 type is representative of the most severe type.

Note:
(1) Seismic Base is taken at the 1st floor slab on grade elevation.

SIGNIFICANT STRUCTURAL DEFICIENCIES
Locker Room Building

List of HAZUS Related Deficiencies (Per CAC 2010 Chapter 6, Section 1.4.5.1.2.2; Sub Section 2.2)

S/N	Description of Deficiencies	Status	Remark
a	Age–Year of the CBC code used for the original design. Post 1961, drawing dated 1967, may be designed per UBC 1964.	Post-61	
b	Materials Tests (Section 2.1.2) – Present materials properties based on test results for OSHPD approval. Per Exception in Section 2.1.2: material testing is not required for reclassification by the collapse probability assessment option as permitted by Section 1.4.5.1.2, where non-availability of materials test is a deficiency per Section 1.4.5.1.2.2.2.2 (b).	False	MT is considered as a deficiency.
c	Load Path (Section 3.1) – Yes, There is complete load path for seismic force effect from any horizontal direction.	True	
d	Mass Irregularity (Section 3.3.4) – Significant mass irregularity does NOT exist.	True	
e	Vertical Discontinuity (Section 3.3.5) – All shear walls and infill walls continue to foundation.	True	
f	Short Captive Column (Section 3.6) – No columns with height-to-depth ratios less than 75% of the nominal height-to-depth ratios of the typical columns at that level.	True	The columns all have same section dimension; so the height-to-depth ratios are all the same. SCC does not exist.
g	Material Deterioration (Section 3.7) – No visible deterioration of concrete or reinforcing steel.	False	There are signs of concrete deterioration.
h	Weak Columns (Section 4.2.8 & 4.3.6) – [Section 4.3.6 does not apply; there is no concrete moment frame in the building system]. This deficiency does not exist.	True	It is concrete shear wall system.

i	Wall anchorage (Section 8.2) – Exterior concrete walls are anchored to each of the diaphragm levels for out-of-plane loads	True	Wall reinforcements extend into floor and roof concrete diaphragms.
j	Redundancy (Section 3.2) – There is redundancy present in the lateral system; the structure will remain laterally stable after the failure of any single wall element.	True	
k	Weak story irregularity (Section 3.3.1) – There are NO significant strength discontinuities in a floor of the vertical elements in the lateral force resisting system; the story strength below a floor is not less than 80% of the strength of the story above.	True	
l	Soft Story irregularity (Section 3.3.2) - There are NO significant stiffness discontinuities in a story of the vertical elements in the lateral force resisting system; the story stiffness of a story is NOT less than 70% of that in the story above.	True	
m	Torsional irregularity (Section 3.3.6) - It appears that significant torsion does NOT exist; the distance between the center of rigidity and center of mass appears to be NOT more than 20% of the width of the structure in either major plan dimension.	True	See structural calculations for TI check; TI does NOT exist.
n	Deflection incompatibility (Section 3.5) - Gravity columns and beams need to be checked for being capable of accommodating imposed building drifts including amplified drift, without loss of vertical load-carrying capacity.	False	DI considered as a deficiency.
o	Cripple walls (Section 5.6.4) – N.A.	NA	This is not a wood frame building; code Section 5.6.4 does not apply.
p	Openings in diaphragm at shear walls (Section 7.1.4) – Does not exist	True	
q	Topping slab missing (Section 7.3 & 7.4) or the building type (structural system) is of lift slab construction – N.A.	True	

Other Information for the building

Code/Year Built	Building Height / (No. of Floors)	SPC Rating	Model Building Type Per Table 1.4.5.1 of Chapter 6, CAC2010
Post-61 (1967) UBC-1964	Above Grade: 12'-6" / (1)		PC2 – Precast Concrete Frames with Shear Walls

Note:
(1) Seismic Base is taken at the 1st floor slab on grade elevation.

(i) Results of Collapse Probability Calculations

Belmont Plaza Olympic Pool Center - Community Building HAZUS 2010 Scenario Studies (10% in 5 Yr Earthquake)

SSD	Case A	Case B	Case C	Case D	Case E	Case F	Remark
Material Test		x	x				
Weak Story			x				
Soft Story			x				
Vertical Discontinuity		x	x				
Concrete Deterioration		x	x				
Torsion Irregularity		x	x				
Deflection Incompatibility		x	x				
P (col) %		0.11%	**0.68%**				

Notes:
1. An "X" in the table corresponding to the row for a given deficiency means tha the deficiency is present.
2. Site Specific Response Parameters based on 10% in 5 years (47.5 years return period)

Belmont Plaza Olympic Pool Center - Natatorium Building HAZUS 2010 Scenario Studies (10% in 5 Yr Earthquake)

MBT PC2 (Precast Frame with Concrete Shear Walls)

SSD	Case A	Case B	Case C	Case D	Case E	Case F	Case G	Remark
Material Test		x	x	x	x			
Weak Story		x	x					
Soft Story		x		x				
Vertical Discontinuity		(x)						(x): not much effect
Torsion Irregularity								
Deflection Incompatibility		(x)						
Concrete Deterioration		x	x	x	x			
Weak Column		x	x	x	x			
P (col) %		**1.51%**	0.85%	0.30%	**0.03%**			

Notes:
1. An "X" in the table corresponding to the row for a given deficiency means tha the deficiency is present.
2. Site Specific Response Parameters based on 10% in 5 years (47.5 years return period)

Belmont Plaza Olympic Pool Center - Locker Building HAZUS 2010 Scenario Studies (10% in 5 Yr Earthquake)

SSD	Case A	Case B	Case C	Case D	Case E	Case F	Remark
Material Test	x	x	x				
Torsion Irregularity							
Deflection Incompatibility	x						
Concrete Deterioration	x	x					
Opening at Shear Wall							
P (col) %	**0.29%**	0.08%	0.08%				

Notes:

1. An "X" in the table corresponding to the row for a given deficiency means tha the deficiency is present.
2. Site Specific Response Parameters based on 10% in 5 years (47.5 years return period)

HAZUS ANALYSIS WORKSHEET

Building Name: **Belmont Plaza Pool, Community Building - Hazus Calculation Case: C**

	Based on Table A6-1 Deficiency	Exists? Yes=1 No=0
1	Age (Pre-1933)?	0
2	Material testing (Section 2.1.2)	1
3	Redundancy (Section 3.2)	0
4	Weak Story (Section 3.3.1)	1
5	Soft story (Section 3.3.2)	1
6	Mass irregularity (Section 3.3.4)	0
7	Vertical Discontinuity (Section 3.3.5)	1
8	Torsional irregularity (Section 3.3.6)	1
9	Deflection Incompatibility (Section 3.5)	1
10	Short Column (Section 3.6)	0
11	Wood deterioration (Section 3.7.1)	0
12	Steel deterioration (Section 3.7.3)	0
13	Conc. deterioration (Section 3.7.4)	1
14	Weak column - steel (Section 4.2.8)	0
15	Weak column - conc. (Section 4.3.6)	0
16	Cripple wall bracing (Section 5.6.4)	0
17	Topping slab missing (Section 7.3 & 7.4)	0
18	Inadequate Wall anchorage (Section 8.2)	0
19	Load Path (Section 3.1)	0
20	Opening at Shear Wall (Section 7.1.4)	0

MBT:	C2
Design Code:	Post-61
No. of Stories:	2
Distance to fault (km):	2.5
Max Magnitude:	7.4
Design Vintage:	1941-1975
Gamma	USB
Lambda	USB
kappa	SB
Δc	USB
Alpha3	USB
βc	SB
$P[COL\|STR_5]$	USB
No. of Stories Interpolated as per Building Height:	3

[7]Ground Motion Parameters Used for HAZUS

SA 0.3:	0.32
SA 1.0:	0.16

Cs =	0.086
H_R (ft) =	31
T_e (sec) =	0.39
Alpha1 =	0.8
Alpha2 =	0.75
Gamma, K =	2.25
Lambda, Σ =	1.5
Mu, T =	4.94
Elastic Damping β_E % =	7
kappa, P =	0.6
Δc =	0.03
Alpha3 =	2.5
βc =	0.95
$P[COL\|STR_5]$ =	0.5
Soil Type =	SD

HAZUS Software Input Data

Building Capacity Parameters		Structural Fragility Parameters	
[2]Dy (in) =	0.3605	[6]$S_{d,C}$ (in) =	3.35
[3]Ay (g) =	0.2419	βc =	0.95
[4]Du (in) =	2.6716	$P[COL\|STR_5]$ =	0.50
[5]Au (g) =	0.3628	kappa =	0.60

Building address: 4000 East Olympic Plaza, Long Beach, CA

Corresponding Latitude & Longitude **Latitude =** 33.7581 **Longitude =** -118.1456

(Source = MELISSA DATA at http://www.melissadata.com/lookups/addressverify.asp)

UBC 97 seismic zone = 4 **Seismic Design Level =** Moderate-Code

HAZUS Results

$P[STR_5]$ = 0.0136

P[COL] = 0.68%

Outcome: Pass **(May be re-classified to SPC-2)**

HAZUS ANALYSIS WORKSHEET

Building Name: **Belmont Plaza, Natatorium Building - 10% in 5 Yr Earthquake (475 Yr Return Period)** — **Type PC2, Case B**

	Based on Table A6-1 Deficiency	Exists? Yes=1 No=0
1	Age (Pre-1933)?	0
2	Material testing (Section 2.1.2)	1
3	Redundancy (Section 3.2)	0
4	Weak Story (Section 3.3.1)	1
5	Soft story (Section 3.3.2)	1
6	Mass irregularity (Section 3.3.4)	0
7	Vertical Discontinuity (Section 3.3.5)	1
8	Torsional irregularity (Section 3.3.6)	0
9	Deflection Incompatibility (Section 3.5)	1
10	Short Column (Section 3.6)	0
11	Wood deterioration (Section 3.7.1)	0
12	Steel deterioration (Section 3.7.3)	0
13	Conc. deterioration (Section 3.7.4)	1
14	Weak column - steel (Section 4.2.8)	0
15	Weak column - conc. (Section 4.3.6)	1
16	Cripple wall bracing (Section 5.6.4)	0
17	Topping slab missing (Section 7.3 & 7.4)	0
18	Inadequate Wall anchorage (Section 8.2)	0
19	Load Path (Section 3.1)	0
20	Opening at Shear Wall (Section 7.1.4)	0

MBT:	PC2
Design Code:	Post-61
No. of Stories:	1
Distance to fault (km):	2.5
Max Magnitude:	7.4
Design Vintage:	1941-1975
Gamma	USB
Lambda	USB
kappa	SB
Δc	SB
Alpha3	USB
βc	SB
$P[COL\|STR_5]$	USB
No. of Stories Interpolated as per Building Height:	5

[7]Ground Motion Parameters Used for HAZUS

SA 0.3:	0.32
SA 1.0:	0.16

Cs =	0.098
H_R (ft) =	50
T_e (sec) =	0.57
Alpha1 =	0.8
Alpha2 =	0.75
Gamma, K =	1.88
Lambda, Σ =	1.33
Mu, T =	4.07
Elastic Damping β_E % =	7
kappa, P =	0.6
Δc =	0.044
Alpha3 =	3.72
βc =	0.93
$P[COL\|STR_5]$ =	0.6
Soil Type =	SD

HAZUS Software Input Data

Building Capacity Parameters		Structural Fragility Parameters	
[2]Dy (in) =	0.7333	[6]$S_{d,C}$ (in) =	5.32
[3]Ay (g) =	0.2303	βc =	0.93
[4]Du (in) =	3.9693	$P[COL\|STR_5]$ =	0.60
[5]Au (g) =	0.3063	kappa =	0.60

Building address: 4000 East Olympic Plaza, Long Beach, CA

Corresponding Latitude & Longitude — **Latitude =** 33.7581 — **Longitude =** -118.1456

(Source = MELISSA DATA at http://www.melissadata.com/lookups/addressverify.asp)

UBC 97 seismic zone = 4 — **Seismic Design Level =** Moderate-Code

HAZUS Results

$P[STR_5]$ = 0.0252

P[COL] = 1.51%

Outcome: Fail

(Cannot be re-classified to SPC-2)

HAZUS ANALYSIS WORKSHEET

Building Name: **Belmont Plaza, Natatorium Building - 10% in 5 Yr Earthquake (47.5 Yr Return Period)** — **Type PC2, Case E**

#	Deficiency (Based on Table A6-1)	Exists? Yes=1 No=0
1	Age (Pre-1933)?	0
2	Material testing (Section 2.1.2)	1
3	Redundancy (Section 3.2)	0
4	Weak Story (Section 3.3.1)	0
5	Soft story (Section 3.3.2)	0
6	Mass irregularity (Section 3.3.4)	0
7	Vertical Discontinuity (Section 3.3.5)	0
8	Torsional irregularity (Section 3.3.6)	0
9	Deflection Incompatibility (Section 3.5)	0
10	Short Column (Section 3.6)	0
11	Wood deterioration (Section 3.7.1)	0
12	Steel deterioration (Section 3.7.3)	0
13	Conc. deterioration (Section 3.7.4)	1
14	Weak column - steel (Section 4.2.8)	0
15	Weak column - conc. (Section 4.3.6)	1
16	Cripple wall bracing (Section 5.6.4)	0
17	Topping slab missing (Section 7.3 & 7.4)	0
18	Inadequate Wall anchorage (Section 8.2)	0
19	Load Path (Section 3.1)	0
20	Opening at Shear Wall (Section 7.1.4)	0

MBT:	PC2
Design Code:	Post-61
No. of Stories:	1
Distance to fault (km):	2.5
Max Magnitude:	7.4
Design Vintage:	1941-1975
Gamma	USB
Lambda	USB
kappa	SB
Δc	SB
Alpha3	B
βc	SB
$P[COL \mid STR_5]$	B
No. of Stories Interpolated as per Building Height:	5

[7]Ground Motion Parameters Used for HAZUS

SA 0.3:	0.32
SA 1.0:	0.16

Cs =	0.098
H_R (ft) =	50
T_e (sec) =	0.57
Alpha1 =	0.8
Alpha2 =	0.75
Gamma, Κ =	1.88
Lambda, Σ =	1.33
Mu, Τ =	4.07
Elastic Damping β_E % =	7
kappa, Ρ =	0.6
Δc =	0.044
Alpha3 =	1.54
βc =	0.93
$P[COL \mid STR_5]$ =	0.15
Soil Type =	SD

HAZUS Software Input Data

Building Capacity Parameters		Structural Fragility Parameters	
[2]Dy (in) =	0.7333	[6]$S_{d,C}$ (in) =	12.86
[3]Ay (g) =	0.2303	βc =	0.93
[4]Du (in) =	3.9693	$P[COL \mid STR_5]$ =	0.15
[5]Au (g) =	0.3063	kappa =	0.60

Building address: 4000 East Olympic Plaza, Long Beach, CA

Corresponding Latitude & Longitude — **Latitude =** 33.7581 — **Longitude =** -118.1456

(Source = MELISSA DATA at http://www.melissadata.com/lookups/addressverify.asp)

UBC 97 seismic zone = 4 — **Seismic Design Level =** Moderate-Code

HAZUS Results

$P[STR_5]$ = 0.0018

P[COL] = 0.03%

Outcome: Pass **(May be re-classified to SPC-2)**

TMAD TAYLOR & GAINES

HAZUS ANALYSIS WORKSHEET

Building Name: **Belmont Plaza Pool, Locker Building, 10% in 50 Year Earthquake (475 Yr Return Period)** — **Type PC2**

#	Deficiency (Based on Table A6-1)	Exists? Yes=1 No=0
1	Age (Pre-1933)?	0
2	Material testing (Section 2.1.2)	1
3	Redundancy (Section 3.2)	0
4	Weak Story (Section 3.3.1)	0
5	Soft story (Section 3.3.2)	0
6	Mass irregularity (Section 3.3.4)	0
7	Vertical Discontinuity (Section 3.3.5)	0
8	Torsional irregularity (Section 3.3.6)	0
9	Deflection Incompatibility (Section 3.5)	0
10	Short Column (Section 3.6)	0
11	Wood deterioration (Section 3.7.1)	0
12	Steel deterioration (Section 3.7.3)	0
13	Conc. deterioration (Section 3.7.4)	1
14	Weak column - steel (Section 4.2.8)	0
15	Weak column - conc. (Section 4.3.6)	0
16	Cripple wall bracing (Section 5.6.4)	0
17	Topping slab missing (Section 7.3 & 7.4)	0
18	Inadequate Wall anchorage (Section 8.2)	0
19	Load Path (Section 3.1)	0
20	Opening at Shear Wall (Section 7.1.4)	0

MBT:	PC2
Design Code:	Post-61
No. of Stories:	1
Distance to fault (km):	2.5
Max Magnitude:	7.4
Design Vintage:	1941-1975
Gamma	USB
Lambda	USB
kappa	SB
Δc	B
Alpha3	B
βc	SB
$P[COL\|STR_5]$	B
No. of Stories Interpolated as per Building Height:	1

Cs =	0.133
H_R (ft) =	12.5
T_e (sec) =	0.35
Alpha1 =	0.8
Alpha2 =	0.75
Gamma, K =	2.7
Lambda, Σ =	1.33
Mu, T =	6
Elastic Damping β_E % =	7
kappa, P =	0.6
Δc =	0.053
Alpha3 =	1
βc =	0.95
$P[COL\|STR_5]$ =	0.15
Soil Type =	SD

[7] Ground Motion Parameters Used for HAZUS

SA 0.3:	0.5	0.32
SA 1.0:	0.16	0.16

HAZUS Software Input Data

Building Capacity Parameters		Structural Fragility Parameters	
[2] Dy (in) =	0.5389	[6] $S_{d,C}$ (in) =	5.96
[3] Ay (g) =	0.4489	βc =	0.95
[4] Du (in) =	4.3002	$P[COL\|STR_5]$ =	0.15
[5] Au (g) =	0.5970	kappa =	0.60

Building address: 4000 East Olympic Plaza, Long Beach, CA

Corresponding Latitude & Longitude Latitude = 33.7581, Longitude = -118.1456 (Source = MELISSA DATA at http://www.melissadata.com/lookups/addressverify.asp)

UBC 97 seismic zone = 4 — **Seismic Design Level =** Moderate-Code

HAZUS Results

$P[STR_5]$ = 0.0055

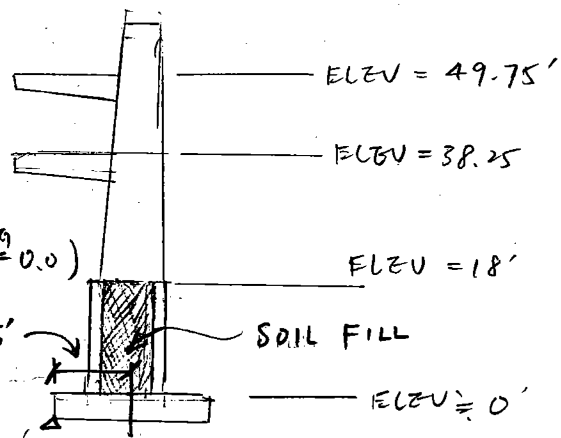
P[COL] = 0.08%

Outcome: Pass **(May be re-classified to SPC-2)**

(j) Diving Platform Overturning Check

10M DIVING TOWER

ESTIMATE WT:



① FTG

$WT \doteq (0.150)(3)(20)(20) = 180^k \quad (H^{cg} = 0.0)$

② $H = 0 \sim 18'$

$WT^{conc} = (0.150) \times [(1')(2.67 \times 3) + (1.5)(1.75) + (1.5 + 0.75)(2.75 \times 2) + (1.5)(4.21)](2)(18)$

$= 159^k \quad (H^{cg} = 9')$

$W^{soil} = (0.110)(9.5)(5) \times (18) = 94^k \quad (H^{cg} = 9')$

③ $H = 18 \sim 38.25'$

$W_T^{①} = (0.150)[(1.25)(1.5)(2) + (2)(5')](20.25) = 42^k \quad (H^{cg} = 28')$

$W_T^{②} \doteq (0.150)(20')(8.75')(2.0) = 53^k \quad (H^{cg} = 37')$

↳ PLATFORM

④ $H = 38.25' \sim 56'$

$W_T^{①} = (0.150)[(1.1)(1.1)(2) + (3.9)(1.8)] \times (17.75) = 26^k \quad (H^{cg} = 47')$

$W_T^{②} \doteq (0.150)(20')(8.75')(2.0') = 53^k \quad (H^{cg} = 48.5')$

↳ PLATFORM

SEISMIC COEFFICIENT

PER ASCE 7-05
TABLE 15.4-2

$C_s = \frac{S_{DS}}{(R/I)} = \frac{1.16}{3/1} = 0.387 > 0.03$

$C_s > \frac{0.8 S_1}{(R/I)} = \frac{0.8 \times 0.67}{(3/1)} = 0.179 \quad$ o.k.

OVERTURNING MOMENT

$$M_{OT} = 0.387\left[(180^k)(0') + (159^k)(9') + (94^k)(9') + (42^k)(28') + (53^k)(37') + (26^k)(47') + (53^k)(48.5')\right]$$

$$= 0.387 \times (9206.5^{k-'}) = 3563^{k-'}$$

RESISTING MOMENT

$$M_R = (11.25')\left[180^k + 159^k + 94^k + 42^k + 53^k + 26^k + 53^k\right]$$

$$= 6829^{k-'} > 3563^{k-'}$$

NO TIPPING

$$6829 \times (0.9 - 0.2 \times S_{DS}) = 4562^{k-'} > 3563^{k-'} \quad \text{O.K.}$$

1.16

Appendix E – Earthquake Damage to Building Similar to Natatorium Building

(Imperial County Service Building, 1979 Earthquake Damage)

Photo 1 – Imperial County Service Building, 1979 Earthquake Damage

Photo 2 – Imperial County Service Building, 1979 Earthquake Damage

Photo 3 – Imperial County Service Building, 1979 Earthquake Damage

Photo 4 – Imperial County Service Building, 1979 Earthquake Damage

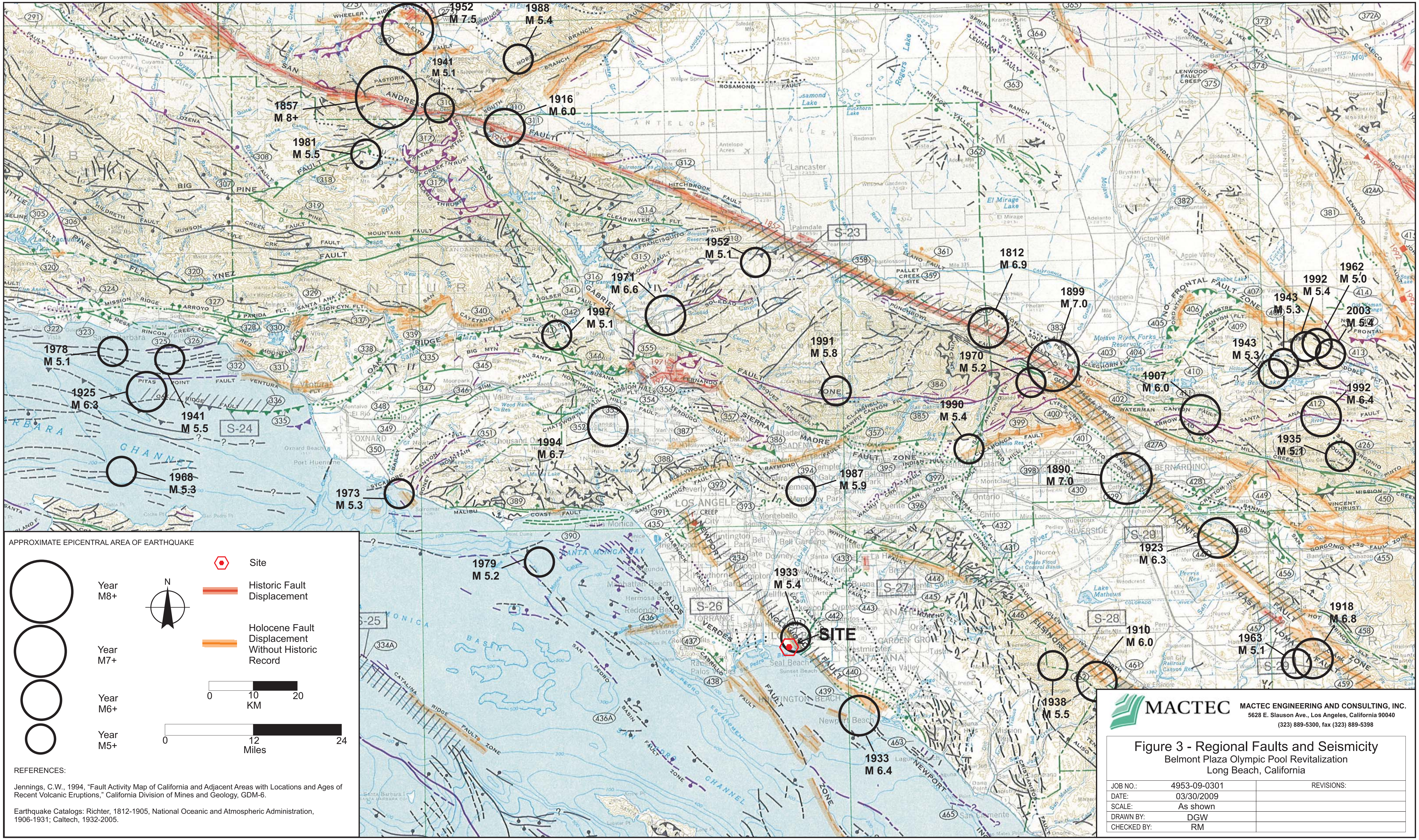


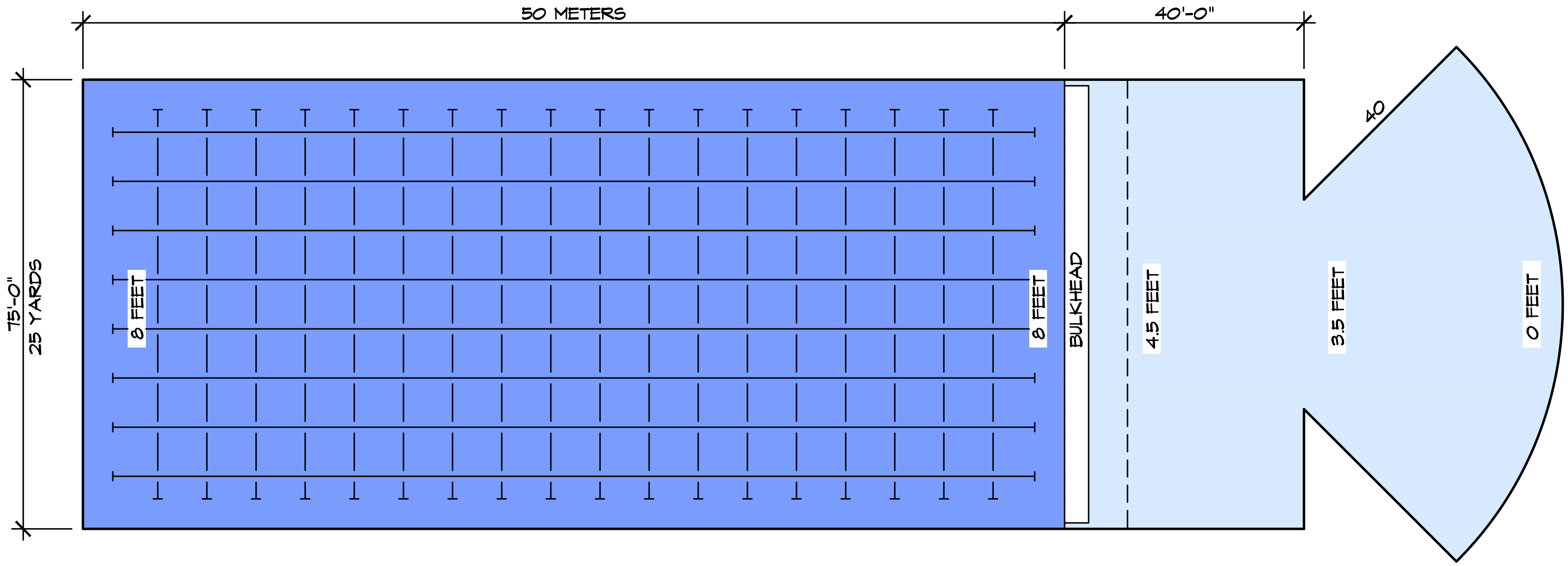
Figure 3 - Regional Faults and Seismicity
Belmont Plaza Olympic Pool Revitalization
Long Beach, California

EXHIBIT C

PROPOSED LONG TERM CONCEPT
BELMONT PLAZA POOL REPLACEMENT
CITY OF LONG BEACH, CALIFORNIA

RJM DESIGN GROUP

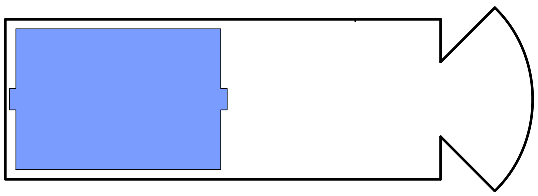
JONES & MADHAVAN ARCHITECTURE ENGINEERING



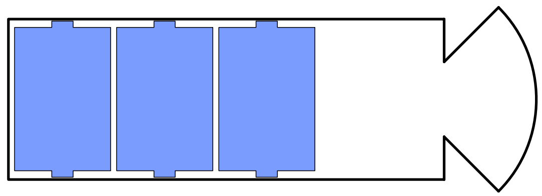
PROPOSED OUTDOOR POOL

Community Inspired Spaces
RJM DESIGN GROUP

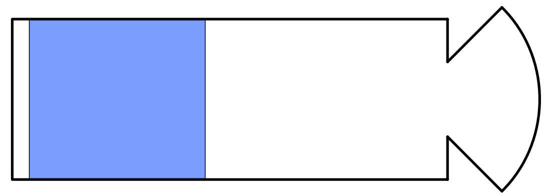
JONES & MADHAVAN
ARCHITECTURE
ENGINEERING



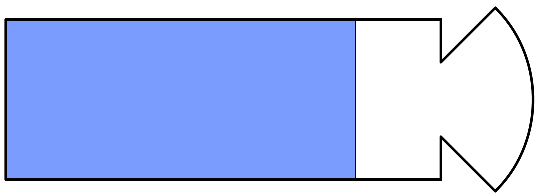
COMPETITION WATER POLO
30 M x 20 M x 8 FT DEEP
W/ FLOATING GOALS



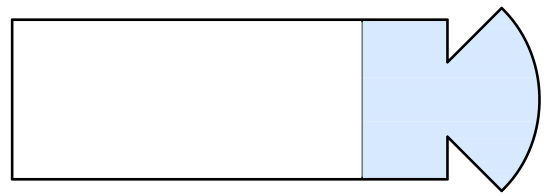
PRACTICE WATER POLO
(2) 75 FT x 45 FT x 8 FT DEEP
W/ FIXED GOALS



COMPETITION SHORT COURSE SWIMMING
(10) 25 YD LANES x 8 FT DEEP

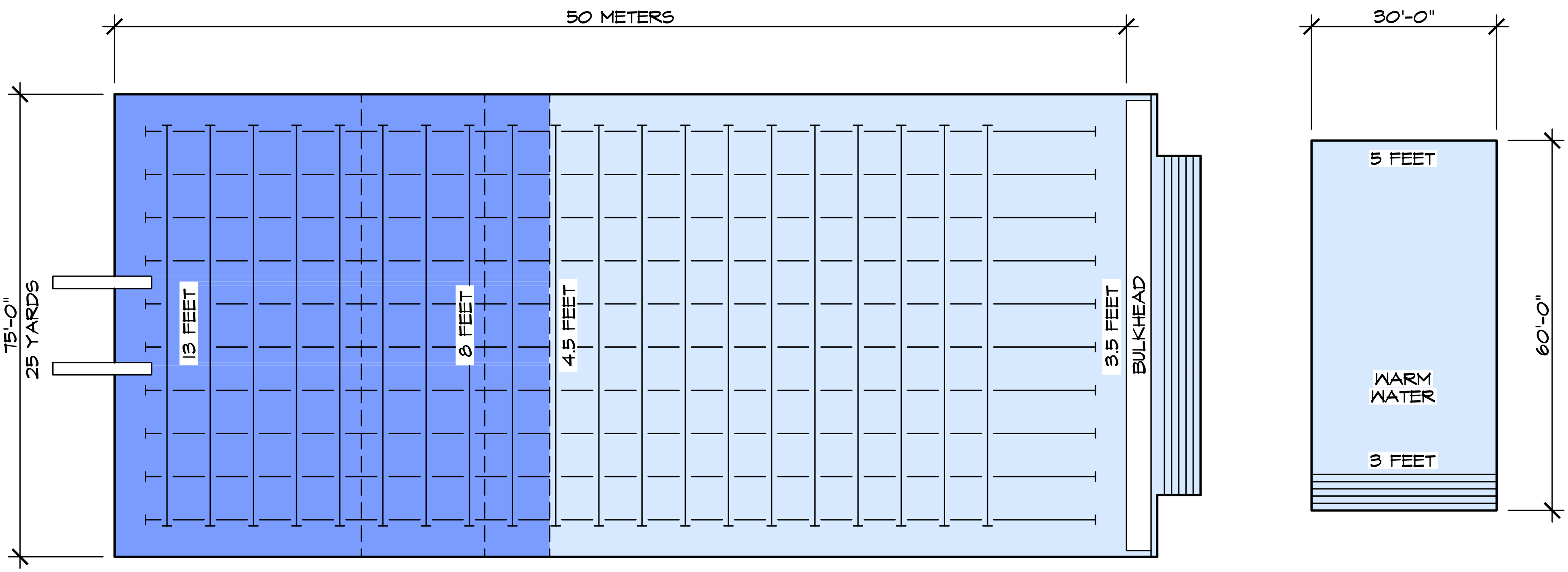


COMPETITION LONG COURSE SWIMMING
(8) 50 M LANES x 8 FT DEEP

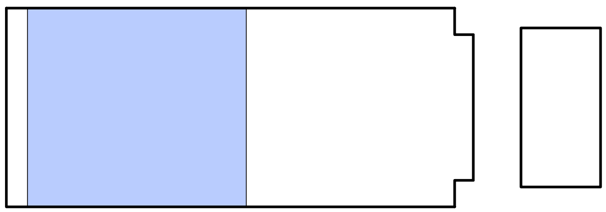


RECREATION AREA
5,200 SQ FT
0 TO 4.5 FT DEEP

PROPOSED OUTDOOR POOL



PROPOSED INDOOR POOL



COMPETITION SHORT COURSE SWIMMING
(8) 25 YD LANES x 8 FT DEEP

PRACTICE WATER POLO
75 FT x 45 FT x 8 FT DEEP
W/ FIXED GOALS

PRACTICE SYNCHRONIZED SWIMMING
25 YD x 12 M x 2 M DEEP W/
12 M x 12 M x 3 M DEEP SECTION WITHIN

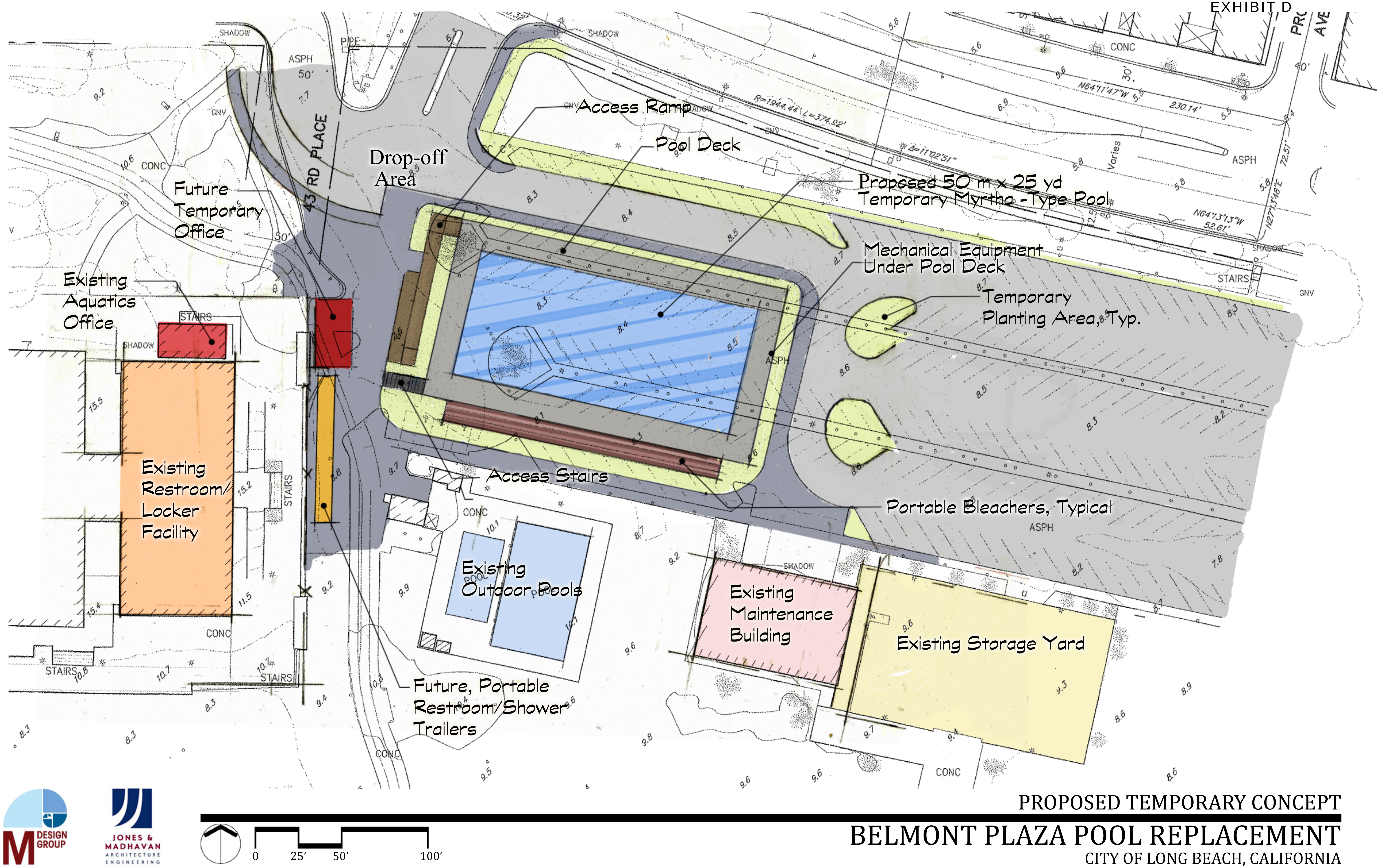
RECREATION AREA
94 FT x 75 FT x 3.5 TO 4.5 FT DEEP
30 FT x 60 FT x 3 TO 5 FT DEEP

COMPETITION LONG COURSE SWIMMING
(10) 50 M LANES x 3.5 - 13 FT DEEP

RECREATIONAL SPRINGBOARD DIVING
40 FT x 40 FT x 13 FT DEEP

PROPOSED INDOOR POOL

EXHIBIT D



PROPOSED TEMPORARY CONCEPT

BELMONT PLAZA POOL REPLACEMENT

CITY OF LONG BEACH, CALIFORNIA