LONG BEACH CITY
LIBRARY BUILDING
333 WEST OCEAN BOULEVARD
LONG BEACH, CA 90802

Life-Safety Performance Evaluation
(Based on FEMA-310 Tier 2)
Draft Report

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(Note; Detailed Results of Structural Calculations are included in a separate book of structural calculations for the same project)
I. INTRODUCTION

TMAD Taylor & Gaines (TMAD T&G) performed a preliminary seismic review of the Long Beach City Library and Park Structure Buildings following the Tier 1 criteria of the FEMA 310 (Handbook for the Seismic Evaluation of Buildings – A Prestandard) to Life-Safety Performance Level (For more details, see the Tier 1 evaluation report dated July 6, 2006) and found that the buildings do not comply with all the Tier 1 requirements of the FEMA 310 evaluation criteria. Hence, it was recommended that further evaluation to be carried out to investigate the seismic performance of the buildings to life-safety performance level following the FEMA 310 Tier 2 criteria.

The Long Beach City decided to carry out a FEMA 310 Tier 2 evaluation of the Library and Park Structure Buildings and TMAD T&G is retained to do the seismic evaluation following the guidelines of FEMA 310 Tier 2 criteria to Life-Safety Performance Level.

The Tier 1 evaluation report recommended carrying out a FEMA 310 Tier 2 evaluation and stated that due to the non-compliances in geometry, vertical discontinuity and torsions identified in the Tier 1 evaluation, a linear dynamic procedure shall be used in order to obtain a realistic distribution of the lateral forces along the height of the building. The Tier 1 report also suggested to giving attention to the following areas in the proposed Tier 2 study.

1. The narrow strips of the Park level are connected to the Library roof (Refer to sketch SK-3 for call-out of the area). Currently, all the Park level decks (including the Library roof) have a massive mass weight due to the planting soil and trees. The effect of the irregular geometry on the distribution of the lateral forces shall be investigated.
2. The Park level deck has a 3” separation joint from the City Hall Tower (See sketch SK-3). The deformation characteristics of the Park level deck should be studied using the LDP procedure and to evaluate whether the 3” separation is enough.
3. Shear force transfer at the vertical discontinuity along grid line K between grid lines 2 to 5 in the Library building (See sketches in Appendix B and structural drawing S2.2).
4. The concrete columns and beams in the concrete moment frames that supporting the park level roof (Refer sketch SK-3 and photos).

TMAD T&G performed a seismic evaluation of the Library buildings following the FEMA 310 Tier 2 criteria with due consideration to the above Tier 1 recommendations. A deficiency-only Tier 2 evaluation that addresses only the deficiencies identified in Tier 1 are required, however we choose to perform a full-building Tier 2 analysis and evaluation of the adequacy of the lateral force resisting system. This report summarizes the work carried out and presents findings and conclusions of the evaluation work.
II. GENERAL SITE & BUILDING DESCRIPTION

The Long Beach City Library and Park Structure site is bound by Broadway Street on the north, Pacific Avenue on the east, Ocean Boulevard on the south, and Chestnut Avenue on the west. The 15 story City Hall tower building is located within the Park / Library Structure complex and seismically separated at the Park level from the rest of building complex by a 3” wide seismic joint. The City Hall tower has been evaluated for seismic performance under a separate project and is not within the scope of this evaluation.

The Library / Park Structure building, constructed circa 1974 and designed per 1970 edition of the Uniform Building Code (UBC), has 2-stories above foundation level and they are referred to as lower level and plaza level. The architectural and structural drawings are dated April 1973. The lower level floors have a footprint of about 590 feet long in the East-West direction and 400 feet in the North-South direction. The Plaza Level is 17 feet above the Lower Level and is partially occupied by the Library Building and the Park structure. See attached schematic structural plan sketches and photos at the end of the report for reference.

Gravity Load Resisting System: The gravity loads resisting system are composed of a mixture normal weight concrete flat slabs, concrete joists and concrete slab over steel beam/girder system. Both concrete columns and steel columns with concrete encasement are used to support horizontal framing.

Seismic Force Resisting System: The seismic loads are typically resisted by concrete shear walls located around the perimeter and inside of the Library building complex. Concrete moment frames provide seismic load resistance to the Park level structure above the Plaza level in areas outside the Library Roof. The concrete floors as the rigid diaphragm will distribute seismic forces to the lateral force resistant systems.

Foundation System: For the gravity load columns, the foundation system consists of square footing of various sizes and thickness (from 4’x4’x1’-6” to 16’x16’x4’-0”) and 30” diameter reinforced belled caissons with a pile cap at South/West corner of the Library building. Continuous footings are used for the walls with width varying from 2’-2” to 9’-4”.
III. DESCRIPTION OF WORK CARRIED OUT

DOCUMENTS REVIEWED

The following documents were reviewed for the present study:

1. Original structural drawings prepared by Bole & Wilson, dated April 27, 1973
2. Original architectural drawings prepared by Allied Architects, dated April 27, 1973
5. “Life-Safety Performance Evaluation Based on FEMA-310, Tier 2”, The City of Long Beach, City Hall FEMA-310 Tier-2 Evaluation” prepared by TMAD Taylor and Gaines, dated August 31, 2005

STRUCTURAL ANALYSIS AND EVALUATION WORK

Apart from reviewing the above documents, we also visited the building site to visually examine the conditions of the buildings. The buildings appear to have been constructed in accordance with the drawings made available to us, however we do not have the as-built drawings and we also did not have documents for structural modifications. Minor cracks in concrete walls and water leakages were observed in a few locations and these will be reported in later part of this report.

In order to analyze the structures following the FEMA 310 Tier 2 guidelines, we created a 3-D computer model using ETABS program based on the original structural and architectural drawings. All elements believed to be contributing to seismic resistance of the building are modeled. Most of the structural members for gravity loads are modeled. Building weight / mass are distributed as realistically as we possibly can and the accidental torsion was initially introduced by allowing a 5% eccentricity. The analysis results with 5% eccentricity show torsion irregularity exists and torsion amplification factor is calculated per FEMA 310 Section 4.2.3.2.
The maximum calculated torsion amplification factor is 2.75 for the roof level and a torsion amplification factor of 3 is used to increase the accidental torsion to 15%. Response spectrum analyses are performed using site-specific response spectrum provided by Earth Mechanics Inc. (EMI). Multi-directional seismic loading effects are combined using the SRSS procedure in accordance with Section 4.2.2.2 of FEMA 310. Results of the analysis are used to perform the FEMA Tier 2 evaluation and will be presented in later part of the report.

The building base shear force from the site specific response spectrum analysis is compared to the base shear values of the CBC 2001 and UBC 1973. The response spectrum base shear is 0.5W (W=building total mass weight), the CBC 2001 base shear is 0.20W, while the 1973 UBC base shear is 0.11W. At the time of the building design, the 1973 UBC code value is applicable and it is believed that the building is design for that base shear requirement. The site specific response spectrum base shear used in this assessment is 4.42 times that of the 1973 UBC value and is 2.50 times that of the 2001 CBC code value.

IV. EVALUATION FINDINGS

We performed a full-building Tier 2 analyses and evaluation based on the FEMA-310 recommended procedures for Life-Safety Performance evaluation, our evaluation findings are presented below.

A. REGION OF SEISMICITY

Based on the “Draft Seismic Report - NEHRP Site-Specific Acceleration Response Spectrum and Probabilistic Seismic Hazard Evaluation, The City of Long Beach, City Hall FEMA 310 Evaluations” prepared by Earth Mechanics Inc. dated April 28, 2006, the building is located in a region of high seismicity. The information in this section has been compiled from the above report.

The notable past earthquakes in Southern California area include 1933 Long Beach earthquake, 1971 San Fernando earthquake, 1987 Whittier earthquake, and 1994 Northridge earthquake. All of these earthquakes had a magnitude of ±6.3 except for the 1987 Whittier earthquake, which had a magnitude of 5.9. Southern California has experienced many earthquakes in the past due to the presence of a number of active as well as potentially active faults in the region that are capable of producing moderate to large magnitude earthquakes. By definition, an “active” fault is the one that has demonstrated surface displacement within Holocene time (about last 11,000 years) while a “potentially-active” fault is the one that has demonstrated surface displacement of Quaternary age deposits (last 1.6 million years).
The principal active faults in the Long Beach region that might contribute to the greatest ground shaking are: Palos-Verdes Fault, Newport-Inglewood Structural Zone (NISZ), and Thums-Huntington Beach (THB) fault. The other geologic hazards that are most likely to cause damage to buildings include, but are not limited to, ground fault rupture, ground shaking intensity and duration, liquefaction, land sliding, tsunamis, seiches, flooding, etc.

**Surface Fault Rupture:** It is the direct manifestation of movement along a fault, projected to the ground surface. It consists of a concentrated and permanent deformation, which could be in horizontal and/or vertical direction, of the ground surface. A ground surface rupture involving more than a few inches of movement within a concentrated area can result in major damage to structures that cross it. Under Alquist-Priolo Special Studies Zone Act, which was signed into law in 1972, the State Geologist is required to delineate special studies zones along traces of known active faults in California and, thereby, prohibit location of structures across the traces of active faults for human occupancy.

The City Hall building site is not located within known fault zones and, thus, the likelihood of surface fault rupture at the site is negligible.

**Ground Shaking:** Southern California has experienced many earthquakes in the past because of the presence of a number of active and potentially active faults in the region that are capable of producing moderate to large magnitude earthquakes. A discussion of site seismicity was presented earlier in this section.

**Liquefaction:** Liquefaction is the transformation of loose, saturated, and sandy soils under sustained strong cyclical shaking into a fluid-like condition that lacks shear strength to support building weight. The liquefaction potential increases with increase in ground acceleration and shaking duration. A structure that did not sustain damage from deformations caused by ground shaking could sustain substantial damage if there is a differential settlement or tilt of the building as a result of liquefaction.

Liquefaction susceptible, saturated, and loose granular soils, which could jeopardize the seismic performance of the building, do not exist within 50 feet under the building.

**Slope Instability, Lateral Spreading, Tsunamis, and Seiches:** Hazards due to these geologic hazards are considered nil to low at the subject site.

**Site Specific Response Spectrum**
Site specific response spectrum is developed by EMI following the procedures prescribed in FEMA 450 (FEMA, 2003). The site specific response spectrum is based on the Maximum Considered Earthquake (MCE is defined as an earthquake with a 2 percent probability of being exceeded in 50 years, which corresponds to a return period of 2475 years) and developed for
an equivalent viscous damping ratio of 5 percent. The site specific spectral acceleration values are about 12 to 15 percent lower than those obtained using mapped accelerations corresponding to the MCE and for 5 percent of critical damping provided in FEMA 450. The site specific response spectrum is used in a 3D ETABS computer model for linear dynamic analysis.

B. BUILDING SYSTEM

The main lateral force resistant system of the building can be classified as Building Type C2, Concrete shear wall building with rigid diaphragms. Concrete moment resistant frames are also used to support the Park structure above the Plaza level.

a. Load path
Compliant. The structure has a complete load path in both East-West and North-South directions to transfer seismic forces to the foundation.

b. Adjacent Buildings
Compliant. The structure is next to the City Hall Tower building at the South West corner at the Park level (Mezzanine Level of the Tower Building). There is a 3” separation between the two. The maximum displacements of both the structures under seismic force are calculated using the 3D ETABS and SAP 2000 programs and the SRSS of the displacements is less than the 3” structural separation at the location.

c. Weak Story.
Not/Appropriate. The building plan dimensions at the Roof/Park level are reduced a lot compared to the Plaza level (the first level). It is not a good measure to compare the capacity of the lateral force resisting system directly between the two levels. We performed a 3D linear dynamic analysis using site specific response spectrum and assessed the capacity of the lateral force resisting elements. Based on the results of the evaluation, some of the shear walls exceeded their shear resistance capacity.

d. Soft Story
Not/Appropriate. The building plan dimensions at the Roof/Park level are reduced a lot compared to the Plaza level (the first level). It is not a good measure to compare the stiffness of the lateral force resisting system directly between the two levels. We performed a 3D linear dynamic analysis using site specific response spectrum and assessed the capacity of the lateral force resisting elements. Based on the results of the evaluation, some of the shear walls exceeded their shear resistance capacity.

e. Geometry
Non-Compliant. The horizontal dimension of the park level varies a lot compared to the level below (Plaza level). The dimensions of the lateral force resisting system supporting
the Park level also change more than 30% compared to that of the Plaza level below. We performed a 3D linear dynamic analysis using site specific response spectrum and assessed the capacity of the lateral force resisting elements. Based on the results of the evaluation, some of the shear walls exceeded their shear resistance capacity.

f. Vertical Discontinuity
Non-Compliant. Vertical elements in the lateral force resisting system along grid line K between grid lines 2 to 5 (See SK-2 in Appendix B. See also structural drawing S2.2 for more details) are not continuous to the foundation. We performed a 3D linear dynamic analysis using site specific response spectrum and assessed the capacity of the lateral force resisting elements. Based on the results of the evaluation, concrete beams and columns supporting the discontinuous walls have adequate strength for the combined gravity and seismic force, however some of the shear walls exceeded their shear resistance capacity.

g. Mass
Non-Compliant. The mass of the Library / Park Roof are much heavier (more than 50%) than that of the floor below at the Plaza level due to the planting soils/planters and trees, which is non compliant according to FEMA-310 criteria. We performed a 3D linear dynamic analysis using site specific response spectrum and assessed the capacity of the lateral force resisting elements. Based on the results of the evaluation, some of the shear walls exceeded their shear resistance capacity.

h. Torsion
Non-Compliant. The distance between the story center of mass and the center of rigidity is more than 20% of the building plan dimensions. We performed a 3D linear dynamic analysis using site specific response spectrum and assessed the capacity of the lateral force resisting elements. The effects of horizontal torsion are considered by following the guidelines of FEMA 310 Section 4.2.3.2. The 5% accidental torsion requirements are increased to 15% based on the calculated amplification factor of about 2.75. The results of the evaluation show that some of the shear walls exceeded their shear resistance capacity.

i. Deterioration of Steel and or Concrete
Compliant. No significant deteriorations of steel or concrete are observed in the vertical or lateral force resisting elements. However, minor cracks (width less than 1/8”) were observed in some of the concrete walls and roof slabs (See Photos in Appendix A ). It is our opinion that these cracks are mainly due to long term shrinkage of concrete and are often seen in concrete walls and slabs of large dimensions without control joints to limit the effect of concrete shrinkage. Water leakages are observed in some of these cracked locations and they should be repaired using epoxy to prevent corrosion of steel reinforcement.
Damages to concrete are also observed at exterior column corners. It is difficult to say whether these damages were caused by past earthquakes or due to poor workmanship and long term exposure to the weather. It appears that the corrosion of the steel reinforcements are causing the concrete to separate from the main body of the column and these damages should be repaired using epoxy concrete or other similar materials to prevent the reinforcement inside the concrete from further corrosion.

Tiles in the Park level above the Library Roof in the circular landscaping pool area separated from the concrete slab below. This may be caused by the shrinkage of concrete slab and long term exposure to weather conditions. These should be repaired.

C. LATERAL FORCE RESISTING SYSTEM

a. Redundancy
Compliant. The numbers of lines of concrete moment frames and shear walls in each principle direction are greater than or equal to 2.

b. Moment frame column Shear Stress Check
Compliant. The shear demands in the moment frame columns supporting the Park Level are within the shear capacity of the columns based on the LDP analysis results.

c. Moment frame column Axial Stress Check
Compliant. The load demands in the moment frame columns supporting the Park Level are evaluated based on the LDP analysis results and the capacity of the columns are sufficient.

d. Moment frame column No Shear Failures
Compliant. The shear capacities of the frame columns are able to develop the moment capacity of at the top and bottom of the columns.

e. Strong Column/Weak Beam
Compliant. The moment capacity of the frame columns supporting the Park Level are 20% greater than that of the beams.

f. Beam Bars
Compliant. At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of the frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment are continuous throughout the length of the members for Life Safety.

g. Column Bar Splices
Compliant. All column bar lap splice lengths are greater than 35 bar diameter for Life Safety and are enclosed by ties spaced at or less than 8 bar diameter for life safety.

**h. Beam Bar Splices**
Compliant. The lap splices for longitudinal beam reinforcing are not located within ¼ span of the joints and are not located in the vicinity of potential plastic hinge locations.

**i. Column Tie Spacing**
Compliant. Frame columns shall have ties spaced at or less than d/4 for Life Safety throughout their length and at or less than 8 bar diameter for Life Safety at all potential plastic hinge locations.

**j. Stirrup Spacing**
Compliant. All beams shall have stirrups spaced at or less than d/2 for Life Safety throughout their length. At potential plastic hinge locations, stirrups shall be spaced at or less than the minimum of 8 bar diameter or d/4 for Life safety.

**k. Joint Reinforcing**
Compliant. Beam-column joints shall have ties spaced at or less than 8 bar diameter for Life Safety. There are no ties in the joint. However, we performed a 3D LDP analysis and based on the results, the joints have sufficient shear capacity to develop the adjoining member forces.

**l. Concrete Shear Wall Shear Stress Check**
Non-Compliant. The shear stress in the concrete shear walls exceeds 100 psi in many of the shear wall. We also performed analysis of the wall shear stress following the procedure of FEMA 310 Section 4.2 using a m factor of 2.5. The stress stresses in some of the shear walls exceed the allowable limits.

**m. Reinforcing Steel**
Compliant. The ratios of reinforcing steel area to gross concrete area are greater than 0.0015 in the vertical direction and 0.0025 in the horizontal direction. The spacing of reinforcing steel is equal or less than 18”.

**n. Coupling Beams**
Non-Compliant. The stirrups in all coupling beams over means of egress shall be spaced at or less than d/2 and shall be anchored into the core with hooks for 135o or more for Life Safety. There are no stirrups in the beams over door openings; these beams will not be adequate for acting as coupling beams. We performed analysis assuming the walls as independent walls without coupling beams.
o. **Wall Connections**
   Compliant. There shall be positive connection between the shear walls and the steel beams and columns for Life Safety.

D. **DIAPHRAGMS**

   a. **Diaphragm Continuity**
      Compliant. The diaphragms shall not be composed of split level floors.

   b. **Diaphragm Cross Ties**
      Compliant. There shall be continuous cross ties between diaphragm chords.

   c. **Roof Chord Continuity**
      Compliant. All chord elements shall be continuous regardless of change in roof elevations.

   d. **Openings at Shear Walls**
      Compliant. Diaphragm openings immediately adjacent to the shear walls shall be less than 25% of the wall length for Life Safety.

E. **CONNECTIONS**

   a) **Wall Anchorage**
      Compliant. Exterior concrete walls shall be anchored for out-of-plane forces at each diaphragm level with steel anchors or straps that are developed into the diaphragm.

   b) **Transfer to Shear Walls**
      Compliant. Diaphragms are reinforced and connected for transfer of loads to the shear walls.

   c) **Concrete Columns**
      Compliant. All concrete columns shall be dowelled into the foundation for Life Safety.

   d) **Wall Reinforcing**
      Compliant. Walls are dowelled into the foundation.
F. FOUNDATION

Liquefaction
Compliant. We reviewed the site seismic and geologic hazards information in the “Preliminary Seismic Report” (FEMA-310 Tier 1 Evaluation by EGA for the City Hall Tower Building) and “Life-Safety Performance Evaluation” (FEMA-310 Tier 2 Evaluation by TTG for the City Hall Tower Building), liquefaction susceptible, saturated, and loose granular soils, which could jeopardize the seismic performance of the building, do not exist within 50 feet under the building.

Surface Fault Rupture
Compliant. The City Library building site is not located within known fault zones and, thus, the likelihood of surface fault rupture at the site is negligible.

We also completed other foundation quick check requirements in accordance with the FEMA-310 checklist 3.8 and found that the foundations of the building are compliant with the requirements.

V. CONCLUSIONS AND RECOMMENDATIONS

Based on the findings of the FEMA-310 Tier 2 Life-Safety Performance evaluation of the building, it is our opinion that the Library and Park structure does not comply with all the requirements of FEMA-310 Tier 2 for life safety performance level.

For the non-compliances in geometry, vertical discontinuity, mass and torsion irregularities identified in the FEMA 310 Tier 1 study, we performed 3D linear dynamic analysis using the site specific response spectrum and to evaluate the capacity of lateral force resisting elements, and we found that some of the shear walls do not have sufficient capacity under design earthquake; the shear stress demand to capacity ratio among all the wall piers has an average value of 0.45 and the maximum demand to capacity ratio is 1.82.

We suggest upgrading the structure by adding shear walls at selected locations as listed below;

1. Add new shear walls in the N-S direction along line 32 between grid line D~E and G~H; total length of about 30 feet long of 12” reinforced concrete wall to achieve better seismic forces distribution and strengthen the lateral resistance in this area. This wall shall be from the roof level to the plaza level.
2. Add new shear walls in the N-S direction along line 29.8 between grid lines E~G total length of about 18” thick by 31 feet long to strengthen the lateral resistance in this area. This wall shall be from the roof level to the foundation level.

3. Add new shear walls in the N-S direction along line 12 between grid line F~J total length of about 18” thick by 48 feet long to strengthen the lateral resistance in this area. This wall shall be from the roof level to the Plaza level.

4. Add new shear walls in the N-S direction along line 8 between grid line A~B total length of about 12” thick by 19 feet long to strengthen the lateral resistance in this area. This wall shall be from the roof level to the foundation level.

5. Add new shear walls in the E-W direction along line R for about 14” thick by 10 feet long to strengthen the lateral resistance in this area. This wall shall be from the roof level continuous to the foundation level.

6. Add new shear walls in the E-W direction along line M between line 30.7~33 of about 18” thick by 40 feet long to strengthen the lateral resistance in this area. This wall shall be from the roof level continuous to the foundation level.

7. Add new shear walls in the E-W direction along line L between line 8~9 of about 18” thick by 20 feet long to strengthen the lateral resistance in this area. This wall shall be from the foundation level to the plaza level to eliminate the vertical discontinuity of the shear wall.

8. Add new shear wall along grid line K from foundation to Plaza level ideally between grid lines 2 to 4 of about 18” thick by 52 feet long to eliminate the vertical discontinuity of the shear wall.

See sketches in Appendix B for location and elevation of these proposed new walls. The exact location and length of the above proposed walls may be adjustable subject to the City’s preference and requirements.

The addition of the above shear walls increases the lateral resistance capacity of the building structure. Before the additions of these shear walls, the demand to capacity ratios of these shear walls have an average value of 0.49 and a maximum+ of 1.82. After the additions of these shear walls, the average ratio is reduced to 0.45 and the maximum ratio is reduced to 0.96. A ratio of less than 1 means performance is satisfactory according to FEMA 310 Tier 2 criteria. The addition of the above shear walls also has the benefits of reducing the torsion irregularity by 15% (as measured by the index of $\Delta_{\text{max}} / (1.2 \Delta_{\text{average}})$ under X-direction seismic load before and after the addition of the shear walls). The overall performance of the building structure under seismic load will be improved with the addition of these shear walls.
Appendix A: Photos of the Library / Park Building
Photo 1  Library view from the East with City Hall Tower in the background

Photo 2  View of the Library main entrance
Photo 3  View of the Library East end

Photo 4  View of the Library from North-East corner
Photo 5  View of the Library from North-West corner

Photo 6  View of the Library from North-West corner
Photo 7  View of the Library complex from South with City Hall Tower in background

Photo 8  View of the curved area from North-West
Photo 9  View of the curved area from South-West

Photo 10  Column at grid line J/14, concrete cover cracked
Photo 11  Column at grid line F/25, concrete cover at corner damaged

Photo 12  Column at grid line D/23, concrete cover at corner damaged
Photo 13  Column at grid D/23, concrete cover at corner damaged

Photo 14  Cracks in concrete wall near grid D/23
Photo 15  Cracks in concrete wall along line 25 near line C

Photo 16  Cracks in concrete wall along line 25 near line B
Photo 17  Cracks in concrete wall at Stair #2 near grid A/7–8

Photo 18  Crack in concrete wall near grid D/18
Photo 19  Water leakage in Park Level concrete slab/beam near gird F/14

Photo 20  Cracks in concrete slab/beam near gird F/28
Photo 21  Cracks in concrete slab/beam near gird K.2/28.1

Photo 22  Cracks in concrete roof slab at the Clear Story roof
Photo 23  Library roof planters and roof tile finishes

Photo 24  Roof tile separated from concrete slab
Photo 25  Library roof clear storey and planters

Photo 26  Beams supporting clear story at grid 5/B
Appendix B: Views of ETABS Computer Model
ETABS Computer Model 3D View 1

ETABS Computer Model 3D View 2
ETABS Computer Model Plan View – Plaza Level

ETABS Computer Model Plan View – Park Level
Appendix C: Schematic Sketches of the Library / Park Building

SK-1: Lower Level Schematic Plan
SK-2: Plaza Level Schematic Plan
SK-3: Park Level Schematic Plan
SK-4: Lower Level Schematic Plan, Proposed New Wall Locations
SK-5: Plaza Level Schematic Plan, Proposed New Wall Locations
SK-6: Park Level Schematic Plan, Proposed New Wall Locations
SK-7: Park Level Schematic Plan, Proposed New Wall Locations
SK-8: Park Level Schematic Plan, Proposed New Wall Locations
SK-9: Park Level Schematic Plan, Proposed New Wall Locations
SK-10: Park Level Schematic Plan, Proposed New Wall Locations