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SCOPING DOCUMENT
(BASIS OF DESIGN)

FOR

LOS ANGELES COUNTY
HALL OF JUSTICE

LOS ANGELES, CA

NYA # 09335.00

AUGUST 24, 2010

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1. PROJECT BACKGROUND

1.1 Building Description

The existing Hall of Justice building is located in downtown Los Angeles, California at 211 West Temple Street. The existing building was designed in 1923 by the Mechanical Department of the County of Los Angeles and constructed in 1925. The height of the building is approximately 175'-4" to the roof level and 193'-5" to the top of the canopy. The building has 14 stories, includes a basement level and has a roof penthouse. The plan dimensions of the building are approximately 225'-2" in the north to south direction and 181'-2" in the east to west direction. The building has two interior light courts that are approximately 45' by 88' in dimension and begin at the 2^{nd} level.

The proposed development of the building is to remain the same as the existing occupancy category (Class 'A' Office Building). The development of the building will include removal of the floor slabs at the 11^{th} and 13^{th} levels allowing for story heights at the 10^{th} and 12^{th} levels to be increased from 9'-6" to 19'-0".

2. EXISTING STRUCTURE DESCRIPTION

2.1 Existing Gravity Load Carrying System

The typical existing gravity frame system includes concrete encased steel beams and columns. The beams are either built up sections fabricated of steel plates and angles or standardized rolled flanged steel sections that were commonly used in 1925. The columns are built up sections fabricated of steel plates and angles. The plates and angles of the built up sections are typically connected by steel rivets commonly used at the time of construction. The concrete encasement of the steel beams and columns serves as the fireproofing. The steel beams and columns support the reinforced concrete floor slabs that range in thicknesses from 4" to 7".

2.2 Existing Lateral Load Carrying System

The existing vertical lateral force resisting system consists of 8" reinforced concrete wall piers around the perimeter of the structure and unreinforced masonry walls around the perimeter of the interior light courts. The 8" wall piers are coupled by exterior steel spandrel beams; however the connection of the beam to pier does not have the required strength and ductility to transfer moments to the wall piers and has been assumed to have limited fixity. The existing steel frame beams are riveted at both the top and bottom flanges with clip angles to the face of the steel columns and also provide a limited amount of stiffness; however the stiffness contribution of the steel frame is substantially less than the concrete and URM wall elements. The reinforced concrete floor slabs transfer the inertial seismic loads to the vertical lateral force resisting system through bearing on the existing steel columns and shear friction of the slab dowels to the exterior concrete walls.

2.3 Existing Foundation System

The foundation system consists of 3'-6" thick perimeter concrete retaining walls sitting on strip footings and isolated pad footings below all steel columns at the basement level.

3. Retrofit Design Basis and Methodology

3.1 Required Performance Objectives

The performance objectives for evaluation of the existing building and design of the seismic strengthening system, as outlined below, are intended to meet both life safety and damage mitigation objectives of the County of Los Angeles for the Los Angeles County Hall of Justice Building.

1. Insure stability (prevent collapse) of the structural system during the maximum capable earthquake.
2. Prevent falling hazards, which pose a significant life safety hazard.
3. Insure safe means of egress from the building.
4. Insure that life safety systems remain operable.
5. Maintain integrity and limit damage to the building exterior façade.
6. Limit damage to the historic interior fabric, building contents, fixtures, etc.

3.2 Design Basis for Scoping Documents

The seismic strengthening system described in the scoping documents was designed, reviewed and plan checked under the regulations of the County of Los Angeles 2002 Building Code.

1. LACBC 2002 Code Minimum Seismic Base Shear:

   Seismic Zone 4

   a. Seismic source type: B
   b. Soil Profile Type: S_c
   c. Near-Source Factors: Na = 1.0, N_v = 1.1
   d. Seismic Coefficients: Ca = 0.40, C_v = 0.63
   e. Seismic Importance Factor: I = 1.0
   f. Ductility Factor: R = 5.5
      (Retrofitted Walls as Non-Bearing Wall System. Existing Steel Frame to be used as primary gravity load carrying system)
   g. Seismic Dead Load of Building, W = 110,000 kips

Code equations for the static lateral base shear force:

$T_{\text{method a}} = C_t(h_n)^{3/4} = 0.02(175'\text{-}4")^{3/4} = 0.96$ sec
$T_{\text{method b}} = 1.34$ sec
$T = T_{max} = 1.3\ T_a = 1.3(096) = 1.25$

$$V_{static} = \frac{C_v I}{RT} W = \frac{(0.63)(1.0)}{(5.5)(1.25)} W = (0.0916)\text{W}$$ ← Governs static load

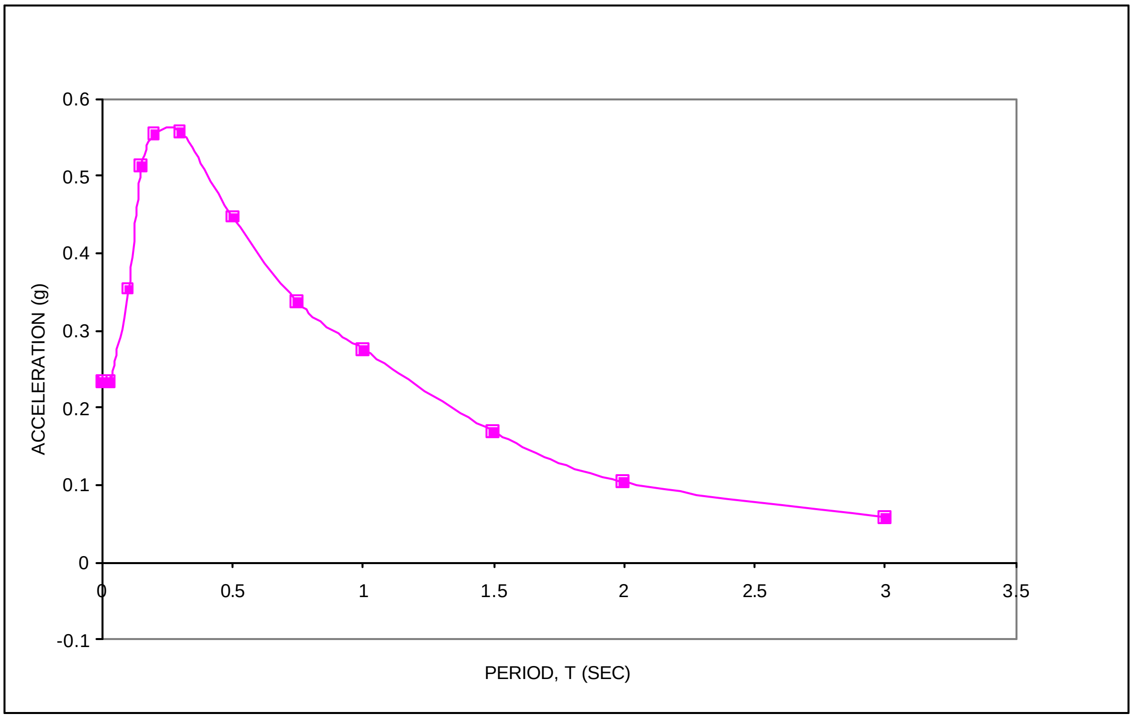
$$V_{static} < \frac{2.5 C_a I}{R} W = \frac{2.5(0.40)(1.0)}{(5.5)} W = (0.182)\text{W}$$

$$V_{static} > .11 C_a I W = .11(0.40)(1.0) W = (0.044)\text{W}$$

$$V_{static} > \frac{0.8 Z N_v I}{R} W = \frac{0.8(.4)(1.1)(1.0)}{5.5} W = (0.064)\text{W}$$

V_{static} = (0.0916)110,000 kips = 10,076 kips

A dynamic response spectrum analysis was performed using acceleration response spectra generated from velocity spectra provided in the geotechnical report. The response spectra are plotted in the following graph:



Site Specific Acceleration Spectra per Geotechnical Report

2. LACBC 2002 Wind Design
   a. Basic Wind Speed: 70 mph

b. Exposure Category: B

3. Superimposed Live Loads:
   a. Roof : 20 psf
   b. Office Floors: 50 psf
   c. Assembly Area: 100 psf

3.3 Design Basis per 2011 LACBC

The selected Design-Build Contractor will be responsible for updating the design shown in the Scoping Documents to conform to the 2011 LACBC. This code is based on the 2010 California Building Code and the 2009 International Building Code which is significantly different from the 1997 Uniform Building Code upon which the 2002 LACBC was based.

1. LACBC 2011 Code Minimum Seismic Base Shear:

a.	Site Class:	D	(To be verified by Geotechnical Engineer)
b.	Ss:	2.235	(To be verified by Geotechnical Engineer)
c.	S1:	0.759	(To be verified by Geotechnical Engineer)
d.	Fa:	1.0	(To be verified by Geotechnical Engineer)
e.	Fv:	1.3	(To be verified by Geotechnical Engineer)

f. Ductility Factor: R = 6
(Retrofitted Walls as Non-Bearing Wall System. Existing Steel Frame to be used as primary gravity load carrying system)

g. Seismic Dead Load of Building, W = 110,000 kips

Code equations for the static lateral base shear force:

$T_a = C_t(h_n)^{3/4} = 0.02(175'\text{-}4'')^{3/4} = 0.96$ sec
$T_b = 1.34$ sec
$T = T_{max} = 1.4\ T_a = 1.4(096) = 1.34$

$S_{DS} = 2/3 * F_a S_s = (2/3)(1.0)(2.235) = 1.49$
$S_{D1} = 2/3 * F_v S_1 = (2/3)(1.3)(0.759) = 0.659$

$$V_{static} = \frac{S_{DS}}{R/I}W = \frac{1.49}{6.0/1.0}W = (0.248)W$$
$$V_{static} < \frac{S_{D1}}{T(R/I)}W = \frac{0.658}{(1.34)(6.0/1.0)}W = (0.0818)W$$
$$V_{static} > 0.01W$$

$V_{static} = (0.0818)110{,}000$ kips $= 9{,}000$ kips

The estimated base shear under the 2011 LACBC is thus approximately 10% less that that of the 2002 LACBC. However, as noted above, the 2011 LACBC seismic site coefficients must be verified by the Geotechnical Engineer. Also, there are several significant differences between the two codes which will require the design of the seismic strengthening to be verified/modified for compliance to the 2011 LACBC. These differences include, but are not limited to the following:

a. Calculations of redundancy.
b. Calculation of torsional irregularity effects.
c. Diaphragm shear phi factor.

2. LACBC 2011 Wind Design

a.	Basic Wind Speed:	100 mph 3-second wind gust
b.	Exposure Category:	B
c.	Wind Directionality Factor:	Kd = 0.85
d.	Importance Factor:	I = 1.0
e.	Design Wind Load:	p = qGfCp

3.4 Local Faults

Southern California is traversed by several active faults that are capable of producing moderate to large magnitude earthquakes. Figure 3.1 shows the major faults affecting the property. The principal faults are **Hollywood, Raymond, Newport-Inglewood, Elysian Park and Verdugo**. The historical data available on each of these faults was reviewed.

Table 3.1 - Major Active Faults Affecting the Site

Fault	Maximum Credible Earthquake Magnitude (Richter Scale)	Recurrence Interval (Years)	Distance to Site (miles)
San Andreas-			
(San Bernardino	7.3	433	36
segment)	6.4	626	4.2
Hollywood	6.5	1541	5.1
Raymond	6.9	1006	6.3
Newport-Inglewood	6.7	549	6.9
Elysian Park	6.7	1608	7.2
Verdugo			

Reference: Fault Activity Map of California, State of California and Peak Acceleration from Maximum Credible Earthquakes in California, CalTrans.

Although the subject-building site may experience moderate ground shaking from earthquakes on a number of fault segments, the fault segment that is likely to generate the strongest ground shaking is the **Hollywood Fault** Segment. Such an earthquake is likely to result in a Modified Mercalli Intensity (MMI) ranging between VII to VIII (7 ~ 8) at the building site

Based on an increased knowledge of ground motions caused by earthquakes, currently adopted building codes have established values for strength and displacement that exceed the capacity of the current building configuration as evident by damage sustained during past earthquakes and as could be expected to occur during future earthquakes. The current building configuration does not have a well-developed system to meet the code-based values for strength and displacement.

4. ANALYSIS OF EXISTING STRUCTURE AND PROPOSED RETROFIT SCHEME

4.1 Seismic Performance of Existing Lateral System

Although the existing lateral force resisting system is inherently redundant, it is evident that the existing wall and slab elements do not meet the current code level strength, ductility, and detailing requirements. The primary lateral resistance in the existing structural system is offered by the exterior 8" concrete wall piers, which are coupled by the exterior steel spandrel beams. The 8" wall pier centerline is eccentric to the centerline of the steel spandrel beam with the inside face of the wall typically aligning with the outside edge of the steel beam flange. The existing concrete wall piers are typically connected to the diaphragm with #3@16+/-"O.C. slab dowels; however the pier is only bonded to the side of the concrete encased steel beam with unreinforced concrete.

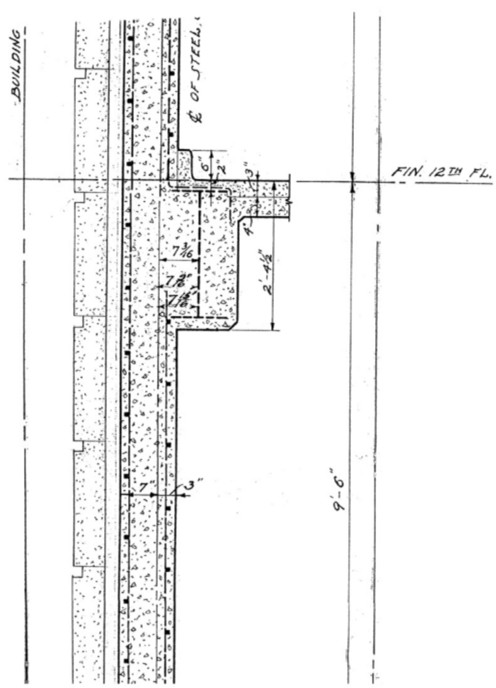
The concrete connection has very little rotational capacity and ductility to transfer moment between the concrete pier and concrete encased beam; with torsional fracture occurring in the unreinforced concrete, at an interstory drift of 0.25%+/-, thus decoupling the pier and spandrel beam in flexure. The #3 slab dowels provide the primary vertical and lateral support to the concrete pier after debonding of the concrete to the steel beam. The decoupled piers behave as slender concrete walls cantilevering from the ground to the roof, offering very little stiffness with little ability to control the drift in the building. The unreinforced masonry infills in the light court also offer significant initial stiffness to the building; but after cyclic loading and drifts in excess of 0.5% will crack with a significant loss in strength and stiffness. Redistribution of seismic force to the existing steel frames will occur after the loss of strength and stiffness in the existing wall elements with interstory drifts exceeding 2%; causing the existing stone veneer and other secondary structural systems to exceed their failure limit states.

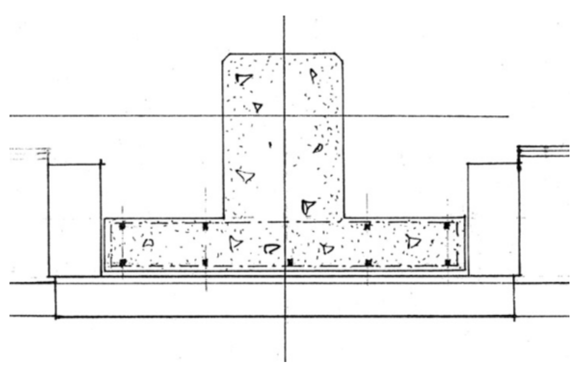
4.2 New shear Wall Strengthening

The proposed seismic strengthening consists of new shear walls from the basement to roof at the exterior corners of the building, with the vertical wall piers coupled by new concrete spandrel beams to form a cantilever wall system. The thickness of the walls varies from 18" at the first floor to 14" at the top of the building. As mentioned in the previous section, the existing building lateral resisting system (exterior 8"wall piers and interior light court URM walls) has very little stiffness and lateral capacity after interstory drift levels have exceeded 0.5%; thus the new shear walls will be designed to resist 100% of the design base shear. The new shear walls will also provide enough stiffness to limit the interstory ratios to 1.0%+/- (acceptable drift performance level after the exterior stone backing systems and interior URM walls are strengthened).

Exterior Stone Cladding System

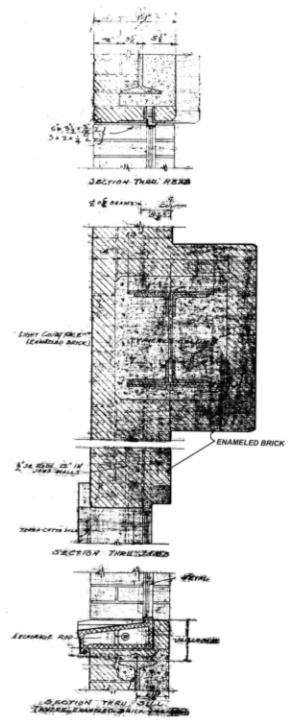
The existing 8" concrete piers will act as a concrete panel veneer backing system with interstory deformation capabilities in excess of 1%. The existing stone panels will be anchored to the 8" concrete piers using Helifix anchors to insure a reliable connection between the stone and the concrete.





URM Walls

The URM in the light courts will also be treated as a veneer and strengthened with a strong back tube system and/or glass fiber backing system. This will allow the URM to exceed lateral deformations of 1.0%, without becoming a falling hazard.



Existing Steel Frame:

The existing steel frame system in the building consists of built-up wide flange columns with wide flange beams riveted to the columns with clip angles at the top and bottom flanges.   This type of connection will behave similar to a partially restrained (partially rigid) moment connection with limited rotational fixity. The stiffness contribution of the steel frame system however will be very low (less than 10%) with respect to the new shear wall system.

Foundations

The existing foundations will be verified to meet the strength objectives required to transfer the seismic forces from the new shear wall system to the bearing strata below. The new shear wall strengthening will be epoxy doweled into the existing basement walls and foundation walls below. This will allow the shear walls to engage the foundations through the entire perimeter of the building by coupling the basement walls as deep beams. The basement walls will be strengthened where required. The existing continuous wall footings will be expanded to reduce bearing pressure under gravity and seismic loads to the allowable soil bearing pressure per the geotechnical report.

5. EXISTING AND NEW MATERIAL PROPERTIES USED FOR DESIGN AND ANALYSIS:

Typical Exterior 8” Concrete (145#/ft^3) Wall Panels…………3,200psi
Typical 4”-7” Concrete (130#/ft^3) Slabs………………………3,200psi
All Other Concrete (130#/ft^3)………………………………………3,200psi
Existing A15 Billet Steel Reinforcing Bars…………………..42,000psi
ASTM A9 Structural Steel Yield Strength……………………30,000psi
ASTM A9 Structural Rivet Steel Yield Strength……………..25,000psi
New Lightweight Concrete (110#/ft^3) Slabs……………………3,000psi
New Concrete (150#/ft^3) Shear Walls and Drag Beams
Above Tenth Floor…………………………………………..6,000psi
Below Tenth Floor…………………………………………..8,000psi
All Other New Concrete (150#/ft^3)……………………………..4,000psi
New Reinforcing Steel – A615-Grade 60……………………..60,000psi
New Weldable Reinforcing Steel-A706 -Grade 60……………60,000psi