Report of Results
from
Structural Analysis and Evaluation
of
Existing Cupertino City Hall

Examining Structure's Compliance with 1985 Uniform Building Code as an Essential Facility

Prepared for

The City of Cupertino

November, 2011

Prepared by

AKH Structural Engineers, Inc.
Consulting Structural Engineers
1505 Meridian Avenue, Suite B
San Jose, California 95125
Ph: (408) 978-1970, FAX: (408) 267-7919
Web: www.AKHSE.com

AKH Project No. M11-040

CONTENTS

Introduction

The original Cupertino City Hall building was designed by San Jose architect, Wilfred Blessing, in 1965. Mr. Blessing retained the firm of Kirk McFarland Engineers, Inc. (KME) to perform the structural engineering, in accordance with the 1964 Uniform Building Code. A building permit was issued on December 2, 1965 and construction by Pursely Construction Company of Sunnyvale was completed a year later on November 19, 1966, at a cost of $433,600. Notice of Completion was filed with the Santa Clara County Recorder's office on December 2, 1966.

Two employees of KME at the time were Dennis Ahearn and William (Bill) Knox. Bill Knox prepared a significant portion of the structural calculations for the building, and prepared and supervised the preparation of the structural drawings, from which the building was constructed.

In 1970, Dennis Ahearn left KME and opened his own engineering firm. In 1973, Bill Knox joined Dennis Ahearn's firm as a partner, at which time the firm became Ahearn and Knox, Inc. In 1983, Tim Hyde joined the firm, later to become a partner in 1993, after which, the firm's name became Ahearn, Knox & Hyde, Inc. The firm is in business and practicing structural engineering in San Jose today, now known as AKH Structural Engineers, Inc. (AKH). All three partners are still active in the firm.

In 2005, the Cupertino City Architect, Terry W. Greene, contacted Bill. Knox to investigate the feasibility of adding a second floor to the existing building, taking into account the 1986 remodel and structural upgrades provided by the architectural firm of Holland, East & Duvivier (HED), with Cygna Consulting Engineers of San Jose, as the structural engineer of record. The 1986 upgrade had been undertaken to qualify as an Essential Facility, meeting the requirements for such in the 1985 Uniform Building Code, to allow for the inclusion of an Emergency Operations Center to be constructed in the then-unused basement.

Mr. Greene sought out Mr. Knox for his structural expertise through the architectural firm of Sugimura & Associates Architects, located in Campbell, California, not knowing that Mr. Knox had been directly involved in the design of the original building. After learning of Mr. Knox's involvement in the original design, his continued review of the 1986 remodel was deemed very valuable. Bill Knox prepared a report of the City Hall building's seismic capacity on April 6, 2006 and delivered it to Gene Sugimura who then provided it to Terry Greene at City Hall.

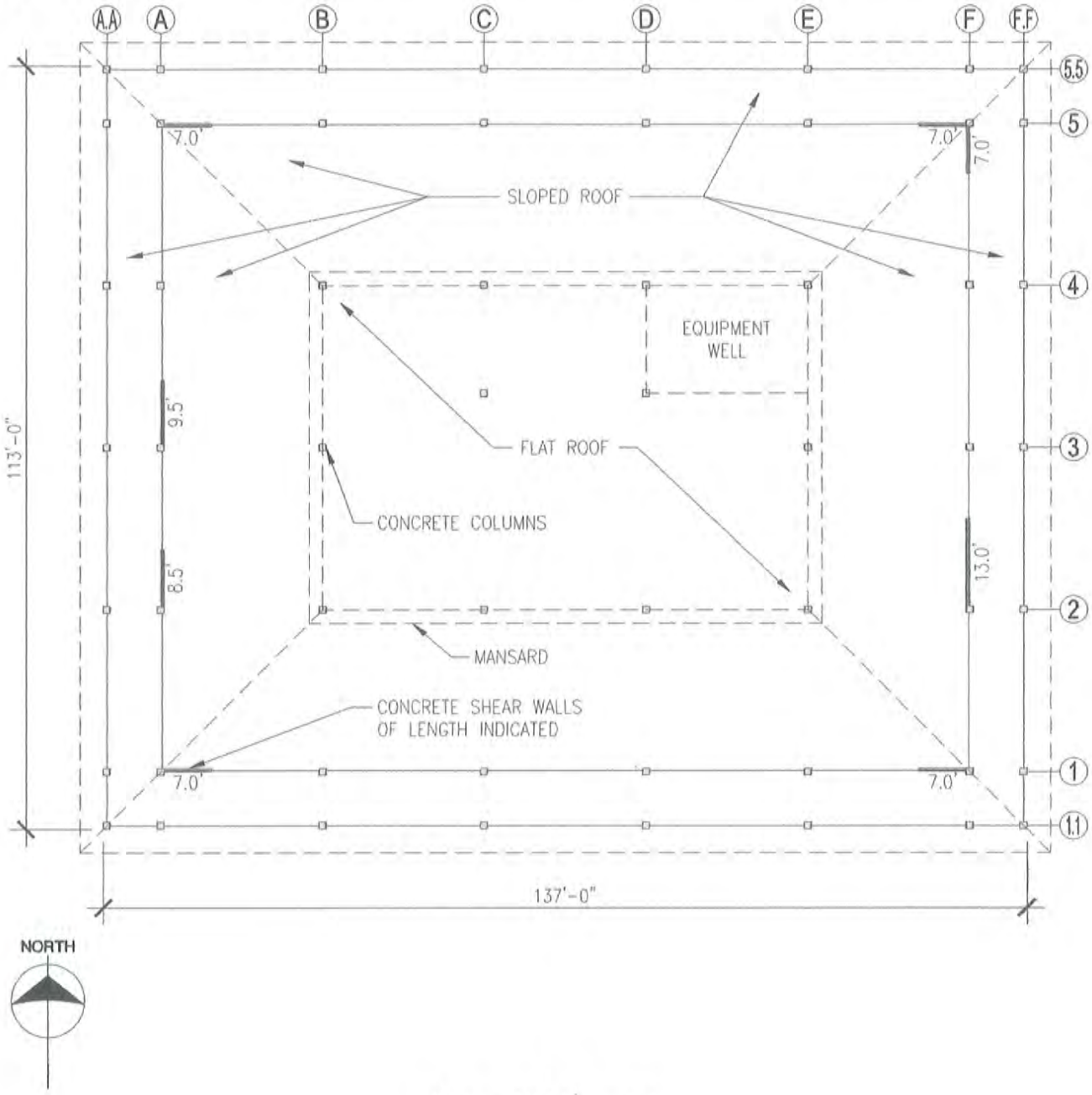
Mr. Greene received a preliminary report from Bill Knox in November of 2005, which became the basis for a staff report by Mr. Greene to the Director of Public Works in December of 2005. Mr. Knox later produced a re-cap of the report, in April of 2006 and transmitted that re-cap to Gene Sugimura. Mr. Sugimura then transmitted the re-cap with calculations, to Mr. Greene on April 6, 2006.

Between April 2006 and July 2011, the City of Cupertino did not act on the recommendations in Mr. Knox's report. In June of 2011, the City approved a Civic Center Master Plan project for the FY 2011/2012 CIP. Mr. Greene subsequently contacted Tim Hyde of AKH to provide a new, comprehensive review of the City Hall building's seismic capacity, especially with regard to the compliance with the 1985 Uniform Building Code, as required for an Essential Facility.

Results and findings from the recent, 2011 analysis follow below. The analysis includes the review of construction data that was not utilized in the 2005 review.

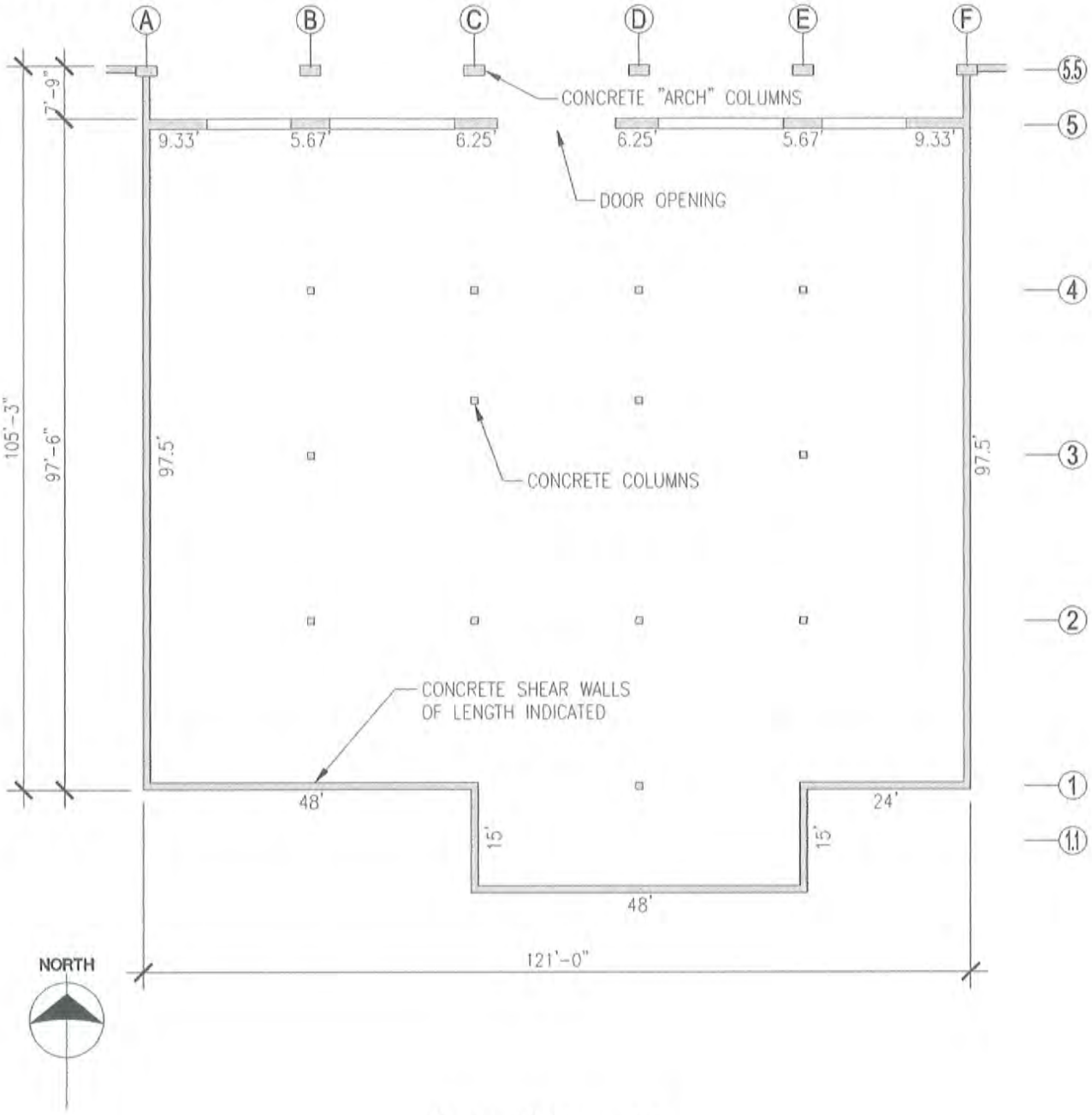
Key Plans

Upper Level:



(Note: Not to scale)

Lower Level:



(Note: Not to scale)

Seismic Analysis - Approach and Applied Concepts

Original Design Engineer's Project File:

As indicated above, Bill Knox and Dennis Ahearn were former employees of Kirk McFarland Engineers, Inc., the Structural Engineering firm that performed the original design of the City Hall building in 1965. Fortunately, KFE's project file for the building's original design was given to Bill Knox and Dennis Ahearn several years ago. Thus, we have some original calculations, testing results, shop drawings and other miscellaneous written correspondence for the project's original design and construction. As part of the project file, we also have the original structural drawings, dated 1965.

We have been able to utilize these documents, including the calculations and shop drawings, to help determine and confirm certain properties of the building, such as material weights and original design assumptions and loading.

Documents for the 1986 Building Alterations:

We are in possession of the structural drawings and calculations prepared at that time by Cygna Consulting Engineers, and have referenced these documents when necessary to confirm certain conditions and design intent.

Structural Modeling:

For our analysis, the Cupertino City Hall structure was modeled in the ETABS computer program, which is a Finite Element Analysis (FEA) application. The model was utilized to determine the seismic forces distributed to the concrete shear wall elements above and below the grade-level floor slab. The use of the FEA program is not inconsistent with the design requirements in the 1985 Uniform Building Code. The program allows for more accurate modeling of the various wall elements, especially those with openings along grid lines 5 and 5.5.

As the wood-framed roof is considered a "flexible diaphragm," the lateral seismic forces at the Upper Level were determined outside of the ETABS program by separate calculation, based on tributary areas to the wall lines. Those forces were then applied to the lines of shear walls in the ETABS model. ETABS distributes those forces to the walls along each wall-line according to the walls' relative rigidities. As steel beams occur along the wall lines, act as collectors/drag struts and connect all of the walls together, this assumption is accurate.

The Lower Level seismic forces were applied to the ETABS model at the grade-level slab, and the total of all forces were then distributed to the all of the Lower Level concrete shear walls, based on their relative rigidities, resulting from direct forces and accidental torsion forces, as required by the 1985 UBC for rigid diaphragm structures.

The walls along grid lines 5 and 5.5 were modeled to reflect their actual configurations, with regard to openings, thicknesses and support conditions.

Building Codes:

The scope of this evaluation includes assessing the structure using seismic forces required in the 1985 Uniform Building Code (UBC), as this was the Code to which the 1986 alterations were designed. As we know, the Building Code has evolved and undergone numerous revisions since that time. As requested, this report presents the results of our analysis with respect to the 1985 UBC. However, in order to provide an overall comparison, the primary force levels required in the 1985 UBC will be compared to the analogous forces required in the current, 2010 California Building Code (CBC).

Seismic Force Levels:

Simply, the lateral seismic force, or Base Shear, for which a building is designed, is merely a percentage or fraction of the building's weight. That percentage is determined by several factors, including (a) the seismic zone in which the building is located, (b) the geologic conditions and soil types present at the site, (c) the building's structural systems that resist the seismic forces, and (d), the intended of use or occupancy of the building. Essential Facilities have occupancies that affect this portion of the Base Shear equation.

Essential Facilities and Seismic Importance Factors:

Essential Facilities are those that must remain operational for emergency purposes, such as after a major earthquake or other disaster. These structures would be designed to resist higher seismic forces and are reflected in the Importance Factor, which is applied to the overall Base Shear force. The Importance factor for most residential and commercial buildings is 1.0. The Importance Factor for Essential Facilities is typically higher than 1.0.

From the 1976 UBC to the 1985 UBC, the Importance Factor for an Essential Facility was 1.5. In the following, 1988 UBC, **the Importance Factor was reduced from 1.5 to 1.25,** effectively reducing the seismic forces for which this type of Essential Facility would be designed, as compared to the previous 1985 UBC. The Importance Factor for Essential Facilities remained 1.25 for the primary elements in the seismic-force-resisting systems, until the 2007 CBC when it changed back to 1.5 and remains as such in the present 2010 CBC.

Existing Concrete Strengths:

The McFarland project file contains numerous concrete compression test results for the various concrete elements in the building, at both the upper and lower levels. The minimum design compressive strength for the structural walls was specified to be 3,000 psi. All of the test results for the wall concrete indicate that the lowest compressive strength at 28 days is 3,490 psi, with the mean value being over 4,620 psi. Based on these results, we have used a concrete strength of 3,500 psi throughout our analysis. Where deficient conditions are found to exist, they were based on this concrete strength. The yield strength used for the reinforcing was 40 ksi. This value is confirmed by test results for the reinforcing found in the original project file.

Comparison of Required Seismic Design Forces in 1964, 1985 & 2010 Building Codes:

1964 Uniform Building Code:

Force levels expressed at Allowable Stress Design (ASD) levels.
W = Weight of structure.

Upper Level: $F_{EQ} = 0.133 \times W =$ **0.13 x W at Upper Level**
Lower Level considered subterranean and not included in seismic design.

1985 Uniform Building Code:

Force levels expressed at Allowable Stress Design (ASD) levels.
Building designed as an **Essential Facility**; Importance Factor, I = 1.5.

Upper Level: $F_{EQ} = 0.14 \times 1.5 \times W =$ **0.21 x W at Upper Level**
Lower Level: $F_{EQ} = 0.19 \times 1.5 \times W =$ **0.28 x W at Lower Level**

2010 California Building Code, which References ASCE7-05:

Force levels expressed at Allowable Stress Design (ASD) levels.
Redundancy factors of 1.3 for Upper Level and 1.0 for Lower Level are included.
Essential Facility, reflected in Occupancy Type and Structural Design Categories.

Upper Level: FEQ = 0.21 x 1.3 x W = **0.27 x W at Upper Level**
Lower Level: FEQ = 0.27 x 1.0 x W = **0.27 x W at Upper Level**

Comparing Base Shear force magnitudes for 2010 CBC over 1985 UBC:
For the Lower Level, there is no increase in applied seismic force

Varying Lateral Force-Resisting System Types:

The 1988 UBC, issued immediately subsequent to the 1985 Code, included certain provisions for the vertical distribution of seismic forces when the lateral force-resisting systems varied at each level. In this building, the upper-level shear walls do not support a significant portion of vertical load, whereas the shear walls at the Lower Level (Basement) do support significant vertical dead and live load. This difference results in different overall seismic factors for each level. The 1988 UBC and later Codes require that the seismic forces from the upper level be scaled up in proportion to the corresponding seismic factors when applied at the lower level. However, the 1985 UBC did NOT include these provisions. Thus, our analysis does not include these provisions, even though the approach required in the current Code would require this more-conservative scaling of forces.

Vertical Re-Distribution of Seismic Forces:

The Building Code requires that a building's seismic forces be distributed over its height, based on a weighted average of each level's combined weight and height above the base. This is a generally accepted means of accounting for the momentum generated by the upper levels moving in an earthquake, perceived as a "whipping" action.

For structures that have levels substantially below adjacent grade, those subterranean levels are not included in the vertical re-distribution as their lateral deflections are dampened by the surrounding earth. In addition, the subterranean portions are typically much more rigid laterally than the superstructures above. For these reasons, such a building's base is considered to be at grade level for the purpose of vertical force re-distribution.

The 1986 alterations to the subject building exposed one full side of the structure that had previously been below adjacent grade. One could argue that the building's lower level should subsequently NOT be considered subterranean, and that the structure's lower level should be included in the vertical distribution of forces.

The stiffness of the lower level, with long shear walls, is substantially higher than the upper level, which has relatively short shear walls and a lower overall stiffness by inspection. In addition, for the most critical wall along grid line 5, which was exposed by excavation and received multiple openings in the 1986 alterations, would receive lateral, in-plane seismic forces when the lateral, seismic forces are acting on and perpendicular to the sides that remain below adjacent grade, moderating the lateral drift and the forces reaching the wall on line 5. Also, seismic forces acting perpendicular to the wall on grid line 5 would be resisted by the long, rigid shear walls at lines A and F, and would not be resisted by the altered wall on grid line 5.

For these reasons, the subterranean, lower level is not included in the re-distribution of lateral seismic forces. A review of Cygna's calculations confirms that this was the same methodology used in their analysis, performed in 1986.

Apparent Errors Found in 1986 Analysis and Design:

The design calculations for the 1986 alterations utilized incorrect tributary areas and/or unit-weights to determine tributary building masses, resulting in calculated masses at both the Upper and Lower levels that were significantly less than represented by the actual conditions. These masses relate directly to the seismic forces for which the building was designed to resist, including the shear walls and the diaphragms.

For example, the effective areas of roof that include the heavy roof tiles were significantly less than the actual areas tributary to each shear wall line. In addition, the six-foot-high parapet, which also supported the heavy roof tiles, was ignored completely in the determination of the building mass at the Upper Level. Also, at both levels, the masses of the concrete walls perpendicular to the direction of seismic force in consideration were not included in the effective building mass.

The 1986 analysis incorrectly calculated the respective masses as follows:

- Upper Level: Underestimated mass by 45% to 59%, depending on direction considered
- Lower Level: Underestimated mass by 24% in either direction considered
- With respect to the overall building, the underestimates amount to approximately 34%.

Analysis of Existing Concrete Floor Girders

Determination of Possible Causes of Visible Cracks:

Some narrow cracks are apparent in the existing cast-in-place concrete floor girders at grade level, over the basement. None of the visible cracks are of widths that are considered significant with respect to the members' ability to resist the supported loading.

We have analyzed the subject girders as well as the smaller concrete joists that are supported by the girders.

It should be noted that reinforced concrete members MUST crack in order to facilitate load transfer to the steel reinforcing, and for the steel reinforcing to provide the resistance to applied loads as designed. Typically, the cracking that occurs is spread throughout the concrete members, and thus, the cracks are smaller than what can be observed visually. In other cases, certain conditions can cause a concentration of strain and the resulting release of stress causes visible cracks. However, visible cracks, in of themselves, are not necessarily indicative of overstressed or deficient conditions.

Our analysis considered the required loading and design concepts in both the 1964 UBC, with regard to the original design of these members, and with regard to the current 2010 CBC, in order to ascertain any possible design criteria or other considerations that exist in the current Code, but that might not have been accounted for over 40 years ago.

There have been some minor revisions over time in the Code-required vertical loads for which these floor members would be designed. These include uniform loading for things such as partitions, live loads for certain occupancies and uses, and live load reductions allowed based on tributary areas being supported by the respective members. In general, the design loading in exit facilities, such as corridors and lobbies, have remained constant at 100psf. Live loads for office areas have also remained constant at 50 psf. Live loads in assembly areas with fixed seating have increased from 50 psf in the 1964 UBC to 60 psf in the 2010 CBC. Finally, the 1964 UBC did not require loading for movable partitions, where the current 2010 CBC requires a uniform load of 15 psf live load for moveable partitions. However, as was common at the time, the design of these members did include partition loading of 20 psf overall office areas.

There have also been significant revisions in the analysis and design of concrete members since the 1964 UBC. The subject concrete members have been examined in terms of the design concepts used in the 1964 Code, as well those used as the 2010 CBC, in order to determine what conditions, theoretically and practically, might be the cause of the cracks.

Concrete Joists:

The analysis of the concrete joists indicated that their capacities in shear and flexure are adequate for all loading conditions, in accordance with the 1964 and 2010 Codes.

Concrete Girders:

The girders were analyzed closely, with the appropriate magnitudes of loading applied as accurately as possible. All four lines of girders have been analyzed. In all cases, the girders have been found to be adequate to resist the calculated flexural moments (bending forces).

With regard to vertical shear in the girders, our analysis considering the 1964 UBC indicates that the amount and/or spacing of the shear reinforcing (stirrups) is adequate in all locations.

However, the intent here is to identify possible causes for the apparent cracks. Thus, we also analyzed the member shear according to the 2010 CBC requirements. In that case, our calculations indicate that the amount and/or spacing of the shear reinforcing (stirrups) is inadequate in certain locations. However, closer inspection of the Code requirements indicate that the shear reinforcing is adequate in the majority of cases, when based only on the actual shear stresses in the members.

Certain prescriptive requirements exist in the current CBC that were not included in the 1964 UBC. These requirements specify a minimum amount of steel shear reinforcing (ties or stirrups), which is based on the maximum stirrup spacing, and is dependent primarily on the members' cross-sectional areas, without regard to actual shear forces or internal stresses.

The magnitudes of girder shear are typically the highest near the supporting columns. Of the four north-south floor girders, according to the 2010 CBC analysis, the areas with excessive stirrup spacing occur along one girder on grid line C, which has a longer interior span between supporting columns, and supports areas of higher live load than the other girders. The three other girders also have some areas of excessive stirrup spacing based on the prescriptive requirements, but they are limited to areas immediately close to the columns.

Total shear resistance in concrete members is provided by two primary components. The first is the concrete itself, where the shear capacity is based on the concrete strength and the member's cross-sectional properties. The second component is the steel shear reinforcing (stirrups). The amount of reinforcing provided can be affected by the size of the bars used in the stirrups, and the spacing of those stirrups along the member's length. These two components combine to provide the total shear resistance, or shear capacity.

In some locations, typically at the ends of the longer girder spans, the spacing of the shear stirrups in the girders is excessive where based on the actual, calculated shear capacity needed in the girder. It should be noted that this is a calculated deficiency only when the applied loading represents the full live load over the majority of the member's tributary area. In the locations where the calculated shear demand exceeds the capacity, the girder is overstressed in shear by up to 21% in areas near two of the supporting columns. The overstress results from the shear stirrups being spaced too far apart in order to provide the necessary total shear capacity. In these subject areas, the spacing of the stirrups is approximately two-times the spacing needed to provide an adequate total shear capacity to meet the demand.

Possible Causes of Cracking:

Based on our close examination of the joists and girders, and the calculated flexural and shear stresses within the members, we have determined that the locations of the apparent cracks do NOT correlate with areas of higher flexural or shear stresses. Rather than resulting from overstress conditions, it is our belief that the observed cracking has likely resulted from one or more of the following long-term causes:

1. Soil consolidation and resulting settlement of the foundations below columns
2. Changes in moisture content in expansive soils below the foundations
3. Shrinkage of the concrete due to curing over time
4. Elastic, long-term deformation of the concrete, known as creep
5. Concentrations or build-up of internal stresses being released by the numerous minor and moderate seismic events that the structure has experienced over its lifetime.

Summary of Identified Structural Deficiencies

All deficiencies listed below are with regard to the 1985 Uniform Building Code, unless indicated otherwise.

Deficiencies Due to Calculated Capacities:

These deficiencies have resulted from calculation that determined the capacities of the respective elements are less than the demand or applied designs forces.

A. Upper Level Concrete Shear Walls:

1. On Grid Lines 1 and 5: These walls are overstressed in **in-plane bending**, as much as 144%, meaning that the demand is approximately 2.44 times the capacity.

2. On Grid Lines A and F: These walls are overstressed in in-plane bending, as much as 150%, meaning that the demand is approximately 1.5 times the capacity.

All Upper Level Shear Walls on All Grid Lines:

3. Based on the calculated compressive stress, the walls require Boundary Members at ends of shear walls. Boundary members are column-like elements with added vertical reinforcement and closely-spaced lateral ties that resist the high compressive forces induced by overturning demands in highly-loaded shear walls. At the ends of these walls not adjacent to a concrete column, no Boundary Members exist. This cannot be expressed as a magnitude of overstress, as the Code requirement is prescriptive. The magnitude of overturning forces in these shear walls requires Boundary Elements, but none are provided.

4. Based on the calculated shear stress, these walls require two curtains of reinforcing due to magnitude of in-plane shear stress. Only one curtain of reinforcing is provided in these 6" thick walls. This cannot be expressed as a magnitude of overstress, as the Code requirement is prescriptive. The magnitude of shear stress requires two curtains of reinforcing, but only one layer is provided.

5. Anchor bolt connections for transferring in-plane seismic shear forces at top of walls are overstressed as much as 26%, meaning that the demand is approximately 1.26 times the capacity.

B. Lower Level Concrete Shear Walls – No Deficiencies. Comments only:

Shear Walls Along Gridlines 5 & 5.5. There appear to be no overstress conditions in the overall shear walls throughout the building, or in the individual shear wall elements on grid lines 5 and 5.5. The added reinforcing within the 6-inch-thick shotcrete added to the remaining wall segments on line 5 in 1986 allows these elements to satisfy the applicable Code requirements. It should be noted that the added columns, as parts of the arches constructed along grid line 5.5 actually resist relatively low forces, and are adequate for the forces acting on them. **No deficiencies identified.**

C. Roof Diaphragm Shears:

Currently, the calculated roof diaphragm shears exceed the shear capacity of the plywood sheathing significantly. The 1/2-inch thick plywood, with nailing as specified on the original drawings, has an allowable shear capacity of 325 plf. The calculated diaphragm shear is as high as 898 plf in the north-south direction, and 690 plf in the east-west direction, resulting in the demand being 2.76 and 2.12 times its capacity, respectively. This represents overstresses of 176% and 112%, respectively.

It should be noted that the drawings do not indicate that Structural I plywood was used, however, reference to Structural I plywood was made in some of the 1986 calculations performed by Cygna Engineers. If Structural I plywood were used, the allowable shear would be 360 plf, however, the overstress conditions would still be significant.

The plywood sheathing lies directly over 3x decking at the sloped areas of roof, which acts as "blocking" at the adjacent panel edges. Generally, this type of decking can also be assumed to resist 100 plf to 300 plf in diaphragm shear, depending on its orientation and nailing. However, even if a 300 plf value is added to the allowable plywood shear value, the roof diaphragm shears still exceed the combined capacity significantly near the diaphragm perimeter, and extending significantly inward toward the center of the building.

D. Diaphragm Chord Connections at Roof:

The connections between the steel beams near the roof's perimeter are overstressed approximately 35% in resisting the highest chord forces, which occur at the middle of the diaphragm. The calculated connection capacities account for the steel shear plates that were added as part of the 1986 alterations. The same connections are adequate to resist the critical collector drag forces, which occur nearest the ends of the concrete shear walls at the Upper Level.

Deficiencies Due to Prescriptive Code Requirements:

The two aspects of the building's design below are part of the original 1964 design and were likely in compliance with the Building Code at that time, but they do not comply with newer requirements in the 1985 UBC. In general, these issues do not necessarily relate to stresses or force-levels, but address prescriptive Code requirements, and thus, a graduated level of compliance cannot be indicated.

E. Concrete Column Ties:

The #2 (1/4"ϕ.) ties around the concrete column vertical bars are inadequate in size and spacing, especially for the upper and lower 20% of the column height, where the actual spacing provided is generally two times, or 100% greater than allowed by the 1985 UBC. Added, external confinement can be installed around existing columns such as these, but no such reinforcing exists.

F. Concrete Column Ties at Top:

The 1985 Code, in this seismic zone, requires that several added, closely-spaced ties be provided at the tops of columns surrounding embedded anchor bolts, but no such ties were provided in the original columns. Added, external confinement can be installed around existing columns such as these, but no such reinforcing exists.

Potential Means to Mitigate Noted Deficiencies

Referencing pages 10 through 11 above, possible means for mitigation of the identified deficiencies are as follows:

Concrete Shear Walls:

A.1 and A.2 – Shear Wall In-Plane Bending:

The deficient bending capacities would typically be addressed by adding steel reinforcing at the extreme ends of the walls, which could resist more net tension induced by in-plane overturning or flexure in the shear walls. This could entail structural steel shapes fastened to the subject walls and extended down to the lower walls, or could include reinforcing added inside new concrete "column" elements, such as with shotcrete. Other methods could include thickening the walls over their lengths, partially or entirely, in order to reduce the tension demand at the wall ends. This could also be provided using shotcrete.

A.3 – Shear Wall Boundary Members:

New Boundary Members could include newly-applied concrete at the ends of the walls, and/or new steel members on the outside of the walls at the ends to provide the necessary stability under compressive loads induced by overturning.

A.4 – Second Layer of Wall reinforcing:

Providing a second curtain of wall reinforcing would require thickening the concrete walls, likely with applied shotcrete, and new reinforcing bars within.

A.5 – Anchor Bolt Connections at Top of Shear Walls:

New anchor bolts could be installed at the top of the walls directly through the existing beam flanges, or through added steel plates or angles welded to the steel beams.

B: Not a deficiency.

C – Roof Diaphragm Shears:

If the heavy Spanish tile roofing were to be replaced with composite asphalt roof tiles over its entirety, on the parapets and sloped roof areas, the correlating seismic force at the Upper Level, and overall, would be reduced significantly. The dead load occurring at the roof alone would be reduced by approximately 33%. This would also significantly reduce the total seismic force acting on the structure at the Lower Level as well. Although the roof diaphragm would still be overstressed in resisting Code-required seismic loading in smaller areas, the effects of using lighter roofing would be significant, and would likely greatly improve the building's performance and level of protection during a moderate or major earthquake.

As the capacity of the plywood roof diaphragm is dependent on the nailing of the plywood along the adjoining panel edges, the diaphragm capacity could be increased significantly with the addition of new nailing between the existing nails. Of course, access to the sheathing would be required. Thus, this method of strengthening the roof diaphragm could only be provided in conjunction with the removal of the existing roofing. As mentioned above, replacing the heavy, existing Spanish tile roofing with lighter roof tiles could yield

significant benefits in reducing the building mass and the resulting seismic forces on the structure. The combined effect of adding nailing to the existing plywood sheathing during the course of replacing the heavy tile roofing with lighter roof is obvious. Using reasonable assumptions as to weight of replacement roofing and maximum amount of added nailing in certain locations, the roof diaphragm shear capacities could be brought to within approximately 90% of the demand, reducing the overstress levels to approximately 11%.

D. - Diaphragm Chord Connections at Roof:

The subject connections could be strengthened with the addition of welding around the plates to the beam webs. This strengthening would be required in only limited places, near the middle of the building along each perimeter beam line, as the chord forces are the highest in the middle of the roof diaphragm, along the chords' lengths.

E. – Concrete Column Ties:

The columns' confinement could be increased through the use of external, surrounding "jackets." These could be of steel, concrete (shotcrete) or carbon fiber. The most cost-efficient and practicable method would likely be using carbon fiber layers applied with resin around each column for its entire height. This would have the least spatial effect, and would not increase the mass of the columns.

F. - Concrete Column Ties at Top:

As this aspect is similar to the deficiency noted immediately above, but more specific to location, means provided above to mitigate this issue would likely be the same. In fact, in the process of addressing the item above, this issue would be addressed and resolved as well.

End of Deficiency Mitigation

Possible Non-Compliance with Current 2010 Building Code

Editions of the Uniform Building Code since 1985, and more recently, the California Building Codes, have addressed **redundancy** and **ductility** in modern buildings' seismic-force-resisting systems. The design force equations in the newer Codes have undergone substantial revisions to more accurately reflect the various types of structural systems used, as well as the probability of major seismic events based on the fault types. However, the most significant revisions to recent seismic design requirements have been focused on ensuring better system performance in terms of the stability and ductility of members, especially when subjected to forces that are beyond the members' elastic range, or when elements undergo partial or complete failure.

At least six Building Code editions have been issued since the 1985 Uniform Building Code was published. Evaluating compliance of the Cupertino City Hall with respect to the most recent of these codes is beyond the scope of this analysis. However, it is expected that certain structural aspects of this building would be found not to comply with particular requirements in the current Code. And just as several aspects of the building have been found not to comply with the 1985 UBC, it is expected that more aspects would be found not to comply with the newer requirements in the most recent 2010 California Building Code.

Summary of Applied Seismic Design Forces

Comparative Seismic Force Factors for 1964 UBC, 1985 UBC and 2010 CBC:

W = Building Weight

According to 1964 Uniform Building Code:

Upper Level: $F_{EQ} = 0.133 \times W =$ **0.13 x W at Upper Level**
Lower Level considered a subterranean basement and not included in seismic design.
Essential Facilities not considered in 1964 UBC

According to 1985 UBC, as Essential Facility

Upper Level: $F_{EQ} = 0.14 \times 1.5 \times W =$ **0.21 x W at Upper Level**
Lower Level: $F_{EQ} = 0.19 \times 1.5 \times W =$ **0.28 x W at Lower Level**

According to 2010 CBC & ASCE7-05, as Essential Facility:

Upper Level: $F_{EQ} = 0.21 \times 1.3 \times W =$ **0.27 x W at Upper Level**
Lower Level: $F_{EQ} = 0.27 \times 1.0 \times W =$ **0.27 x W at Upper Level**

The 1.3 factor at Upper Level is the required Redundancy Factor.
Comparing Base Shear force magnitudes for 2010 CBC over 1985 UBC:
For Upper Level, there is a **30% increase** in applied seismic force over 1985 UBC levels
For Lower Level, there is no increase in applied seismic force

Comparing Base Shear force magnitudes for 2010 CBC over 1964 UBC:
For Upper Level, there is a **108% increase** in applied seismic force from 1964 UBC levels.
For Lower Level, the structure was considered a basement therefore no comparison is made.

End of Report

APPENDICES

The following Appendices to the Report are available at request:

Structural Calculations for Analysis by AKH Structural Engineers, Inc., 2011

Original Structural Calculations by Kirk McFarland Engineers, 1965

APPENDICES

Structural Calculations by AKH

Structural Calculations by Kirk McFarland Engineers

AKH Structural Engineers, Inc.
1505 Meridian Avenue, Suite B
San Jose, CA 95125

Phone (408) 978-1970
Fax (408) 267-7919
www.akhse.com

Structural Calculations
Cupertino City Civic Center

Project Number:	M05-036	Date:	October 2011
Project Engineer:	Ward	Code:	**1985 UBC**
Seismic Zone:	4	Wind Zone:	70 mph
Checked By:	Hyde	Date:	

Original Signature Required
To Be Valid Seal

TABLE OF CONTENTS

Design References:

1. Uniform Building Code, 1985 Edition
2. Building Code Requirements for Structural Concrete (ACI 318-83)
3. Manual of Steel Construction, 8th Edition

STRUCTURAL LOADS

ROOF TYPE #1: Existing Tile Roof Slope, in/ft.: 4

Material	Decking	Purlins	Beams	Seismic	
Spanish Tile Roofing	21.0	21.0	21.0	21.0	
1/2" Plywood	1.5	1.5	1.5	1.5	
3x Decking	8.0	8.0	8.0	8.0	
Insulation	0.5	0.5	0.5	0.5	
Fire Sprinklers		4.0	3.0	2.0	
Ceiling - Suspended wood w/ tile		5.0	5.0	5.0	
Purlins - 6x16 @ 6'-0" o.c.		3.5	3.5	3.5	
Beams - 16B31 @ 24'-0" o.c.			1.5	1.5	
Beams - 7"x19-1/2" Glue-lam beam			1.0	1.0	
Miscellaneous	5.1	3.9	4.3	2.5	
Sub-Total:	36.1	47.4	49.3	46.5	
Slope Factor	2.0	2.6	2.7	2.5	
TOTAL DEAD LOAD:	38.0	50.0	52.0	49.0	psf

ROOF TYPE #2: Existing Built-up Slope, in/ft.: 0.25

Material	Decking	Joists	Beams	Seismic	
Roofing - Built-up, gravel	6.5	6.5	6.5	6.5	
1/2" Plywood	1.5	1.5	1.5	1.5	
Insulation	0.5	0.5	0.5	0.5	
Fire Sprinklers		4.0	3.0	2.0	
Joists - 2x14 @ 16" o.c.		4.0	4.0	4.0	
HVAC Equipment		5.0	5.0	5.0	
Ceiling - Suspended wood w/ tile		5.0	5.0	5.0	
Beams - 21WF55 @ 24'-0" o.c.			2.5	2.5	
Miscellaneous	4.5	4.5	4.0	2.0	
Sub-Total:	13.0	31.0	32.0	29.0	
Slope Factor	0.0	0.0	0.0	0.0	
TOTAL DEAD LOAD:	13.0	31.0	32.0	29.0	psf

ROOF TYPE #3: Existing Covered Walk Slope, in/ft.: 2

Material	Decking	Purlins	Beams	Seismic	
Spanish Tile Roofing	21.0	21.0	21.0	21.0	
1/2" Plywood	1.5	1.5	1.5	1.5	
2x Decking	4.5	4.5	4.5	4.5	
Insulation	0.5	0.5	0.5	0.5	
Fire Sprinklers		4.0	3.0	2.0	
Purlins - 4x14 @ 6'-0" o.c.		2.5	2.5	2.5	
Beams - 16B31			1.5	1.5	
Beams - 10"x18" conc.			16.0	16.0	
Miscellaneous	4.8	3.9	3.6	1.7	
Sub-Total:	32.3	37.9	54.1	51.2	
Slope Factor	1.7	2.1	2.9	2.8	
TOTAL DEAD LOAD:	34.0	40.0	57.0	54.0	psf

FLOOR TYPE #1:

Material	Slab	Joists	Beams	Seismic	
Flooring	4.0	4.0	4.0	4.0	
Concrete Slab, 3"	37.5	37.5	37.5	37.5	
Fire Sprinklers		4.0	3.0	2.0	
Ceiling		2.5	2.5	2.5	
Joists - 6"x12" @ 3'-0" o.c.		29.0	29.0	29.0	
Ribs - 6"x12" @ 24'-0" o.c.		3.5	3.5	3.5	
Beams - 16"x33" @ 24'-0" o.c.			26.0	26.0	
Miscellaneous	4.5	4.5	3.5	1.5	
Sub-Total:	46.0	85.0	109.0	106.0	
Partitions		20.0	20.0	20.0	
TOTAL DEAD LOAD:	46.0	105.0	129.0	126.0	psf

WALL TYPE #1: Stud Wall w/ Gyp Board Each Side

Material	Weight	
Studs @ 16" o.c.	1.5	
Gypsum Board	5.0	
Miscellaneous	1.5	
TOTAL WEIGHT:	8.0	psf

WALL TYPE #2: Stud Wall w/ Spanish Tile One Side & Plywood One Side -- Mansard

Material	Weight	
Studs @ 16" o.c.	2.5	
Spanish Tile Roofing	21.0	
1/2" Plywood - 2 sides	3.0	
Built-up Roofing, cap	2.0	
Miscellaneous	1.5	
TOTAL WEIGHT:	30.0	psf

WALL TYPE #3:	6" Concrete	75	psf
WALL TYPE #4:	10" Concrete	125	psf
WALL TYPE #5:	12" Concrete	150	psf
WALL TYPE #6:	18" Concrete	225	psf

LIVE LOADS: Floor

Office Areas	50	psf
Corridors & Lobbies	100	psf
Assembly, Open	100	psf

LIVE LOADS: Roof	0 - 200	201 - 600	> 600	sq. ft.
Slope < 4:12	20	16	12	psf
Slope 4:12 to < 12:12	16	14	12	psf
Slope > 12:12	12	12	12	psf

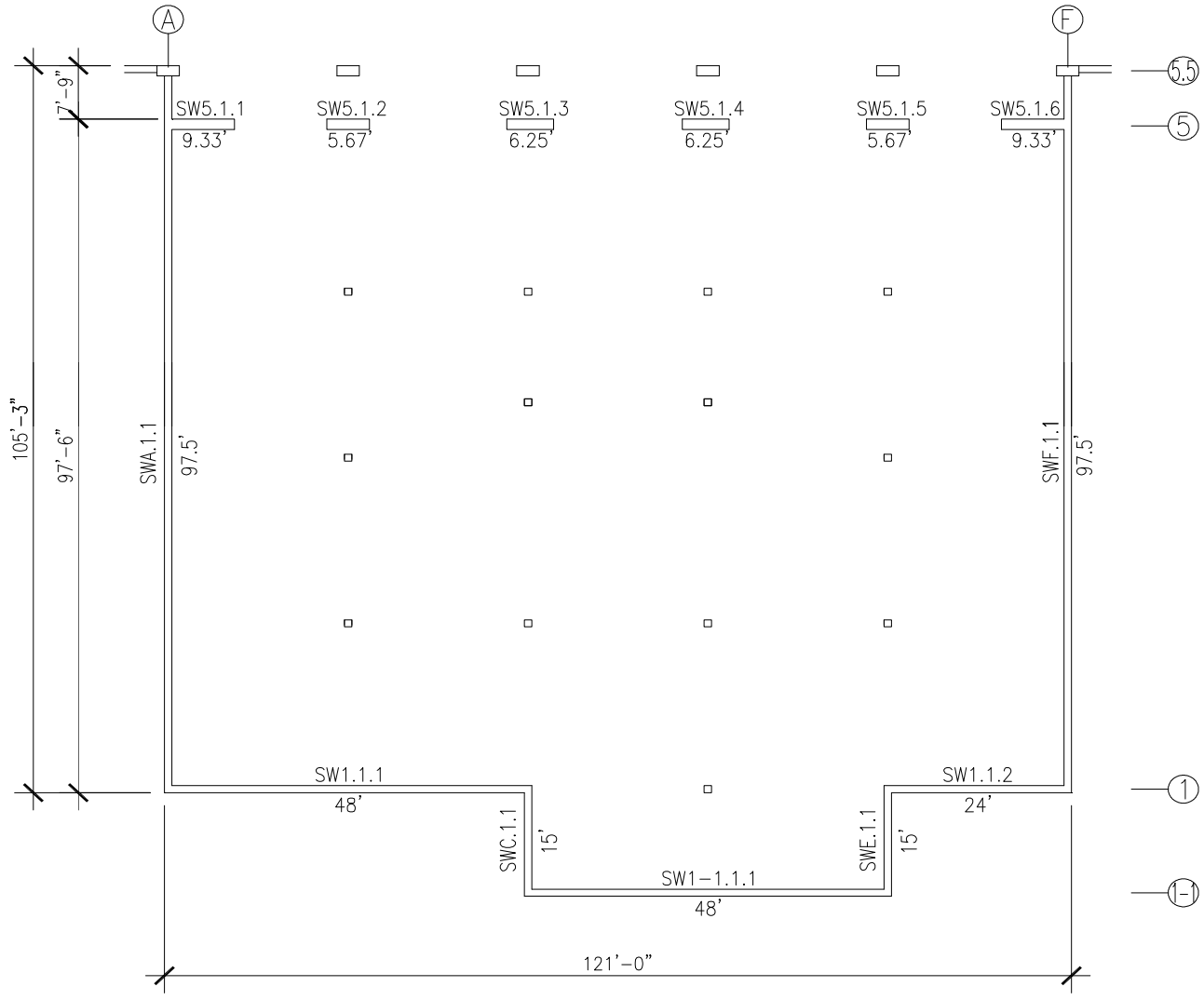
LATERAL ANALYSIS & DESIGN

L1

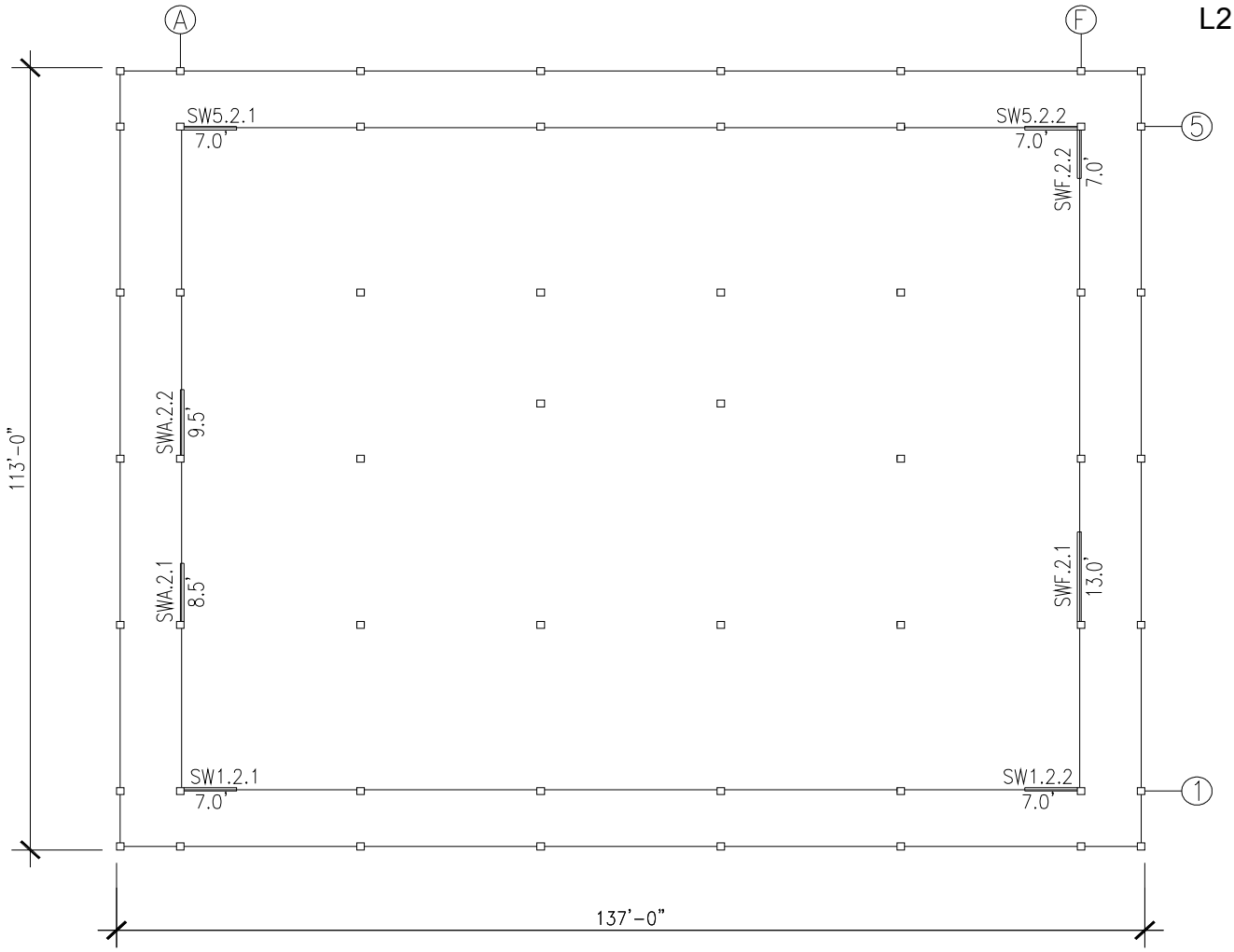
The building was originally designed in 1964 as a non-essential facitlity. The upper level of the building lateral system consists of existing wood roof diaphragm and concrete shear walls. The lower level consists of a concrete joist floor system over concrete walls & columns. The lower portion was originally a full basement. In 1986, modifications & upgrades were made to the building. The North side of the building was excavated to the depth of the basement and the North basement wall opened up to a new patio. The building currently houses the City's Emergency Operations Center and is therefore an essential facility.

Note: The following analysis is to verify that the existing building, including the 1987 modifications, is adequate as an essential facility under the 1985 Uniform Building Code.

Earthquake Data:
Seismic Zone 4 (UBC Figure No. 1)



LOWER LEVEL PLAN



UPPER LEVEL PLAN

Building Mass:

Roof	Area	Unit Load	Mass
Flat Roof	3456	29	100224
Tile Roof	8281	49	405769
Walkway	5808	54	313632
Mansard	1440	30	43200
Ext. Walls	396	75	29700
Partitions	11305	5	56525
Columns	297	150	44550
		Total w_r =	993600 lbs

Roof height h_r = 18 ft.

==> 993.6 kips

Floor	Area	Unit Load	Mass
Floor	14464	106	1533228
Ext. Walls	345	225	77625
Ret. Walls	345	150	51750
Int. Walls	1157	75	86738
Partitions	11305	10	113050
Columns	234	150	35100
		Total w_f =	1897491 lbs

Floor height h_f = 12 ft.

==> 1897.5 kips

SEISMIC FACTORS

Seismic Base Shear:
V = ZIKCSW

Z = 1 (Zone 4 - Figure No. 1)
I = 1.5 (Essential Facility - Table No. 23-K)

At Upper Level:
K = 1 (Framing System - Table No. 23-I)

h_n = 18 ft. D = 137 ft.
$T = 0.05 * h_n / (D)^{1/2}$ = 0.077 sec.
$C = 1 / [15 * (T)^{1/2}]$ = 0.240 > 0.12 Therefore, use C = 0.12

S = 1.5 (Soil Type S_3 for unknown soil)
CS = 0.18 > 0.14 Therefore, use CS = 0.14

V =	0.210	W

At Lower Level:
K = 1.33 (Shear Wall System - Table No. 23-I)

h_n = 12 ft. D = 121 ft.
$T = 0.05 * h_n / (D)^{1/2}$ = 0.055 sec.
$C = 1 / [15 * (T)^{1/2}]$ = 0.285 > 0.12 Therefore, use C = 0.12

S = 1.5 (Soil Type S_3 for unknown soil)
CS = 0.18 > 0.14 Therefore, use CS = 0.14

V =	0.279	W

Elements of Structures: Wall Seismic Factor:
$F_p = ZIC_pW_p$

C_p = 0.3 (Bearing Walls - Table No. 23-J)

F_p =	0.450	W_p

Diaphragm & Collector Seismic Factor:
$F_{px} = (\Sigma F_n/\Sigma w_n)w_{px}$

Max. $F_{px} = 0.30Zlw_{px}$ = 0.450 w_{px}
Min. $F_{px} = 0.14Zlw_{px}$ = 0.210 w_{px}

For one-story building, F_{px} = V for Upper Level
F_{px} = 0.210 w_{px} < Max. F_{px} = 0.450 w_{px}
F_{px} = 0.210 w_{px} = Min. F_{px} = 0.210 w_{px}

F_{px} =	0.210	w_{px}

Upper Level Seismic Loads:

Wood frame roof with plywood diaphragm. Concrete shear walls.

Height H = 12 ft. Note: All wall thicknesses t = 6 in.

Note: Wall rigidities are of fixed-fixed walls: $R = t / [0.1(H/L)^3 + 0.3(H/L)]$

Wall	Length L	Rigidity	Relative R
SW1.2.1	7	5.893	0.5
SW1.2.2	7	5.893	0.5
	ΣR =	12	
SW5.2.1	7	5.893	0.5
SW5.2.2	7	5.893	0.5
	ΣR =	12	

Wall	Length L	Rigidity	Relative R
SWA.2.1	8.5	8.512	0.452
SWA.2.2	9.5	10.336	0.548
	ΣR =	19	
SWF.2.1	13	16.874	0.741
SWF.2.2	7	5.893	0.259
	ΣR =	23	

Total Base Shear V = 208.66 kips

Load to each shear wall line F_2 = 104.33 kips

Wall	Shear
SW1.2.1	52.16
SW1.2.2	52.16
"1" Total:	104.33

Wall	Shear
SW5.2.1	52.16
SW5.2.2	52.16
"5" Total:	104.33

Wall	Shear
SWA.2.1	47.12
SWA.2.2	57.21
"A" Total:	104.33

Wall	Shear
SWF.2.1	77.32
SWF.2.2	27.01
"F" Total:	104.33

Out of Plane Loads to Upper Level Walls

12 ft. high x 6" thick walls spanning from podium slab to roof

Fp = 0.45 (150pcf x 0.5ft) = 33.75 psf

Assuming pin-pin wall connections (top & bottom)

v_{max} = Fp x H/2 = 202.5 lbs/ft.

M_{max} = Fp x H^2/8 = 607.5 ft-lbs

unsupported length, l_u = 144 inches

radius of gyration, r = 0.3 x 6" = 1.8 inches

$kl/r = 1.0*l_u/r$ = 80 > 22, slenderness effects must be considered

Load combinations to consider:

1.05D + 1.28L + 1.4E 0.9D+ 1.1E

See Enercalc analysis of slender wall

6" wall w/ #4 @ 12"o.c. ea. way is adequate to span vertically

Anchorage for out-of-plane loads:

existing 7/8"dia. x 12" a.b. w/ 3" edge distance:

Allowable shear, V = 4.15k - [(5.25-3)(4.15-2.075)/(5.25-2.63)] = 2.37 kips/bolt

(per 1985 UBC table 26-G for 3ksi concrete)

Reaction at top of walls, R = 0.204 klf (ASD)

for 7' long wall, number of A.B. required = R x 7/V = 1 bolt

for 13'-10" long wall, number of A.B. required = R x 13.83/V = 2 bolts

See Enercalc Anaysis of Slender Wall

Existing Concrete Walls at the Upper Level are adequate for Out-Of-Plane Bending, Deflection and for Anchorage.

DEFICIENCIES: None

Lower Level Seismic Loads:

Concrete floor slab. Concrete shear walls.

Height H = 12 ft.

Note: All wall thicknesses t = 12 in., except along Line 5, where t = 18 in.

Note: Wall rigidities are of fixed-fixed walls: $R = t / [0.1(H/L)^3 + 0.3(H/L)]$

Wall	Length L	Rigidity	Relative R
SW1.1.1	48	156.735	0.680
SW1.1.2	24	73.846	0.320
	ΣR =	231	

Wall	Length L	Rigidity	Relative R
SWA.1.1	97.5	323.367	1.000
SWF.1.1	97.5	323.367	1.000

Wall Line 5: Wall has new window & door openings due to 1986 modifications.

Header height H_1 = 3.875 ft.

Pier height H_2 = 5.75 ft.

Sill height H_3 = 2.375 ft.

Note: Wall deflections are of fixed-fixed walls: $\Delta = [0.1(H/L)^3 + 0.3(H/L)] / t$

Wall	Length L	Rigidity	Relative R
SW5.1.1	9.33	86.416	0.238
SW5.1.2	5.67	44.061	0.121
SW5.1.3	6.25	50.866	0.140
SW5.1.4	6.25	50.866	0.140
SW5.1.5	5.67	44.061	0.121
SW5.1.6	9.33	86.416	0.238
	ΣR =	362.686	
$\Delta_{Piers} = 1/\Sigma R$ =		0.0028	

Wall	Length L	Deflection
Header	121	0.0005

Wall	Length L	Rigidity	Relative R
Sill	51.92	1310.749	0.500
ΣR = 2 * R =		2621.498	
$\Delta_{Sills} = 1/\Sigma R$ =		0.0004	

$\Delta_{Total} = \Delta_{Header} + \Delta_{Piers} + \Delta_{Sills}$ = 0.0037 in.

$R_{Total} = 1/\Delta_{Total}$ = 272.287

Total Base Shear V = 530.0 kips from lower level mass only
Depth D = 97.5 ft.
Length L = 121 ft.
Transverse (N-S) load W_{N-S} = V / L = 4.38 kip/ft.
Longitudinal (E-W) load W_{E-W} = V / D = 5.44 kip/ft.
Load to each wall line from upper level F_1 = 104.33 kips
Longitudinal seismic distribution, see Enercalc Torsional Analysis of Rigid Diaphragm Program, following pages.
Load to each shear wall line:

Line	Shear	Total Shear	
1	406.15	510.48	kips
5	153.58	257.91	kips
A	282.88	387.21	kips
F	282.88	387.21	kips

Roof Diaphragm

Diaphragm length L = 97 ft. Entire length L_1 = 121 ft.
Diaphragm depth D = 121 ft. Entire depth D_1 = 145 ft.
Note: The seismic coefficients for base shear & the diaphragm force, at the upper level, are equal to each other.
Diaphragm force F_{px} = V = 208.66 kips

Transverse Direction:

Diaphragm shear v = F_{px} * (D / D_1) * 1000 / L / 2 = 898 plf
Existing diaphragm is 1/2" CD-X plywood over 3x T&G decking w/ 10d @ 6" o.c. at all edges, blocked.
Allowable shear for plywood v_{ap} = 325 plf (UBC Table No. 25-J-1)
Allowable shear for decking v_{ad} = 100 plf (UCBC values.)
Allowable shear v_a = v_{ap} + v_{ad} = 425 plf < v = 898 plf **NG!**
Overstress = 111.2% > 5.0% **NG!**

Longitudinal Direction:

Diaphragm shear v = F_{px} * (L / L_1) * 1000 / D / 2 = 691 plf
Existing diaphragm is 1/2" CD-X plywood over 3x T&G decking w/ 10d @ 6" o.c. at all edges, blocked.
Allowable shear v_a = 425 plf < v = 691 plf **NG!**
Overstress = 62.6% > 5.0% **NG!**

Existing Roof Diaphragm Is Highly <u>Overstressed</u>. Therefore, Diaphragm Is <u>NOT</u> Adequate.
<u>DEFICIENCIES</u>: Insufficient Amount Of Diaphragm Nailing -- Diaphragm Nail Spacing Is Too Large.

Chord & Collector Stresses

Existing chord/collector is 16B31 (W16x31) A36 steel w/ splice plate 1/4"x9"x12" on both sides & (3) 5/8"ϕ M.B.'s (A307 steel) at each end -- bolts are in double shear. (Note: Originally, there was only a single splice plate, but in the 1986 upgrades, an additional splice plate was added to the opposite side.)

Bolts:

d_b = 0.625 in. $d'_b = d_b + .0625"$ = 0.6875 in.

Beam:

t_w = 0.275 in. A = 9.12 in^2

F_y = 36 ksi F_u = 58 ksi

Plates:

t_p = 0.25 in. d_p = 9 in.

F_y = 36 ksi F_u = 58 ksi

Allowable shear load on bolts = 6.1 * 3 * 1.33 = 24.3 kips

Allowable tension load on plates (gross) = 0.60 * F_y * (2 * t_p * d_p) * 1.33 = 129.3 kips

Allowable tension load on plates (net) = 0.50 * F_u * {2 * t_p * [d_p - (3 * d'_b)]} * 1.33 = 133.8 kips

Allowable tension load on beam (gross) = 0.60 * F_y * A * 1.33 = 262.0 kips

Allowable tension load on beam (net) = 0.50 * F_u * [A - (3 * t_w * d'_b)] * 1.33 = 329.9 kips

Min. allowable load = 24.3 kips <== Bolts capacity governs.

Existing 16B31 perimeter beams are anchored to top of concrete shear walls w/ 7/8"ϕ A.B.'s (A307 steel).
Allowable shear load on anchor bolts = 4.15 * 1.33 = 5.52 kips (UBC Table No. 26-G)

Chord Stress:

Line A (Line F Same)

Chord stress T = F_{px} * D^2 / 8 / L / D_1 = 27.2 kips

Allowable load = 24.3 kips < T = 27.2 kips **NG!**

Overstress = 11.5% > 5.0% **NG!**

Line 1 (Line 5 Same)

Chord stress T = F_{px} * L^2 / 8 / D / L_1 = 16.8 kips

Allowable load = 24.3 kips > T = 16.8 kips **OK**

Collector Stresses:

Line A

Force to Line A F_A = 104.33 kips

See Program PL-02 - Collector Stress Analysis, following pages.

Max. collector load to SWA.2.1 = 26.9 kips

Max. collector load to SWA.2.2 = 41.4 kips

Max. collector load F = 41.4 kips

Check splice connection:

Allowable load = 24.3 kips < F = 41.4 kips **NG!**

Overstress = 70.1% > 5.0% **NG!**

Check wall anchorage:

Number of anchor bolts N at SWA.2.1 = 6 anchor bolts

Allowable load = 5.52 * N = 33.1 kips > F = 26.9 kips **OK**

Number of anchor bolts N at SWA.2.2 = 8 anchor bolts

Allowable load = 5.52 * N = 44.2 kips > F = 41.4 kips **OK**

Line F

Force to Line F F_F = 104.33 kips

See Program PL-02 - Collector Stress Analysis, following pages.

Max. collector load to SWF.2.1 = 26.9 kips

Max. collector load to SWF.2.2 = 27.9 kips

Max. collector load F = 27.9 kips

Check splice connection:

Allowable load = 24.3 kips < F = 27.9 kips **NG!**

Overstress = 14.6% > 5.0% **NG!**

Check wall anchorage:

Number of anchor bolts N at SWF.2.1 = 10 anchor bolts

Allowable load = 5.52 * N = 55.2 kips > F = 26.9 kips **OK**

Number of anchor bolts N at SWF.2.2 = 4 anchor bolts

Allowable load = 5.52 * N = 22.1 kips < F = 27.9 kips **NG!**

Overstress = 26.4% > 5.0% **NG!**

Line 1 (Line 5 Same)

Force to Line 1 F_1 = 104.33 kips

See Program PL-02 - Collector Stress Analysis, following pages.

Max. collector load F = 45.3 kips

Check splice connection:

Allowable load = 24.3 kips < F = 45.3 kips **NG!**

Overstress = 86.0% > 5.0% **NG!**

Check wall anchorage:

Number of anchor bolts N = 7 anchor bolts

Allowable load = 5.52 * N = 38.6 kips < F = 45.3 kips **NG!**

Overstress = 17.2% > 5.0% **NG!**

Existing 16B31 (W16x31) Perimeter Beam Splice Connection, for Collector and Chord forces, Is Highly Overstressed. Therefore, Beam Splice Connection Is NOT Adequate.

DEFICIENCIES: Insufficient Number Of Bolts At Each Beam End At Splice Connection.

Existing Anchorage of 16B31 Perimeter Beams To Top Of Concrete Shear Walls, for In-plane shear, Is Overstressed. Therefore, Beam Anchorage Is NOT Adequate.

DEFICIENCIES: Insufficient Number Of Anchor Bolts From Perimeter Beams To Top Of Most Concrete Shear Walls for In-Plane Shear. Walls With Insufficient Anchor Bolts Include All Walls Along Lines 1 & 5 and Wall Along Line F, Near Line 5 Corner.

Upper Level:

Wall SW1.2.1 (Walls SW1.2.2, SW5.2.1, & SW5.2.2 Similar)

Wall height H = 12 ft.
Wall length L = 7 ft.
Wall thickness t = 6 in.

Roof tributary width 1 L_{r1} = 12 ft. Roof tributary width 2 L_{r2} = 4 ft.
Roof load $W_r = [(49 * L_{r1}) + (54 * L_{r2})] * L / 1000$ = 5.63 kips at 3.5 ft. from left end of wall.

Wall weight $W_w = 150 * t * H * L / 12 / 1000$ = 6.3 kips at 3.5 ft. from left end of wall.
Wall seismic weight $F_w = 0.210 * W_w$ = 1.3 kips at 6 ft. from base of wall.

Applied lateral load F = 52.16 kips at 12 ft. from base of wall.
See Program PC-02 - Concrete Shear Wall, following pages.
Max. M_u = 887.2 ft.-kips > ϕM_n = 361.6 ft.-kips **NG!**
Overstress = 145.4% > 5.0% **NG!**

Check existing dowelling from wall base to foundation wall:

V_u = 74.8 kips
Existing #5 dowels @ 6" o.c.: ϕ = 0.85
d = 0.8 * L * 12 = 67.2 in. s = 6 in.
A_{vf} = 0.31 in^2 f_y = 40 ksi
f'_c = 3.5 ksi $b_w = t$ = 6 in.
$\phi V_s = \phi * (A_{vf} * f_y * d / s)$ = 118.1 kips
$\phi V_c = \phi * [2 * (f'_c)^{1/2} * b * d]$ = 40.6 kips
$\phi V_n = \phi V_s + \phi V_c$ = 158.6 kips < $\phi V_c = \phi * [10 * (f'_c)^{1/2} * b * d]$ = 202.8 kips
ϕV_n = 158.6 kips > V_u = 74.8 kips **OK**

Note: Starting with the 1967 UBC, (vertical) boundary members & boundary ties were required for shear walls that were part of dual systems with frames (K = 0.80). However, in the 1985 UBC, the boundary member requirements were changed to include all shear walls in general.

Existing Concrete Shear Walls Along Lines 1 & 5 Are Highly Overstressed in In-Plane bending. Therefore, Walls Are NOT Adequate.
DEFICIENCIES: Boundary Members Required. Double Layer of Reinforcing Required.

Wall SWA.2.1

Wall height H = 12 ft.
Wall length L = 8.5 ft.
Wall thickness t = 6 in.

Roof tributary width 1 L_{r1} = 12 ft. Roof tributary width 2 L_{r2} = 4 ft.
Roof load $W_r = [(49 * L_{r1}) + (54 * L_{r2})] * L / 1000$ = 6.83 kips at 4.25 ft. from left end of wall.

Wall weight $W_w = 150 * t * H * L / 12 / 1000$ = 7.7 kips at 4.25 ft. from left end of wall.
Wall seismic weight $F_w = 0.210 * W_w$ = 1.6 kips at 6 ft. from base of wall.

Applied lateral load F = 47.12 kips at 12 ft. from base of wall.
See Program PC-02 - Concrete Shear Wall, following pages.
Max. M_u = 805.1 ft.-kips > ϕM_n = 285.4 ft.-kips **NG!**
Overstress = 182.1% > 5.0% **NG!**

Check existing dowelling from wall base to foundation wall:
V_u = 74.8 kips
Existing #5 dowels @ 6" o.c.: ϕ = 0.85
d = 0.8 * L * 12 = 81.6 in. s = 6 in.
A_{vf} = 0.31 in^2 f_y = 40 ksi
f'_c = 3.5 ksi $b_w = t$ = 6 in.
$\phi V_s = \phi * (A_{vf} * f_y * d / s)$ = 143.3 kips
$\phi V_c = \phi * [2 * (f'_c)^{1/2} * b * d]$ = 49.2 kips
$\phi V_n = \phi V_s + \phi V_c$ = 192.6 kips < $\phi V_c = \phi * [10 * (f'_c)^{1/2} * b * d]$ = 246.2 kips
ϕV_n = 192.6 kips > V_u = 74.8 kips **OK**

Existing Concrete Shear Wall Along Line A Is Highly Overstressed in In-Plane bending. Therefore, Wall Is NOT Adequate.
DEFICIENCIES: Boundary Members Required.

Wall SWA.2.2

Wall height H = 12 ft.
Wall length L = 9.5 ft.
Wall thickness t = 6 in.

Roof tributary width 1 L_{r1} = 12 ft. Roof tributary width 2 L_{r2} = 4 ft.
Roof load $W_r = [(49 * L_{r1}) + (54 * L_{r2})] * L / 1000$ = 7.64 kips at 4.75 ft. from left end of wall.

Wall weight $W_w = 150 * t * H * L / 12 / 1000$ = 8.6 kips at 4.75 ft. from left end of wall.
Wall seismic weight $F_w = 0.210 * W_w$ = 1.8 kips at 6 ft. from base of wall.

Applied lateral load F = 57.21 kips at 12 ft. from base of wall.
See Program PC-02 - Concrete Shear Wall, following pages.
Max. M_u = 976.2 ft.-kips > ϕM_n = 406.2 ft.-kips **NG!**
Overstress = 140.3% > 5.0% **NG!**

Existing Concrete Shear Wall Along Line A Is Highly Overstressed in In-Plane bending. Therefore, Wall Is NOT Adequate.
DEFICIENCIES: Boundary Members Required. Double Layer of Reinforcing Required.

Wall SWF.2.1

Wall height H = 12 ft.
Wall length L = 13 ft.
Wall thickness t = 6 in.

Roof tributary width 1 L_{r1} = 12 ft. Roof tributary width 2 L_{r2} = 4 ft.
Roof load $W_r = [(49 * L_{r1}) + (54 * L_{r2})] * L / 1000$ = 10.45 kips at 6.5 ft. from left end of wall.

Wall weight $W_w = 150 * t * H * L / 12 / 1000$ = 11.7 kips at 6.5 ft. from left end of wall.
Wall seismic weight $F_w = 0.210 * W_w$ = 2.5 kips at 6 ft. from base of wall.

Applied lateral load F = 77.32 kips at 12 ft. from base of wall.
See Program PC-02 - Concrete Shear Wall, following pages.
Max. M_u = 1320.0 ft.-kips > ϕM_n = 699.2 ft.-kips **NG!**
Overstress = 88.8% > 5.0% **NG!**

Existing Concrete Shear Wall Along Line F Is Highly Overstressed in In-Plane bending. Therefore, Wall Is NOT Adequate.
DEFICIENCIES: Double Layer of Reinforcing Required.

Wall SWF.2.2

Wall height H = 12 ft.
Wall length L = 7 ft.
Wall thickness t = 6 in.

Roof tributary width 1 L_{r1} = 12 ft. Roof tributary width 2 L_{r2} = 4 ft.
Roof load $W_r = [(49 * L_{r1}) + (54 * L_{r2})] * L / 1000$ = 5.63 kips at 3.5 ft. from left end of wall.

Wall weight $W_w = 150 * t * H * L / 12 / 1000$ = 6.3 kips at 3.5 ft. from left end of wall.
Wall seismic weight $F_w = 0.210 * W_w$ = 1.3 kips at 6 ft. from base of wall.

Applied lateral load F = 27.01 kips at 12 ft. from base of wall.
See Program PC-02 - Concrete Shear Wall, following pages.
Max. M_u = 464.7 ft.-kips > ϕM_n = 232.4 ft.-kips **NG!**
Overstress = 100.0% > 5.0% **NG!**

Existing Concrete Shear Wall Along Line F Is Highly Overstressed in In-Plane bending. Therefore, Wall Is NOT Adequate.
DEFICIENCIES: Boundary Members Required.

Lower Level:

Wall SW1.5.1 (Walls along gridline 5 similar)

Wall height H = 5.75 ft.
Wall length L = 5.67 ft.
Wall thickness t = 18 in.

Roof tributary width 1 L_{r1} = ft. Roof tributary width 2 L_{r2} = ft.
Roof load $W_r = [(49 * L_{r1}) + (54 * L_{r2})] * L / 1000$ = 0.00 kips at 2.835 ft. from left end of wall.

Wall weight $W_w = 150 * t * H * L / 12 / 1000$ = 7.3 kips at 2.835 ft. from left end of wall.
Wall seismic weight $F_w = 0.210 * W_w$ = 1.5 kips at 2.875 ft. from base of wall.

Applied lateral load F = 27.40 kips at 5.75 ft. from base of wall.
See Program PC-02 - Concrete Shear Wall, following pages.
Max. M_u = 236.7 ft.-kips > ϕM_n = 115.3 ft.-kips **NG!**
Overstress = 105.3% > 5.0% **NG!**

Existing Concrete Shear Walls Along Line 5 Are Adequate in In-Plane Shear, Bending and for Compressive Forces Due to Overturning.
DEFICIENCIES: None

Wall SW1.1.2 (Walls along gridline 1 similar)

Wall height H = 13 ft.
Wall length L = 25 ft.
Wall thickness t = 12 in.

Roof tributary width 1 L_{r1} = ft. Roof tributary width 2 L_{r2} = ft.
Roof load $W_r = [(49 * L_{r1}) + (54 * L_{r2})] * L / 1000$ = 0.00 kips at 12.5 ft. from left end of wall.

Wall weight $W_w = 150 * t * H * L / 12 / 1000$ = 48.8 kips at 12.5 ft. from left end of wall.
Wall seismic weight $F_w = 0.210 * W_w$ = 10.2 kips at 6.5 ft. from base of wall.

Applied lateral load F = 119.50 kips at 13 ft. from base of wall.
See Program PC-02 - Concrete Shear Wall, following pages.
Max. M_u = 2268.0 ft.-kips < ϕM_n = 2293.0 ft.-kips **OK**
Overstress = -1.1% < 5.0% **OK**

Existing Concrete Shear Walls Along Line 1 Are Adequate in In-Plane Shear, Bending and for Compressive Forces Due to Overturning.
DEFICIENCIES: Double layer or reinforcing required

Girder Check

Applied Loads:

Dead Load =	109	psf GIRDERS
Dead Load =	83	psf JOISTS
Live Load =	50	psf for office spaces
	100	psf for assembly, corridor spaces

Live Load reduction $L = L_o(0.25+15/\text{sqrt}(K_{LL}A_T))$

K_{LL} = 2 for interior beams

A_T = 768 sq. ft. for 32 ft. span girder > 400 sf, reducible

576 sq. ft. for 24 ft. span girder > 400 sf, reducible

no LL reduction for joists

LL_{office} = 41.5 psf at 24 ft. span (> 0.5LL, OK)

$LL_{assembly}$ = 60.0 psf at 32 ft. span (no reduction for public assembly)

$LL_{corridor}$ = 63.3 psf at 32 ft. span (> 0.5LL, OK)

P_{DL} Beam B4 = 2600 lbs (framing into girder on gridline E)

fy main rebar = 40 ksi
fy ties = 40 ksi

slab effective width used for girder flange:
overhanging flange shall be
smallest of: 8 x slab thickness = 24 in. <-- governs
clear span/2 = 136.02 in.
total width is 2 x 24in + 16in = 64 in.
total shall not exceed girder span/4 = 68.01 in.

slab effective width used for joist flange:
overhanging flange shall be
smallest of: 8 x slab thickness = 24 in. <-- governs
clear span/2 = 136.02 in.
total width is 2 x 24in + 12in = 60 in.
total shall not exceed joist span/4 = 68.01 in.

Floor Live Load reduction:

A_{trib} = 768 sf A_{trib} = 576 sf

can be reduced at a rate of 0.08 percent of area supported by the member

R = 49%

R_{max} = 40% <---governs, reduced $LL_{corridor}$ can be 40psf, LL_{office} can be 20psf;

R_{max} = 48% 50 psf used in all spaces in original analysis

See Enercalc Analyses for Joists & Girders

Original Design; Shear in Girders

Working Stress Design 1964 UBC:

V_{max} = 60700 lbs (from original shear diagrams)

v_{allow} = 90 psi for 3ksi concrete w/ stirrups

$V' = V_{max} - v_{allow}(bjd)$ = 19120 lbs shear carried by web reinforcement

$s_{req'd} = A_v f_v jd/V'$ = 13.3 in. Stirrups provided at 12" spacing

Current Design; Shear in Girders

Strength Design 2010 CBC:

Vc = 62474 lbs f'c = 3500 psi

fVc = 46855 lbs for (2) #3 tie legs, s = 0.22(40000)33"/Vs = 290400 /Vs

fVc/2 = 23428 lbs

Vu	Vs	<fVc/2?	spcg for A_{vmin1}	spcg for A_{vmin2}	spcg for A_v	spacing	
80000	44193	no	12.4	11	6	6	
70000	30860	no	12.4	11	9	9	Per Enercalc Analysis:
65000	24193	no	12.4	11	12	11	Vu max = 78730 lbs
60000	17526	no	12.4	11	16	11	ϕVn for ties @ 12" = 65000 lbs
50000	4193	no	12.4	11	69	11	max overstress = 1.21
40000	0	no	12.4	11	not req'd	11	
30000	0	no	12.4	11	not req'd	11	

Deflection limitations

1964 UBC & 2010 CBC

L/360 for LL

L/240 for DL + LL

50 psf LL

32' span D limit = 1.07 in. (LL only)

1.60 in. (DL + LL) Dmax = 0.154 in.

24' span D limit = 0.80 in. (LL only)

1.20 in. (DL + LL) Dmax = 0.039 in.

Long term deflections due to creep

For 32' span girder

$I_D = z/(1+50r')$ = 1.41 in.

Total long term deflection = I_D + Dmax = 1.57 in. which is still within code limits for Total Deflection

Existing Concrete Girder Along Line C Is Adequate in Flexure, But is Overstressed in Shear at Two Columns (Gridlines 2 & 3).

DEFICIENCIES: Shear Reinforcement Spacing is Too Large at Some Locations

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:
Project Notes :

Job # M11-040

Printed: 5 OCT 2011, 12:38PM

Concrete Slender Wall

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.30, Ver:6.11.8.1

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Existing 6" wall

General Information

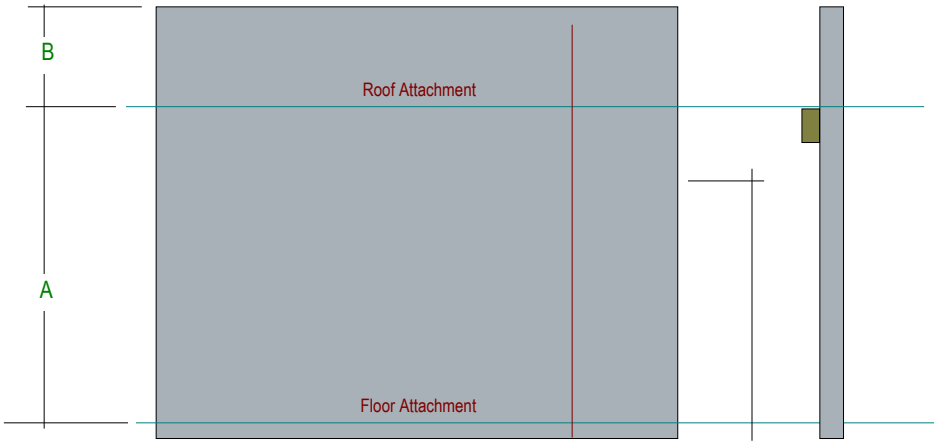
Calculations per ACI 318-08 Sec 14.8, IBC 2009, CBC 2010, ASCE 7-05

f'c : Concrete 28 day strength	=	3.50	ksi	Wall Thickness	6.0	in	Temp Diff across thickness = deg F
Fy : Rebar Yield	=	40.0	ksi	Rebar at wall center			Min Allow Out-of-Plane Defl Ratio = L / 150.0
Ec : Concrete Elastic Modulus	=	3,372.17	ksi	Rebar "d" distance	3.0	in	Minimum Vertical Steel % = 0.0020
λ : Lt Wt Conc Factor	=	1.0		Lower Level Rebar . . .			Using Stiffness Reduction Factor per ACI R.10.12.3
Fr : Rupture Modulus	=	295.80	psi	Bar Size	# 4		
Max % of ρ balanced	=	0.60		Bar Spacing	12.0	in	
Max Pu/Ag = f'c *	=	0.060					
Concrete Density	=	144.0	pcf				
Width of Design Strip	=	12.0	in				

One-Story Wall Dimensions

A Clear Height = 12.0 ft
B Parapet height = ft

Wall Support Condition: Top & Bottom Pinned



Vertical Loads

Vertical Uniform Loads . . . (Applied per foot of Strip Width)		DL : Dead Load	Lr : Roof Live Load	Lf : Floor Live Load	S : Snow Load	
Ledger Load	Eccentricity in	0.880	0.2560			k/ft
Concentric Load						k/ft

Lateral Loads

Full area WIND load 15.0 psf
Fp 1.0 = 34.0 psf

Wall Weight Seismic Load Input Method : Direct entry of Lateral Wall Weight
Seismic Wall Lateral Load 34.0 psf

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check	Maximum Bending Stress Ratio = **0.420**			
	+1.050D+1.280Lr+1.40E	Max Mu	0.8592 k-ft	Phi * Mn	2.046 k-ft
PASS	Service Deflection Check	Min. Defl. Ratio	9,242.33	Max Allow Ratio	150.0
	D + L + S + E/1.4	Max. Deflection	0.01558 in	Max. Allow. Defl.	0.960 in
PASS	Axial Load Check	Max Pu / Ag	26.631 psi	0.06 * f'c	210.0 psi
	+1.40D at 5.20 to 5.60				
PASS	Reinforcing Limit Check	Controlling As/bd	0.005556	As/bd = 0.50 rho bal	0.02598
	+1.40D				
FAIL	Minimum Moment Check	Mcracking	1.775 k-ft	Minimum Phi Mn	1.733 k-ft
	+1.40D	Maximum Reactions . . .	*for Load Combination....*		
		Top Horizontal	E Only		0.2040 k
		Base Horizontal	E Only		0.2040 k
		Vertical Reaction	D + L + Lr		2.0 k

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 5 OCT 2011, 12:38PM

Concrete Slender Wall

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.30, Ver:6.11.8.1

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Existing 6" wall

Design Maximum Combinations - Moments

Load Combination	Axial Load Pu k	0.06*f'c*b*t k	Moment Values Mcr k-ft	Mu k-ft	Phi	Phi Mn k-ft	As in^2	As Eff in^2	As Ratio	0.6 * rho bal
+1.40D at 5.20 to 5.60	1.917	15.120	1.77	0.00	0.88	2.08	0.200	0.248	0.0056	0.0260
+1.050D+1.280Lr+1.40E at 5.60 to 6.00	1.736	15.120	1.77	0.86	0.88	2.05	0.200	0.243	0.0056	0.0260
+0.90D+1.10E at 5.60 to 6.00	1.207	15.120	1.77	0.67	0.89	1.96	0.200	0.230	0.0056	0.0260

Design Maximum Combinations - Deflections

Load Combination	Axial Load Pu k	Moment Values Mcr k-ft	Mactual k-ft	Stiffness I gross in^4	I cracked in^4	I effective in^4	Deflections Deflection in	Defl. Ratio
D + L + Lr at 5.60 to 6.00	1.597	1.77	0.00	216.00	13.02	216.000	0.000	0.0
D + L + W at 5.60 to 6.00	1.341	1.77	0.27	216.00	12.73	216.000	0.010	14,963.8
D + L + W + S/2 at 5.60 to 6.00	1.341	1.77	0.27	216.00	12.73	216.000	0.010	14,963.8
D + L + S + W/2 at 5.60 to 6.00	1.341	1.77	0.14	216.00	12.73	216.000	0.005	29,927.6
D + L + S + E/1.4 at 5.60 to 6.00	1.341	1.77	0.44	216.00	12.73	216.000	0.016	9,242.3
D + 0.5(L+Lr) + 0.7W at 5.60 to 6.00	1.469	1.77	0.19	216.00	12.88	216.000	0.007	21,368.9
D + 0.5(L+Lr)+ 0.7E at 5.60 to 6.00	1.469	1.77	0.43	216.00	12.88	216.000	0.015	9,427.5

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	1.744 k
S Only	0.0 k	0.00 k	0.000 k
W Only	0.1 k	0.09 k	0.000 k
E Only	0.2 k	0.20 k	0.000 k
D + L + Lr	0.0 k	0.00 k	2.000 k
D + L + S	0.0 k	0.00 k	1.744 k
D + L + W + S/2	0.1 k	0.09 k	1.744 k
D + L + S + W/2	0.0 k	0.05 k	1.744 k
D + L + S + E/1.4	0.1 k	0.15 k	1.744 k

Program: PL-02 COLLECTOR STRESS ANALYSIS

Designation: Wall Line 1 & 5

Input Data:

Total Lateral Load:	104330	lbs.
Overall Collector Length:	121	ft.

Wall No.	Length (ft)	Begin (ft.)*	Wall No.	Length (ft)	Begin (ft.)*
1	7	1	6		
2	7	113	7		
3			8		
4			9		
5			10		

* Begin location is distance from left end of overall collector length to left end of shear wall.

Collector Load =	862.2	plf
Shear Wall Load =	7452.1	plf

Collector Loads:

Wall No.	Begin (lbs.)	End (lbs.)	Wall No.	Begin (lbs.)	End (lbs.)
1	862	-45267	6	0	0
2	45267	-862	7	0	0
3	0	0	8	0	0
4	0	0	9	0	0
5	0	0	10	0	0

Program: PL-02 COLLECTOR STRESS ANALYSIS

Designation: Wall Line A

Input Data:

Total Lateral Load:	104330	lbs.
Overall Collector Length:	97	ft.

Wall No.	Length (ft)	Begin (ft.)*	Wall No.	Length (ft)	Begin (ft.)*
1	8.5	25	6		
2	9.5	49	7		
3			8		
4			9		
5			10		

* Begin location is distance from left end of overall collector length to left end of shear wall.

Collector Load =	1075.6	plf
Shear Wall Load =	5796.1	plf

Collector Loads:

Wall No.	Begin (lbs.)	End (lbs.)	Wall No.	Begin (lbs.)	End (lbs.)
1	26889	-13235	6	0	0
2	3436	-41409	7	0	0
3	0	0	8	0	0
4	0	0	9	0	0
5	0	0	10	0	0

Program: PL-02 COLLECTOR STRESS ANALYSIS

Designation: Wall Line F

Input Data:

Total Lateral Load:	104330	lbs.
Overall Collector Length:	97	ft.

Wall No.	Length (ft)	Begin (ft.)*	Wall No.	Length (ft)	Begin (ft.)*
1	13	25	6		
2	7	89	7		
3			8		
4			9		
5			10		

* Begin location is distance from left end of overall collector length to left end of shear wall.

Collector Load =	1075.6	plf
Shear Wall Load =	5216.5	plf

Collector Loads:

Wall No.	Begin (lbs.)	End (lbs.)	Wall No.	Begin (lbs.)	End (lbs.)
1	26889	-26943	6	0	0
2	27911	-1076	7	0	0
3	0	0	8	0	0
4	0	0	9	0	0
5	0	0	10	0	0

Program: PC-03 CONCRETE SHEAR WALL

Designation: SW1.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	7	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	8	(3-10)
Number of Bars	2	
Right End Bar Size	8	(3-10)
Number of Bars	2	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	5.63		3.5	
2	6.3		3.5	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	52.16	12
2	1.3	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Double Reinforcing Curtain Required

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check:

ϕ = 0.85

V_u = 74.8 kips ϕV_n = 107.6 kips OK DCR = 0.70

Bending Check:

ϕ = 0.88

M_u left = 887.2 ft-kips ϕM_n = 364.1 ft-kips Overstressed! DCR = 2.44

M_u right = 887.2 ft-kips ϕM_n = 364.1 ft-kips Overstressed! DCR = 2.44

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	12.6	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.6	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.6	sq.in.	Supplied Number Of Bars =	2
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWA.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	8.5	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	9	(3-10)
Number of Bars	1	
Right End Bar Size	9	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	6.83		4.25	
2	7.7		4.25	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	47.12	12
2	1.6	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Single Reinforcing Curtain Allowed

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check: ϕ = 0.85

V_u =	68.2	kips	ϕV_n =	120.1	kips	OK	DCR =	0.57

Bending Check: ϕ = 0.88

M_u left =	805.1	ft-kips	ϕM_n =	285.4	ft-kips	Overstressed!	DCR =	2.82
M_u right =	805.1	ft-kips	ϕM_n =	285.4	ft-kips	Overstressed!	DCR =	2.82

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	15.3	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.7	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.0	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWA.2.2

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	9.5	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	10	(3-10)
Number of Bars	1	
Right End Bar Size	10	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	7.64		4.75	
2	8.6		4.75	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	57.21	12
2	1.8	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Double Reinforcing Curtain Required

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check:

ϕ = 0.85

V_u = 82.6 kips ϕV_n = 134.2 kips OK DCR = 0.62

Bending Check:

ϕ = 0.88

M_u left = 976.2 ft-kips ϕM_n = 406.2 ft-kips Overstressed! DCR = 2.40

M_u right = 976.2 ft-kips ϕM_n = 406.2 ft-kips Overstressed! DCR = 2.40

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	17.1	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.8	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.3	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWF.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	13	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	8	(3-10)
Number of Bars	2	
Right End Bar Size	8	(3-10)
Number of Bars	2	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	10.45		6.5	
2	11.7		6.5	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	77.32	12
2	2.5	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing IS Required			
Maximum Axial Load Check:	OK			

Shear Check:

ϕ = 0.85

V_u =	111.7	kips	ϕV_n =	183.7	kips	OK	DCR = 0.61

Bending Check:

ϕ = 0.88

M_u left =	1320.0	ft-kips	ϕM_n =	699.2	ft-kips	Overstressed!	DCR = 1.89
M_u right =	1320.0	ft-kips	ϕM_n =	699.2	ft-kips	Overstressed!	DCR = 1.89

Boundary Member Check:

Boundary Member Required: NO

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWA.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	8.5	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	9	(3-10)
Number of Bars	1	
Right End Bar Size	9	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	6.83		4.25	
2	7.7		4.25	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	47.12	12
2	1.6	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Single Reinforcing Curtain Allowed

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check: ϕ = 0.85

V_u =	68.2	kips	ϕV_n =	120.1	kips	OK	DCR =	0.57

Bending Check: ϕ = 0.88

M_u left =	805.1	ft-kips	ϕM_n =	285.4	ft-kips	Overstressed!	DCR =	2.82
M_u right =	805.1	ft-kips	ϕM_n =	285.4	ft-kips	Overstressed!	DCR =	2.82

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	15.3	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.7	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.0	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWA.2.2

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	9.5	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	10	(3-10)
Number of Bars	1	
Right End Bar Size	10	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	7.64		4.75	
2	8.6		4.75	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	57.21	12
2	1.8	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Double Reinforcing Curtain Required

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check:

ϕ = 0.85

V_u = 82.6 kips ϕV_n = 134.2 kips OK DCR = 0.62

Bending Check:

ϕ = 0.88

M_u left = 976.2 ft-kips ϕM_n = 406.2 ft-kips Overstressed! DCR = 2.40

M_u right = 976.2 ft-kips ϕM_n = 406.2 ft-kips Overstressed! DCR = 2.40

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	17.1	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.8	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.3	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWF.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	13	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	8	(3-10)
Number of Bars	2	
Right End Bar Size	8	(3-10)
Number of Bars	2	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	10.45		6.5	
2	11.7		6.5	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	77.32	12
2	2.5	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing IS Required			
Maximum Axial Load Check:	OK			

Shear Check:

ϕ = 0.85

V_u =	111.7	kips	ϕV_n =	183.7	kips	OK	DCR = 0.61

Bending Check:

ϕ = 0.88

M_u left =	1320.0	ft-kips	ϕM_n =	699.2	ft-kips	Overstressed!	DCR = 1.89
M_u right =	1320.0	ft-kips	ϕM_n =	699.2	ft-kips	Overstressed!	DCR = 1.89

Boundary Member Check:

Boundary Member Required: NO

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWF.2.2

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	7	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	9	(3-10)
Number of Bars	1	
Right End Bar Size	9	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	5.63		3.5	
2	6.3		3.5	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	27.01	12
2	1.3	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK
Double Curtain Reinforcing Check:	Single Reinforcing Curtain Allowed			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing IS Required			
Maximum Axial Load Check:	OK			

Shear Check: ϕ = 0.85

V_u =	39.6	kips	ϕV_n =	107.6	kips	OK	DCR = 0.37

Bending Check: ϕ = 0.88

M_u left =	464.7	ft-kips	ϕM_n =	232.4	ft-kips	Overstressed!	DCR = 2.00
M_u right =	464.7	ft-kips	ϕM_n =	232.4	ft-kips	Overstressed!	DCR = 2.00

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	12.6	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.6	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.0	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SW1.1.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	49	ft.
Wall Thickness	12	in.
Unsupported Height	13	ft.
Horizontal Bars	5	(3-9)
Spacing	10	in.
Vertical Bars	6	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	6	(3-10)
Number of Bars	6	
Right End Bar Size	6	(3-10)
Number of Bars	6	
Distance From End	12	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	95.6		24.5	
2				
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	225.3	13
2	10.2	6.5
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0015	Actual:	0.0031	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0020	Actual:	0.0026	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	10	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing Not Required			
Maximum Axial Load Check:	OK			

Shear Check: ϕ = 0.85

V_u =	329.7	kips	ϕV_n =	1347.4	kips	OK	DCR = 0.24

Bending Check: ϕ = 0.89

M_u left =	4193.3	ft-kips	ϕM_n =	4503.2	ft-kips	OK	DCR = 0.93
M_u right =	4193.3	ft-kips	ϕM_n =	4503.2	ft-kips	OK	DCR = 0.93

Boundary Member Check:

Boundary Member Required: NO

Program: PC-03 CONCRETE SHEAR WALL

Designation: SW1.1.2

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	25	ft.
Wall Thickness	12	in.
Unsupported Height	13	ft.
Horizontal Bars	5	(3-9)
Spacing	10	in.
Vertical Bars	6	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	6	(3-10)
Number of Bars	6	
Right End Bar Size	6	(3-10)
Number of Bars	6	
Distance From End	6	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	48.8		12.5	
2				
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	119.5	13
2	10.2	6.5
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0015	Actual:	0.0031	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0020	Actual:	0.0026	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	10	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing Not Required			
Maximum Axial Load Check:	OK			

Shear Check: ϕ = 0.85

V_u =	181.6	kips	ϕV_n =	687.4	kips	OK	DCR =	0.26

Bending Check: ϕ = 0.89

M_u left =	2267.7	ft-kips	ϕM_n =	2292.8	ft-kips	OK	DCR =	0.99
M_u right =	2267.7	ft-kips	ϕM_n =	2292.8	ft-kips	OK	DCR =	0.99

Boundary Member Check:

Boundary Member Required: NO

Program: PC-03 CONCRETE SHEAR WALL

Designation: SW5.1.5

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	5.67	ft.
Wall Thickness	18	in.
Unsupported Height	5.75	ft.
Horizontal Bars	5	(3-9)
Spacing	12	in.
Vertical Bars	5	(3-9)
Spacing	12	in.
Single or Double Curtain	D	S/D
Left End Bar Size	6	(3-10)
Number of Bars	3	
Right End Bar Size	6	(3-10)
Number of Bars	3	
Distance From End	3	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	18.9		2.835	
2				
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	27.4	5.75
2	4	2.875
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0012	Actual:	0.0029	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0020	Actual:	0.0029	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing Not Required			
Maximum Axial Load Check:	OK			

Shear Check:

ϕ = 0.85

V_u =	44.0	kips	ϕV_n =	243.4	kips	OK	DCR =	0.18

Bending Check:

ϕ = 0.89

M_u left =	236.7	ft-kips	ϕM_n =	252.6	ft-kips	OK	DCR =	0.94
M_u right =	236.7	ft-kips	ϕM_n =	252.6	ft-kips	OK	DCR =	0.94

Boundary Member Check:

Boundary Member Required: NO

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 Licensee : AHEARN & KNOX, INC

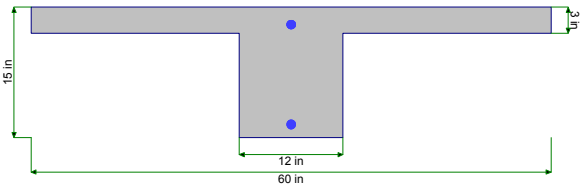
Description : Joist J-1 (betw. gl 2 & 3; assembly)

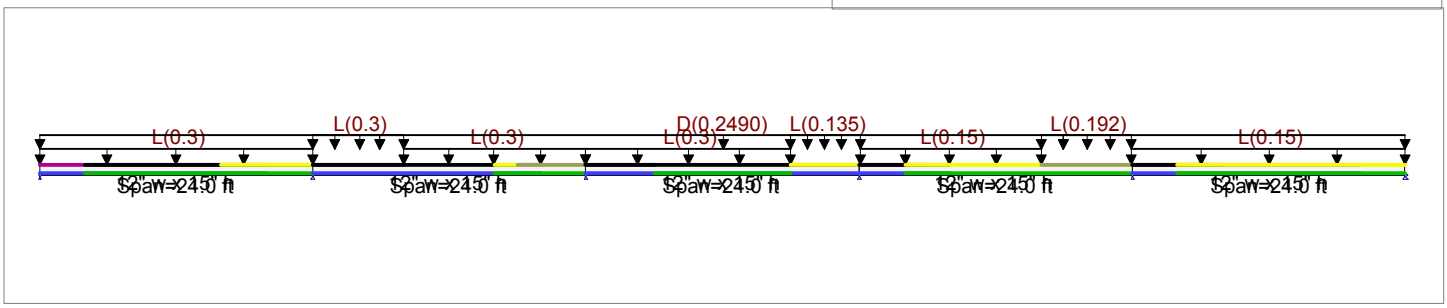
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c $^{1/2}$ * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup	=	2

Load Combination 2006 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 12.0 in, Total Height = 15.0 in, Top Flange Width = 60.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span

Span #2 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 16.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 to 24.0 ft in this span

Span #3 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 6.0 to 18.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 24.0 ft in this span

Span #4 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span

Span #5 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.0830
Uniform Load on ALL spans : D = 0.0830 ksf, Tributary Width = 3.0 ft

Load for Span Number 1
Uniform Load : L = 0.10 ksf, Tributary Width = 3.0 ft, (corridor LL)

Load for Span Number 2
Uniform Load : L = 0.10 ksf, Extent = 8.0 -->> 24.0 ft, Tributary Width = 3.0 ft, (assembly LL)
Uniform Load : L = 0.10 ksf, Extent = 0.0 -->> 8.0 ft, Tributary Width = 3.0 ft, (coridor)

Load for Span Number 3
Uniform Load : L = 0.10 ksf, Extent = 0.0 -->> 18.0 ft, Tributary Width = 3.0 ft, (assembly LL)
Uniform Load : L = 0.0450 ksf, Extent = 18.0 -->> 24.0 ft, Tributary Width = 3.0 ft, (office LL)

Load for Span Number 4
Uniform Load : L = 0.050 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 3.0 ft, (office LL)
Uniform Load : L = 0.0640 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 3.0 ft, (corridor LL)

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381

Licensee : AHEARN & KNOX, INC

Description : Joist J-1 (betw. gl 2 & 3; assembly)

Load for Span Number 5

Uniform Load : L = 0.050 ksf, Tributary Width = 3.0 ft, (office LL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.882** : 1

Section used for this span **Typical Section**

Mu : Applied -50.717 k-ft

Mn * Phi : Allowable 57.522 k-ft

Load Combination +1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (LL*L*)

Location of maximum on span 0.000 ft

Span # where maximum occurs Span # 2

Maximum Deflection

Max Downward L+Lr+S Deflection	0.111 in	Ratio =	2595
Max Upward L+Lr+S Deflection	-0.066 in	Ratio =	4357
Max Downward Total Deflection	0.154 in	Ratio =	1869
Max Upward Total Deflection	-0.041 in	Ratio =	6972

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5	Support 6
Overall MAXimum	5.442	15.052	13.998	11.326	11.528	4.012
D Only	2.359	6.762	5.819	5.819	6.762	2.359
L Only, LL Comb Run (****L)	0.004	-0.026	0.103	-0.388	2.347	1.559
L Only, LL Comb Run (***L*)	-0.014	0.081	-0.325	2.123	2.264	-0.194
L Only, LL Comb Run (***LL)	-0.009	0.056	-0.222	1.736	4.611	1.365
L Only, LL Comb Run (**L**)	0.090	-0.539	3.950	3.126	-0.501	0.083
L Only, LL Comb Run (**L*L)	0.094	-0.564	4.054	2.738	1.846	1.642
L Only, LL Comb Run (**LL*)	0.076	-0.457	3.625	5.249	1.763	-0.110
L Only, LL Comb Run (**LLL)	0.081	-0.483	3.728	4.861	4.110	1.449
L Only, LL Comb Run (*L***)	-0.353	3.919	4.125	-0.620	0.155	-0.026
L Only, LL Comb Run (*L**L)	-0.349	3.893	4.229	-1.008	2.502	1.533
L Only, LL Comb Run (*L*L*)	-0.367	4.000	3.800	1.503	2.419	-0.219
L Only, LL Comb Run (*L*LL)	-0.362	3.974	3.903	1.116	4.766	1.339
L Only, LL Comb Run (*LL**)	-0.263	3.380	8.076	2.505	-0.346	0.058
L Only, LL Comb Run (*LL*L)	-0.259	3.354	8.179	2.118	2.001	1.616
L Only, LL Comb Run (*LLL*)	-0.277	3.461	7.750	4.629	1.918	-0.136
L Only, LL Comb Run (*LLLL)	-0.273	3.436	7.854	4.241	4.265	1.423
L Only, LL Comb Run (L****)	3.118	4.694	-0.775	0.207	-0.052	0.009
L Only, LL Comb Run (L***L)	3.122	4.668	-0.672	-0.181	2.295	1.567
L Only, LL Comb Run (L**L*)	3.104	4.775	-1.101	2.330	2.212	-0.185
L Only, LL Comb Run (L**LL)	3.108	4.749	-0.997	1.943	4.559	1.374
L Only, LL Comb Run (L*L**)	3.207	4.155	3.175	3.332	-0.552	0.092
L Only, LL Comb Run (L*L*L)	3.212	4.129	3.279	2.945	1.795	1.651
L Only, LL Comb Run (L*LL*)	3.194	4.237	2.850	5.456	1.712	-0.102
L Only, LL Comb Run (L*LLL)	3.198	4.211	2.953	5.068	4.058	1.457
L Only, LL Comb Run (LL***)	2.765	8.612	3.350	-0.413	0.103	-0.017
L Only, LL Comb Run (LL**L)	2.769	8.587	3.454	-0.801	2.450	1.542
L Only, LL Comb Run (LL*L*)	2.751	8.694	3.025	1.710	2.367	-0.211
L Only, LL Comb Run (LL*LL)	2.755	8.668	3.128	1.322	4.714	1.348
L Only, LL Comb Run (LLL**)	2.854	8.074	7.301	2.712	-0.397	0.066
L Only, LL Comb Run (LLL*L)	2.859	8.048	7.404	2.325	1.950	1.625
L Only, LL Comb Run (LLLL*)	2.841	8.155	6.975	4.836	1.867	-0.127
L Only, LL Comb Run (LLLLL)	2.845	8.129	7.079	4.448	4.213	1.431
D+L, LL Comb Run (****L)	2.363	6.736	5.922	5.431	9.109	3.918
D+L, LL Comb Run (***L*)	2.345	6.844	5.493	7.942	9.026	2.165
D+L, LL Comb Run (***LL)	2.350	6.818	5.597	7.555	11.373	3.724
D+L, LL Comb Run (**L**)	2.449	6.224	9.769	8.944	6.262	2.442
D+L, LL Comb Run (**L*L)	2.453	6.198	9.872	8.557	8.609	4.001
D+L, LL Comb Run (**LL*)	2.435	6.305	9.444	11.068	8.526	2.249
D+L, LL Comb Run (**LLL)	2.439	6.279	9.547	10.680	10.872	3.808
D+L, LL Comb Run (*L***)	2.006	10.681	9.944	5.199	6.917	2.333
D+L, LL Comb Run (*L**L)	2.010	10.655	10.047	4.811	9.264	3.892
D+L, LL Comb Run (*L*L*)	1.992	10.762	9.619	7.322	9.181	2.140
D+L, LL Comb Run (*L*LL)	1.997	10.736	9.722	6.934	11.528	3.698
D+L, LL Comb Run (*LL**)	2.096	10.142	13.894	8.324	6.417	2.417
D+L, LL Comb Run (*LL*L)	2.100	10.117	13.998	7.937	8.764	3.975
D+L, LL Comb Run (*LLL*)	2.082	10.224	13.569	10.448	8.681	2.223
D+L, LL Comb Run (*LLLL)	2.086	10.198	13.672	10.060	11.027	3.782
D+L, LL Comb Run (L****)	5.390	11.653	4.904	6.063	6.701	2.369
D+L, LL Comb Run (L***L)	5.392	11.632	5.004	5.676	9.048	3.928
D+L, LL Comb Run (L**L*)	5.382	11.722	4.588	8.184	8.966	2.175
D+L, LL Comb Run (L**LL)	5.384	11.700	4.688	7.797	11.312	3.734

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Joist J-1 (betw. gl 2 & 3; assembly)

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5	Support 6
D+L, LL Comb Run (L*L**)	5.439	11.206	8.790	9.206	6.196	2.453
D+L, LL Comb Run (L*L*L)	5.442	11.185	8.890	8.819	8.543	4.012
D+L, LL Comb Run (L*LL*)	5.432	11.273	8.474	11.326	8.461	2.260
D+L, LL Comb Run (L*LLL)	5.434	11.252	8.574	10.940	10.808	3.818
D+L, LL Comb Run (LL***)	5.290	15.003	9.425	5.342	6.882	2.339
D+L, LL Comb Run (LL**L)	5.289	14.987	9.521	4.956	9.228	3.898
D+L, LL Comb Run (LL*L*)	5.290	15.052	9.122	7.460	9.147	2.145
D+L, LL Comb Run (LL*LL)	5.290	15.037	9.218	7.074	11.493	3.704
D+L, LL Comb Run (LLL**)	5.279	14.691	13.218	8.507	6.371	2.424
D+L, LL Comb Run (LLL*L)	5.278	14.676	13.314	8.122	8.717	3.983
D+L, LL Comb Run (LLLL*)	5.281	14.737	12.917	10.625	8.636	2.230
D+L, LL Comb Run (LLLLL)	5.280	14.723	13.012	10.239	10.983	3.789

Shear Stirrup Requirements

Between 0.00 to 19.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 20.40 to 27.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 27.60 to 45.00 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 45.60 to 50.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 51.00 to 70.20 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 70.80 to 71.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 72.00 to 95.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 96.00 to 96.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 97.20 to 119.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-43.98	57.52	0.76
Span # 2	2	24.000	-50.72	57.52	0.88
Span # 3	3	48.000	-42.92	57.52	0.75
Span # 4	4	72.000	-35.54	57.52	0.62
Span # 5	5	96.000	-35.80	57.52	0.62
+1.40D					
Span # 1	1	23.400	-18.16	57.52	0.32
Span # 2	2	24.000	-21.14	57.52	0.37
Span # 3	3	48.000	-15.85	57.52	0.28
Span # 4	4	95.400	-18.56	57.52	0.32
Span # 5	5	96.000	-21.14	57.52	0.37
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-15.40	57.52	0.27
Span # 2	2	24.000	-17.95	57.52	0.31
Span # 3	3	48.000	-14.25	57.52	0.25
Span # 4	4	95.400	-24.87	57.52	0.43
Span # 5	5	96.000	-27.38	57.52	0.48
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-16.07	57.52	0.28
Span # 2	2	24.000	-18.64	57.52	0.32
Span # 3	3	71.400	-19.05	57.52	0.33
Span # 4	4	84.000	16.39	35.90	0.46
Span # 5	5	96.000	-25.55	57.52	0.44
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-15.91	57.52	0.28
Span # 2	2	24.000	-18.47	57.52	0.32
Span # 3	3	71.400	-16.65	57.52	0.29
Span # 4	4	95.400	-30.37	57.52	0.53
Span # 5	5	96.000	-34.81	57.52	0.61
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.77	35.90	0.24
Span # 2	2	47.400	-24.96	57.52	0.43
Span # 3	3	48.000	-27.37	57.52	0.48
Span # 4	4	72.000	-26.40	57.52	0.46
Span # 5	5	96.000	-14.91	57.52	0.26
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.80	35.90	0.25
Span # 2	2	47.400	-25.60	57.52	0.45
Span # 3	3	48.000	-28.04	57.52	0.49
Span # 4	4	72.000	-23.92	57.52	0.42

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381
Licensee : AHEARN & KNOX, INC

Description : Joist J-1 (betw. gl 2 & 3; assembly)

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 2	2	24.000	-27.01	57.52	0.47
Span # 3	3	48.000	-21.20	57.52	0.37
Span # 4	4	95.400	-18.01	57.52	0.31
Span # 5	5	96.000	-20.22	57.52	0.35
+1.20D+0.50L+0.20S+E, LL Comb Run (					
Span # 1	1	23.400	-23.40	57.52	0.41
Span # 2	2	24.000	-27.23	57.52	0.47
Span # 3	3	48.000	-20.35	57.52	0.35
Span # 4	4	72.000	-19.21	57.52	0.33
Span # 5	5	96.000	-19.65	57.52	0.34
+1.20D+0.50L+0.20S+E, LL Comb Run (					
Span # 1	1	23.400	-23.35	57.52	0.41
Span # 2	2	24.000	-27.18	57.52	0.47
Span # 3	3	48.000	-20.55	57.52	0.36
Span # 4	4	95.400	-19.73	57.52	0.34
Span # 5	5	96.000	-22.54	57.52	0.39
+0.90D+1.60W+1.60H					
Span # 1	1	23.400	-11.67	57.52	0.20
Span # 2	2	24.000	-13.59	57.52	0.24
Span # 3	3	48.000	-10.19	57.52	0.18
Span # 4	4	95.400	-11.93	57.52	0.21
Span # 5	5	96.000	-13.59	57.52	0.24
+0.90D+E+1.60H					
Span # 1	1	23.400	-11.67	57.52	0.20
Span # 2	2	24.000	-13.59	57.52	0.24
Span # 3	3	48.000	-10.19	57.52	0.18
Span # 4	4	95.400	-11.93	57.52	0.21
Span # 5	5	96.000	-13.59	57.52	0.24

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*L)	1	0.1540	10.800	D+L, LL Comb Run (L*L*L)	-0.0109	25.200
D+L, LL Comb Run (*L*L*)	2	0.0641	13.200	D+L, LL Comb Run (L*L*L)	-0.0280	3.600
D+L, LL Comb Run (L*L*L)	3	0.0787	10.800	D+L, LL Comb Run (L*L*L)	-0.0060	25.200
D+L, LL Comb Run (*L*L*)	4	0.0427	10.800	D+L, LL Comb Run (L*L*L)	-0.0149	20.400
D+L, LL Comb Run (L*L*L)	5	0.0843	13.200		0.0000	20.400

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 Licensee : AHEARN & KNOX, INC

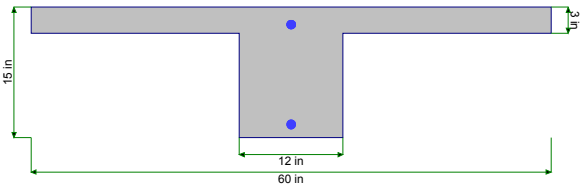
Description : Joist J-1 (betw. gl 1 & 2; corridor)

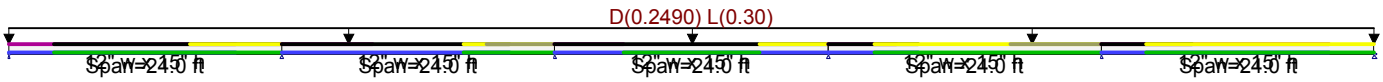
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c $^{1/2}$ * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup	=	2

Load Combination 2006 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 12.0 in, Total Height = 15.0 in, Top Flange Width = 60.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span

Span #2 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 16.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 to 24.0 ft in this span

Span #3 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 6.0 to 18.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 24.0 ft in this span

Span #4 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span

Span #5 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...

D = 0.0830, L = 0.10

Uniform Load on ALL spans : D = 0.0830, L = 0.10 ksf, Tributary Width = 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.890** : 1	Maximum Deflection	
Section used for this span	**Typical Section**	Max Downward L+Lr+S Deflection	0.113 in Ratio = 2549
Mu : Applied	-51.188 k-ft	Max Upward L+Lr+S Deflection	-0.067 in Ratio = 4301
Mn * Phi : Allowable	57.522 k-ft	Max Downward Total Deflection	0.156 in Ratio = 1846
Load Combination	1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*L*LL)	Max Upward Total Deflection	-0.044 in Ratio = 6573
Location of maximum on span	0.000 ft		
Span # where maximum occurs	Span # 5		

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381
Licensee : AHEARN & KNOX, INC

Description : Joist J-1 (betw. gl 1 & 2; corridor)

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5	Support 6
Overall MAXimum	5.448	15.098	14.275	14.275	15.107	5.448
D Only	2.359	6.762	5.819	5.819	6.762	2.359
L Only, LL Comb Run (****L)	0.009	-0.052	0.207	-0.775	4.694	3.118
L Only, LL Comb Run (***L*)	-0.026	0.155	-0.620	4.125	3.919	-0.353
L Only, LL Comb Run (***LL)	-0.017	0.103	-0.413	3.350	8.612	2.765
L Only, LL Comb Run (**L**)	0.095	-0.568	4.074	4.074	-0.568	0.095
L Only, LL Comb Run (**L*L)	0.103	-0.620	4.280	3.299	4.125	3.212
L Only, LL Comb Run (**LL*)	0.069	-0.413	3.454	8.199	3.350	-0.258
L Only, LL Comb Run (**LLL)	0.078	-0.465	3.660	7.424	8.044	2.859
L Only, LL Comb Run (*L***)	-0.353	3.919	4.125	-0.620	0.155	-0.026
L Only, LL Comb Run (*L**L)	-0.344	3.867	4.332	-1.395	4.849	3.092
L Only, LL Comb Run (*L*L*)	-0.379	4.074	3.505	3.505	4.074	-0.379
L Only, LL Comb Run (*L*LL)	-0.370	4.022	3.712	2.730	8.767	2.739
L Only, LL Comb Run (*LL**)	-0.258	3.350	8.199	3.454	-0.413	0.069
L Only, LL Comb Run (*LL*L)	-0.250	3.299	8.406	2.678	4.280	3.187
L Only, LL Comb Run (*LLL*)	-0.284	3.505	7.579	7.579	3.505	-0.284
L Only, LL Comb Run (*LLLL)	-0.276	3.454	7.786	6.804	8.199	2.833
L Only, LL Comb Run (L****)	3.118	4.694	-0.775	0.207	-0.052	0.009
L Only, LL Comb Run (L***L)	3.126	4.642	-0.568	-0.568	4.642	3.126
L Only, LL Comb Run (L**L*)	3.092	4.849	-1.395	4.332	3.867	-0.344
L Only, LL Comb Run (L**LL)	3.100	4.797	-1.189	3.557	8.561	2.773
L Only, LL Comb Run (L*L**)	3.212	4.125	3.299	4.280	-0.620	0.103
L Only, LL Comb Run (L*L*L)	3.221	4.074	3.505	3.505	4.074	3.221
L Only, LL Comb Run (L*LL*)	3.187	4.280	2.678	8.406	3.299	-0.250
L Only, LL Comb Run (L*LLL)	3.195	4.229	2.885	7.631	7.992	2.868
L Only, LL Comb Run (LL***)	2.765	8.612	3.350	-0.413	0.103	-0.017
L Only, LL Comb Run (LL**L)	2.773	8.561	3.557	-1.189	4.797	3.100
L Only, LL Comb Run (LL*L*)	2.739	8.767	2.730	3.712	4.022	-0.370
L Only, LL Comb Run (LL*LL)	2.747	8.716	2.937	2.937	8.716	2.747
L Only, LL Comb Run (LLL**)	2.859	8.044	7.424	3.660	-0.465	0.078
L Only, LL Comb Run (LLL*L)	2.868	7.992	7.631	2.885	4.229	3.195
L Only, LL Comb Run (LLLL*)	2.833	8.199	6.804	7.786	3.454	-0.276
L Only, LL Comb Run (LLLLL)	2.842	8.147	7.011	7.011	8.147	2.842
D+L, LL Comb Run (****L)	2.369	6.701	6.063	4.904	11.653	5.390
D+L, LL Comb Run (***L*)	2.333	6.917	5.199	9.944	10.681	2.006
D+L, LL Comb Run (***LL)	2.339	6.881	5.343	9.419	15.013	5.285
D+L, LL Comb Run (**L**)	2.454	6.194	9.892	9.892	6.194	2.454
D+L, LL Comb Run (**L*L)	2.465	6.128	10.155	8.909	11.181	5.442
D+L, LL Comb Run (**LL*)	2.428	6.349	9.272	14.018	10.113	2.101
D+L, LL Comb Run (**LLL)	2.436	6.302	9.459	13.325	14.684	5.273
D+L, LL Comb Run (*L***)	2.006	10.681	9.944	5.199	6.917	2.333
D+L, LL Comb Run (*L**L)	2.016	10.621	10.183	4.301	11.784	5.375
D+L, LL Comb Run (*L*L*)	1.980	10.836	9.324	9.324	10.836	1.980
D+L, LL Comb Run (*L*LL)	1.986	10.803	9.457	8.841	15.107	5.286
D+L, LL Comb Run (*LL**)	2.101	10.113	14.018	9.272	6.349	2.428
D+L, LL Comb Run (*LL*L)	2.111	10.048	14.275	8.308	11.309	5.428
D+L, LL Comb Run (*LLL*)	2.075	10.268	13.398	13.398	10.268	2.075
D+L, LL Comb Run (*LLLL)	2.082	10.224	13.573	12.751	14.774	5.276
D+L, LL Comb Run (L****)	5.390	11.653	4.904	6.063	6.701	2.369
D+L, LL Comb Run (L***L)	5.396	11.601	5.143	5.143	11.601	5.396
D+L, LL Comb Run (L**L*)	5.375	11.784	4.301	10.183	10.621	2.016
D+L, LL Comb Run (L**LL)	5.377	11.755	4.443	9.645	14.974	5.285
D+L, LL Comb Run (L*L**)	5.442	11.181	8.909	10.155	6.128	2.465
D+L, LL Comb Run (L*L*L)	5.448	11.125	9.166	9.166	11.125	5.448
D+L, LL Comb Run (L*LL*)	5.428	11.309	8.308	14.275	10.048	2.111
D+L, LL Comb Run (L*LLL)	5.432	11.270	8.493	13.569	14.642	5.274
D+L, LL Comb Run (LL***)	5.290	15.003	9.425	5.342	6.882	2.339
D+L, LL Comb Run (LL**L)	5.290	14.964	9.651	4.442	11.755	5.377
D+L, LL Comb Run (LL*L*)	5.290	15.098	8.846	9.457	10.803	1.986
D+L, LL Comb Run (LL*LL)	5.288	15.082	8.970	8.964	15.092	5.283
D+L, LL Comb Run (LLL**)	5.278	14.673	13.333	9.458	6.303	2.436
D+L, LL Comb Run (LLL*L)	5.279	14.631	13.576	8.491	11.271	5.432
D+L, LL Comb Run (LLLL*)	5.282	14.763	12.758	13.571	10.224	2.082
D+L, LL Comb Run (LLLLL)	5.279	14.740	12.922	12.914	14.751	5.274

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Joist J-1 (betw. gl 1 & 2; corridor)

Shear Stirrup Requirements

Between 0.00 to 19.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 20.40 to 27.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 27.60 to 45.00 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 45.60 to 51.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 51.60 to 68.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 69.00 to 74.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 75.00 to 92.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 93.00 to 99.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 100.20 to 119.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-44.44	57.52	0.77
Span # 2	2	24.000	-51.19	57.52	0.89
Span # 3	3	48.000	-44.34	57.52	0.77
Span # 4	4	95.400	-44.93	57.52	0.78
Span # 5	5	96.000	-51.19	57.52	0.89
+1.40D					
Span # 1	1	23.400	-18.16	57.52	0.32
Span # 2	2	24.000	-21.14	57.52	0.37
Span # 3	3	48.000	-15.85	57.52	0.28
Span # 4	4	95.400	-18.56	57.52	0.32
Span # 5	5	96.000	-21.14	57.52	0.37
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-15.24	57.52	0.27
Span # 2	2	24.000	-17.79	57.52	0.31
Span # 3	3	48.000	-14.91	57.52	0.26
Span # 4	4	95.400	-33.84	57.52	0.59
Span # 5	5	##.###	23.10	35.90	0.64
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-16.53	57.52	0.29
Span # 2	2	24.000	-19.11	57.52	0.33
Span # 3	3	71.400	-25.90	57.52	0.45
Span # 4	4	84.000	26.00	35.90	0.72
Span # 5	5	96.000	-31.68	57.52	0.55
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-16.21	57.52	0.28
Span # 2	2	24.000	-18.78	57.52	0.33
Span # 3	3	71.400	-21.10	57.52	0.37
Span # 4	4	95.400	-44.06	57.52	0.77
Span # 5	5	96.000	-50.20	57.52	0.87
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.80	35.90	0.25
Span # 2	2	47.400	-25.70	57.52	0.45
Span # 3	3	48.000	-28.14	57.52	0.49
Span # 4	4	72.000	-28.14	57.52	0.49
Span # 5	5	96.000	-14.48	57.52	0.25
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.85	35.90	0.25
Span # 2	2	47.400	-26.98	57.52	0.47
Span # 3	3	48.000	-29.46	57.52	0.51
Span # 4	4	95.400	-30.66	57.52	0.53
Span # 5	5	##.###	23.65	35.90	0.66
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.65	35.90	0.24
Span # 2	2	47.400	-21.86	57.52	0.38
Span # 3	3	71.400	-37.08	57.52	0.64
Span # 4	4	72.000	-43.02	57.52	0.75
Span # 5	5	96.000	-28.04	57.52	0.49
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.70	35.90	0.24
Span # 2	2	47.400	-23.14	57.52	0.40
Span # 3	3	71.400	-32.28	57.52	0.56
Span # 4	4	95.400	-40.88	57.52	0.71
Span # 5	5	96.000	-46.56	57.52	0.81
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-28.79	57.52	0.50
Span # 2	2	36.000	26.00	35.90	0.72

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Joist J-1 (betw. gl 1 & 2; corridor)

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
+0.90D+1.60W+1.60H					
Span # 1	1	23.400	-11.67	57.52	0.20
Span # 2	2	24.000	-13.59	57.52	0.24
Span # 3	3	48.000	-10.19	57.52	0.18
Span # 4	4	95.400	-11.93	57.52	0.21
Span # 5	5	96.000	-13.59	57.52	0.24
+0.90D+E+1.60H					
Span # 1	1	23.400	-11.67	57.52	0.20
Span # 2	2	24.000	-13.59	57.52	0.24
Span # 3	3	48.000	-10.19	57.52	0.18
Span # 4	4	95.400	-11.93	57.52	0.21
Span # 5	5	96.000	-13.59	57.52	0.24

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*L)	1	0.1560	10.800	D+L, LL Comb Run (L*L*L)	-0.0112	25.200
D+L, LL Comb Run (*L*L*)	2	0.0667	13.200	D+L, LL Comb Run (L*L*L)	-0.0289	3.600
D+L, LL Comb Run (L*L*L)	3	0.0869	13.200		0.0000	3.600
D+L, LL Comb Run (*L*L*)	4	0.0667	10.800	D+L, LL Comb Run (L*L*L)	-0.0289	20.400
D+L, LL Comb Run (L*L*L)	5	0.1560	13.200		0.0000	20.400

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

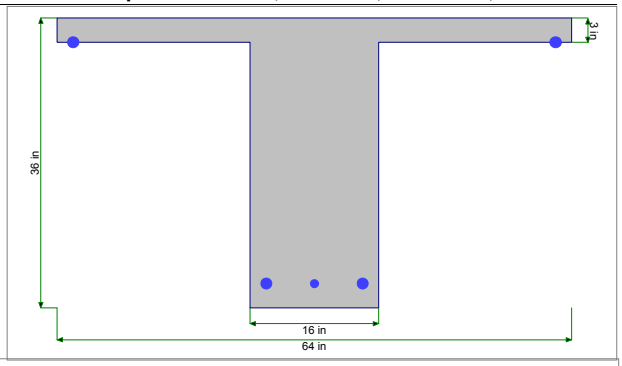
Description : Girder G1 (gridline B)

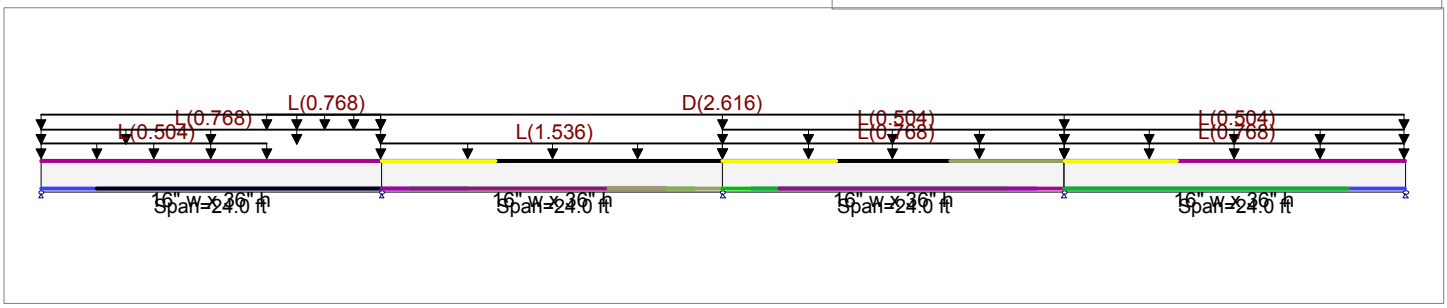
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	=	443.71 psi		Shear :	0.750
Ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup	=	2

Load Combination 2009 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 16.0 in, Total Height = 36.0 in, Top Flange Width = 64.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 8.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 4.0 to 24.0 ft in this span

Span #2 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 6.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 0.0 to 20.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 8.0 ft in this span
- 2-#8 at 3.0 in from Bottom, from 2.0 to 22.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#8 at 3.0 in from Bottom, from 16.0 to 24.0 ft in this span

Span #3 Reinforcing....
- 2-#8 at 3.0 in from Bottom, from 2.0 to 22.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 0.0 to 20.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#11 at 3.0 in from Top, from 16.0 to 24.0 ft in this span

Span #4 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#11 at 3.0 in from Top, from 0.0 to 8.0 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.1090
Uniform Load on ALL spans : D = 0.1090 ksf, Tributary Width = 24.0 ft

Load for Span Number 1
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 12.0 ft, (Multiuse LL)
Uniform Load : L = 0.0640 ksf, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0640 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 12.0 ft, (corridor LL)

Load for Span Number 2
Uniform Load : L = 0.0640 ksf, Tributary Width = 24.0 ft, (corridor LL)

Load for Span Number 3
Uniform Load : L = 0.0640 ksf, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0420 ksf, Tributary Width = 12.0 ft, (Office LL)

Load for Span Number 4
Uniform Load : L = 0.0640 ksf, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0420 ksf, Tributary Width = 12.0 ft, (office LL)

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Girder G1 (gridline B)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.542** : 1
Section used for this span **Typical Section**
Mu : Applied -352.01 k-ft
Mn * Phi : Allowable 649.56 k-ft
Load Combination 1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (LL*L)
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 2

Maximum Deflection

Max Downward L+Lr+S Deflection	0.032 in	Ratio =	8907
Max Upward L+Lr+S Deflection	-0.037 in	Ratio =	7683
Max Downward Total Deflection	0.063 in	Ratio =	4605
Max Upward Total Deflection	-0.009 in	Ratio =	32410

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum	38.150	111.577	96.809	106.834	37.978
D Only	24.665	71.753	58.299	71.753	24.665
L Only, LL Comb Run (***L)	-0.136	0.818	-3.271	19.898	13.220
L Only, LL Comb Run (**L*)	0.409	-2.453	17.445	16.627	-1.499
L Only, LL Comb Run (**LL)	0.273	-1.635	14.174	36.525	11.721
L Only, LL Comb Run (*L**)	-1.810	20.078	21.065	-2.962	0.494
L Only, LL Comb Run (*L*L)	-1.947	20.895	17.794	16.935	13.713
L Only, LL Comb Run (*LL*)	-1.401	17.625	38.510	13.665	-1.005
L Only, LL Comb Run (*LLL)	-1.538	18.442	35.239	33.562	12.214
L Only, LL Comb Run (L***)	13.441	21.955	-3.480	0.870	-0.145
L Only, LL Comb Run (L**L)	13.304	22.772	-6.751	20.768	13.075
L Only, LL Comb Run (L*L*)	13.850	19.501	13.964	17.497	-1.644
L Only, LL Comb Run (L*LL)	13.713	20.319	10.693	37.395	11.576
L Only, LL Comb Run (LL**)	11.630	42.032	17.585	-2.092	0.349
L Only, LL Comb Run (LL*L)	11.494	42.850	14.314	17.806	13.568
L Only, LL Comb Run (LLL*)	12.039	39.579	35.029	14.535	-1.150
L Only, LL Comb Run (LLLL)	11.903	40.397	31.758	34.432	12.069
D+L, LL Comb Run (***L)	24.517	72.639	54.783	91.982	37.742
D+L, LL Comb Run (**L*)	25.074	69.300	75.744	88.380	23.166
D+L, LL Comb Run (**LL)	24.995	69.776	73.933	106.155	37.333
D+L, LL Comb Run (*L**)	22.855	91.831	79.365	68.791	25.159
D+L, LL Comb Run (*L*L)	22.712	92.768	75.451	89.640	37.956
D+L, LL Comb Run (*LL*)	23.264	89.378	96.809	85.418	23.660
D+L, LL Comb Run (*LLL)	23.153	90.040	94.223	104.307	37.332
D+L, LL Comb Run (L***)	38.005	93.947	54.631	72.681	24.510
D+L, LL Comb Run (L**L)	37.947	94.617	51.366	92.701	37.673
D+L, LL Comb Run (L*L*)	38.122	92.149	71.626	89.413	22.994
D+L, LL Comb Run (L*LL)	38.150	92.351	70.188	106.834	37.308
D+L, LL Comb Run (LL**)	37.585	110.898	77.866	69.199	25.091
D+L, LL Comb Run (LL*L)	37.550	111.577	74.239	89.824	37.978
D+L, LL Comb Run (LLL*)	37.593	109.348	94.685	85.976	23.567
D+L, LL Comb Run (LLLL)	37.622	109.663	92.523	104.501	37.387

Shear Stirrup Requirements

Between 0.00 to 4.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 5.40 to 13.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 14.40 to 20.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 21.00 to 25.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 10.000 in
Between 26.40 to 32.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 33.00 to 40.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 41.40 to 45.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 46.20 to 49.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 12.000 in
Between 50.40 to 54.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 55.20 to 64.20 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 64.80 to 69.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 69.60 to 74.40 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 10.000 in
Between 75.00 to 81.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 82.20 to 90.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 91.20 to 95.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-305.27	649.56	0.47
Span # 2	2	24.000	-352.01	649.56	0.54

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Girder G1 (gridline B)

Load Combination Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 4	4	72.000	-145.30	751.89	0.19

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*)	1	0.0625	10.800	D+L, LL Comb Run (L*L*)	-0.0033	25.200
D+L, LL Comb Run (*L*L)	2	0.0291	13.200		0.0000	25.200
D+L, LL Comb Run (L*L*)	3	0.0261	10.800	D+L, LL Comb Run (*L*L)	-0.0078	20.400
D+L, LL Comb Run (*L*L)	4	0.0608	13.200		0.0000	20.400

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 Licensee : AHEARN & KNOX, INC

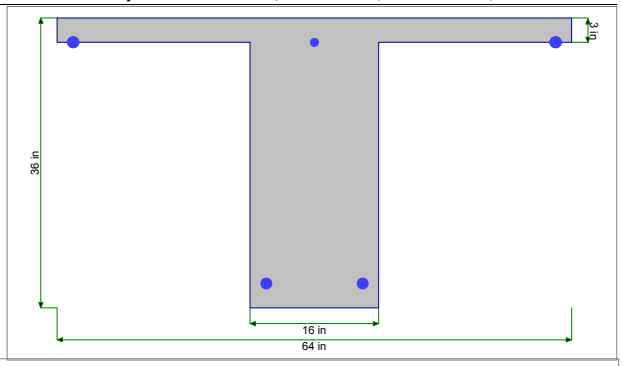
Description : Girder G2 (gridline C; fixed seating)

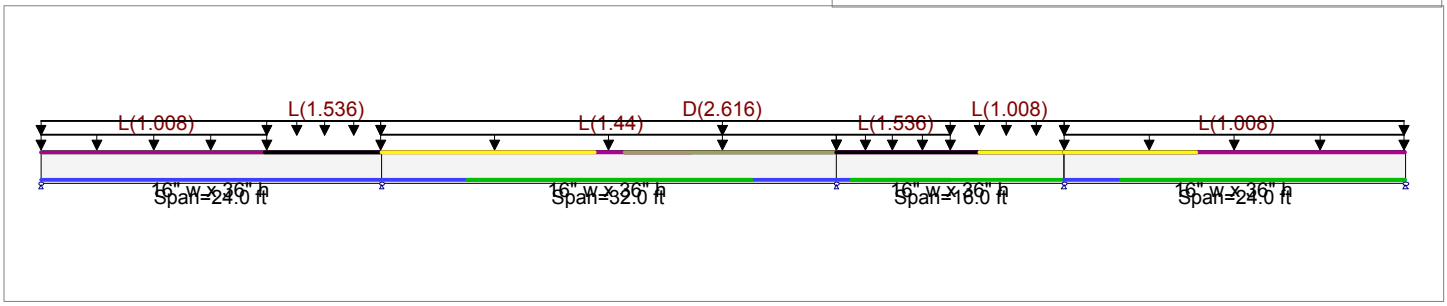
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size # =		# 3
			Number of Resisting Legs Per Stirrup =		2

Load Combination 2009 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 16.0 in, Total Height = 36.0 in, Top Flange Width = 64.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 15.750 to 24.0 ft in this span

Span #2 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 21.750 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 15.0 ft in this span
- 2-#8 at 3.0 in from Bottom, from 6.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 17.10 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 17.10 to 24.0 ft in this span

Span #3 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 8.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Top, from 10.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 1.0 to 16.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 16.0 ft in this span

Span #4 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 9.250 ft in this span
- 2-#8 at 3.0 in from Bottom, from 4.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 9.250 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.1090
Uniform Load on ALL spans : D = 0.1090 ksf, Tributary Width = 24.0 ft

Load for Span Number 1
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 24.0 ft, (Office LL)
Uniform Load : L = 0.0640 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (Corridor LL)

Load for Span Number 2
Uniform Load : L = 0.060 ksf, Tributary Width = 24.0 ft, (Assembly LL)

Load for Span Number 3
Uniform Load : L = 0.0640 ksf, Extent = 0.0 -->> 8.0 ft, Tributary Width = 24.0 ft, (Corridor LL)
Uniform Load : L = 0.0420 ksf, Extent = 8.0 -->> 16.0 ft, Tributary Width = 24.0 ft, (Office LL)

Load for Span Number 4
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (Office LL)

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Girder G2 (gridline C; fixed seating)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.841** : 1
Section used for this span **Typical Section**
Mu : Applied -369.18 k-ft
Mn * Phi : Allowable 439.06 k-ft
Load Combination 1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*LL*)
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 3

Maximum Deflection

Max Downward L+Lr+S Deflection	0.120 in	Ratio =	3204
Max Upward L+Lr+S Deflection	-0.073 in	Ratio =	5289
Max Downward Total Deflection	0.181 in	Ratio =	2118
Max Upward Total Deflection	-0.028 in	Ratio =	6791

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum	34.529	124.971	108.545	82.567	37.310
D Only	22.211	84.361	63.586	54.369	26.609
L Only, LL Comb Run (***L)	-0.099	0.434	-3.681	17.326	10.212
L Only, LL Comb Run (**L*)	0.154	-0.674	11.731	9.694	-0.553
L Only, LL Comb Run (**LL)	0.055	-0.240	8.050	27.020	9.659
L Only, LL Comb Run (*L**)	-3.190	25.474	31.350	-8.393	0.839
L Only, LL Comb Run (*L*L)	-3.289	25.908	27.669	8.933	11.052
L Only, LL Comb Run (*LL*)	-3.035	24.800	43.081	1.300	0.286
L Only, LL Comb Run (*LLL)	-3.135	25.234	39.400	18.626	10.499
L Only, LL Comb Run (L***)	11.130	18.971	-2.721	1.152	-0.115
L Only, LL Comb Run (L**L)	11.031	19.404	-6.402	18.478	10.097
L Only, LL Comb Run (L*L*)	11.284	18.297	9.010	10.845	-0.668
L Only, LL Comb Run (L*LL)	11.185	18.730	5.329	28.171	9.544
L Only, LL Comb Run (LL**)	7.940	44.445	28.628	-7.242	0.724
L Only, LL Comb Run (LL*L)	7.841	44.878	24.948	10.084	10.936
L Only, LL Comb Run (LLL*)	8.094	43.771	40.360	2.452	0.171
L Only, LL Comb Run (LLLL)	7.995	44.204	36.679	19.778	10.383
D+L, LL Comb Run (***L)	22.159	84.759	59.111	72.937	36.362
D+L, LL Comb Run (**L*)	22.294	83.829	75.204	64.110	26.051
D+L, LL Comb Run (**LL)	22.213	84.260	71.035	82.183	35.989
D+L, LL Comb Run (*L**)	19.060	108.859	98.305	43.131	27.860
D+L, LL Comb Run (*L*L)	18.992	109.053	94.061	61.991	37.310
D+L, LL Comb Run (*LL*)	19.140	108.723	108.545	53.993	27.166
D+L, LL Comb Run (*LLL)	19.095	108.770	105.038	71.975	36.883
D+L, LL Comb Run (L***)	34.487	101.063	62.618	54.820	26.564
D+L, LL Comb Run (L**L)	34.441	101.444	58.234	73.263	36.362
D+L, LL Comb Run (L*L*)	34.529	100.611	74.175	64.584	26.004
D+L, LL Comb Run (L*LL)	34.491	100.956	70.115	82.567	35.968
D+L, LL Comb Run (LL**)	31.840	124.675	97.656	43.696	27.765
D+L, LL Comb Run (LL*L)	31.741	124.971	93.294	62.569	37.249
D+L, LL Comb Run (LLL*)	31.974	124.310	108.401	54.186	27.113
D+L, LL Comb Run (LLLL)	31.909	124.446	104.758	72.205	36.858

Shear Stirrup Requirements

Between 0.00 to 3.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 4.20 to 13.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 14.40 to 19.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 20.40 to 28.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 5.000 in
Between 29.60 to 36.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 36.80 to 44.00 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 44.80 to 50.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 51.20 to 56.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 8.000 in
Between 57.20 to 61.20 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 61.60 to 68.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 69.20 to 71.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 72.00 to 72.60 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 12.000 in
Between 73.20 to 79.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 80.40 to 90.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 91.20 to 95.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-415.20	646.99	0.64
Span # 2	2	24.000	-463.37	576.91	0.80

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Girder G2 (gridline D)

Load Combination Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 4	4	85.800	121.61	474.11	0.26

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*)	1	0.0426	10.800	D+L, LL Comb Run (*L*L)	-0.0155	20.400
D+L, LL Comb Run (*L*L)	2	0.2161	17.600	D+L, LL Comb Run (*L*L)	-0.0085	32.800
D+L, LL Comb Run (*L*L)	3	0.0089	17.200	D+L, LL Comb Run (*L*L)	-0.0315	7.200
D+L, LL Comb Run (*L*L)	4	0.0744	13.200		0.0000	7.200

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 Licensee : AHEARN & KNOX, INC

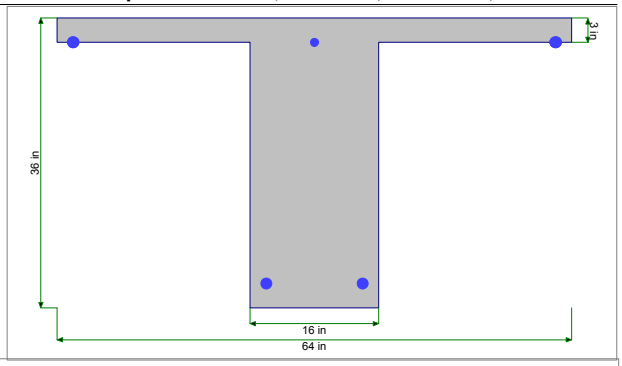
Description : Girder G2 (gridline D)

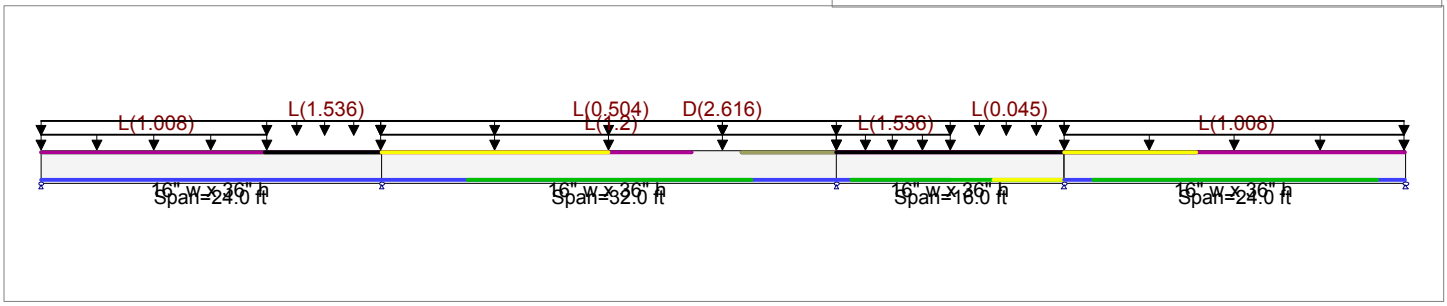
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size # =		# 3
			Number of Resisting Legs Per Stirrup =		2

Load Combination 2009 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 16.0 in, Total Height = 36.0 in, Top Flange Width = 64.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 15.750 to 24.0 ft in this span

Span #2 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 21.750 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 2-#8 at 3.0 in from Bottom, from 6.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 25.250 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 25.250 to 24.0 ft in this span

Span #3 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 8.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 11.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 1.0 to 16.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 16.0 ft in this span

Span #4 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 9.250 ft in this span
- 2-#8 at 3.0 in from Bottom, from 2.0 to 22.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 9.250 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.1090
Uniform Load on ALL spans : D = 0.1090 ksf, Tributary Width = 24.0 ft

Load for Span Number 1
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 24.0 ft, (Office LL)
Uniform Load : L = 0.0640 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (Corridor LL)

Load for Span Number 2
Uniform Load : L = 0.10 ksf, Tributary Width = 12.0 ft, (Assembly LL)
Uniform Load : L = 0.0420 ksf, Tributary Width = 12.0 ft, (office LL)

Load for Span Number 3
Uniform Load : L = 0.0640 ksf, Extent = 0.0 -->> 8.0 ft, Tributary Width = 24.0 ft, (Corridor LL)
Uniform Load : L = 0.0450 k/ft, Extent = 8.0 -->> 16.0 ft, Tributary Width = 1.0 ft, (Office LL)

Load for Span Number 4
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (Office LL)

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Girder G2 (gridline D)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.894** : 1
Section used for this span **Typical Section**
Mu : Applied -392.60 k-ft
Mn * Phi : Allowable 439.06 k-ft
Load Combination 1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*LL*)
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 3

Maximum Deflection

Max Downward L+Lr+S Deflection	0.155 in	Ratio =	2481
Max Upward L+Lr+S Deflection	-0.072 in	Ratio =	5354
Max Downward Total Deflection	0.216 in	Ratio =	1776
Max Upward Total Deflection	-0.032 in	Ratio =	6086

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum	34.517	129.909	112.925	76.522	37.528
D Only	22.211	84.361	63.586	54.369	26.609
L Only, LL Comb Run (***L)	-0.099	0.434	-3.681	17.326	10.212
L Only, LL Comb Run (**L*)	0.108	-0.474	9.797	3.513	-0.296
L Only, LL Comb Run (**LL)	0.009	-0.041	6.117	20.839	9.916
L Only, LL Comb Run (*L**)	-3.774	30.144	37.097	-9.932	0.993
L Only, LL Comb Run (*L*L)	-3.873	30.578	33.416	7.394	11.205
L Only, LL Comb Run (*LL*)	-3.666	29.670	46.894	-6.420	0.697
L Only, LL Comb Run (*LLL)	-3.765	30.104	43.213	10.906	10.909
L Only, LL Comb Run (L***)	11.130	18.971	-2.721	1.152	-0.115
L Only, LL Comb Run (L**L)	11.031	19.404	-6.402	18.478	10.097
L Only, LL Comb Run (L*L*)	11.238	18.496	7.076	4.664	-0.411
L Only, LL Comb Run (L*LL)	11.139	18.930	3.395	21.990	9.801
L Only, LL Comb Run (LL**)	7.356	49.115	34.376	-8.780	0.878
L Only, LL Comb Run (LL*L)	7.257	49.549	30.695	8.546	11.090
L Only, LL Comb Run (LLL*)	7.464	48.640	44.173	-5.268	0.582
L Only, LL Comb Run (LLLL)	7.365	49.074	40.492	12.058	10.794
D+L, LL Comb Run (***L)	22.159	84.759	59.111	72.937	36.362
D+L, LL Comb Run (**L*)	22.263	83.999	73.294	57.918	26.309
D+L, LL Comb Run (**LL)	22.196	84.412	69.017	76.172	36.179
D+L, LL Comb Run (*L**)	18.191	114.026	103.990	41.343	28.113
D+L, LL Comb Run (*L*L)	18.140	114.157	99.805	60.226	37.528
D+L, LL Comb Run (*LL*)	18.258	113.845	112.925	45.582	27.703
D+L, LL Comb Run (*LLL)	18.228	113.884	109.103	64.048	37.240
D+L, LL Comb Run (L***)	34.487	101.063	62.618	54.820	26.564
D+L, LL Comb Run (L**L)	34.441	101.444	58.234	73.263	36.362
D+L, LL Comb Run (L*L*)	34.517	100.745	72.293	58.382	26.263
D+L, LL Comb Run (L*LL)	34.476	101.102	68.122	76.522	36.170
D+L, LL Comb Run (LL**)	30.957	129.726	103.781	41.580	28.037
D+L, LL Comb Run (LL*L)	30.883	129.909	99.567	60.443	37.470
D+L, LL Comb Run (LLL*)	31.029	129.570	112.628	45.862	27.638
D+L, LL Comb Run (LLLL)	30.985	129.618	108.922	64.170	37.225

Shear Stirrup Requirements

Between 0.00 to 3.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 4.20 to 13.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 14.40 to 19.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 20.40 to 29.60 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 4.000 in
Between 30.40 to 36.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 36.80 to 44.00 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 44.80 to 49.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 50.40 to 56.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 6.000 in
Between 57.20 to 61.20 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 61.60 to 69.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 70.00 to 71.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 72.00 to 72.00 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 12.000 in
Between 72.60 to 79.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 80.40 to 90.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 91.20 to 95.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-437.09	646.99	0.68
Span # 2	2	24.000	-485.82	576.91	0.84

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Girder G2 (gridline D)

Load Combination Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 4	4	85.800	121.61	474.11	0.26

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*)	1	0.0426	10.800	D+L, LL Comb Run (*L*L)	-0.0155	20.400
D+L, LL Comb Run (*L*L)	2	0.2161	17.600	D+L, LL Comb Run (*L*L)	-0.0085	32.800
D+L, LL Comb Run (*L*L)	3	0.0089	17.200	D+L, LL Comb Run (*L*L)	-0.0315	7.200
D+L, LL Comb Run (*L*L)	4	0.0744	13.200		0.0000	7.200

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

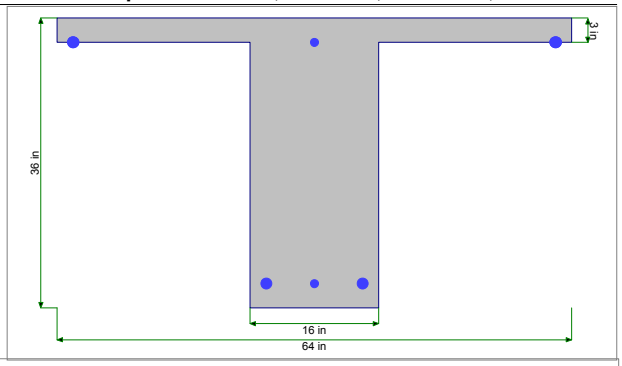
Description : Girder G3 (gridline E)

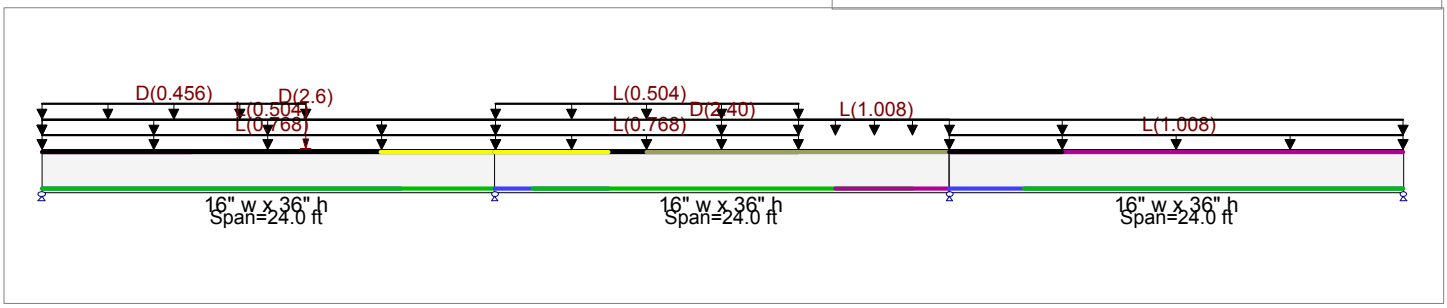
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c $^{1/2}$ * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup	=	2

Load Combination 2009 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 16.0 in, Total Height = 36.0 in, Top Flange Width = 64.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 1-#8 at 3.0 in from Bottom, from 0.0 to 19.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 3.0 in from Top, from 18.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span

Span #2 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 6.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 18.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 2.0 to 22.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Top, from 8.0 to 24.0 ft in this span

Span #3 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 6.0 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.10
Uniform Load on ALL spans : D = 0.10 ksf, Tributary Width = 24.0 ft

Load for Span Number 1
Uniform Load : L = 0.0640 ksf, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0420 ksf, Tributary Width = 12.0 ft, (office LL)
Uniform Load : D = 0.0380 ksf, Extent = 0.0 -->> 14.0 ft, Tributary Width = 12.0 ft, (addl slab DL)
Point Load : D = 2.60 k @ 14.0 ft

Load for Span Number 2
Uniform Load : L = 0.0640 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0420 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (office LL)
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 12.0 ft, (office LL)

Load for Span Number 3
Uniform Load : L = 0.0420 ksf, Tributary Width = 24.0 ft, (office LL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.997 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward L+Lr+S Deflection	0.043 in	Ratio = 6648
Mu : Applied		118.47 k-ft	Max Upward L+Lr+S Deflection	-0.041 in	Ratio = 6969
Mn * Phi : Allowable		118.80 k-ft	Max Downward Total Deflection	0.081 in	Ratio = 3572
Load Combination		1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*L*)	Max Upward Total Deflection	-0.017 in	Ratio = 16621
Location of maximum on span		12.200 ft			
Span # where maximum occurs		Span # 2			

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381

Licensee : AHEARN & KNOX, INC

Description : Girder G3 (gridline E)

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	40.666	103.500	92.137	34.360
D Only	27.966	68.266	62.342	23.210
L Only, LL Comb Run (**L)	0.403	-2.419	15.725	10.483
L Only, LL Comb Run (*L*)	-1.470	16.434	14.870	-1.418
L Only, LL Comb Run (*LL)	-1.067	14.015	30.595	9.065
L Only, LL Comb Run (L**)	13.229	19.843	-3.053	0.509
L Only, LL Comb Run (L*L)	13.632	17.424	12.672	10.992
L Only, LL Comb Run (LL*)	11.758	36.278	11.817	-0.909
L Only, LL Comb Run (LLL)	12.162	33.858	27.542	9.574
D+L, LL Comb Run (**L)	28.375	65.818	78.111	33.673
D+L, LL Comb Run (*L*)	26.550	84.579	77.292	21.778
D+L, LL Comb Run (*LL)	26.815	82.807	92.137	32.632
D+L, LL Comb Run (L**)	40.558	89.547	58.323	23.883
D+L, LL Comb Run (L*L)	40.666	87.714	73.764	34.360
D+L, LL Comb Run (LL*)	40.193	103.500	74.841	22.194
D+L, LL Comb Run (LLL)	40.274	101.926	89.845	32.876

Shear Stirrup Requirements

Between 0.00 to 5.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 5.60 to 13.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 14.00 to 20.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 20.20 to 25.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 9.000 in
Between 26.00 to 31.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 31.80 to 40.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 40.80 to 46.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 46.80 to 49.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 12.000 in
Between 50.00 to 56.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 56.80 to 67.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 67.60 to 71.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.800	-304.88	367.89	0.83
Span # 2	2	36.200	118.47	118.80	1.00
Span # 3	3	48.000	-277.39	367.89	0.75
+1.40D					
Span # 1	1	23.800	-205.60	367.89	0.56
Span # 2	2	47.800	-180.07	296.54	0.61
Span # 3	3	48.000	-187.83	367.89	0.51
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-160.88	367.89	0.44
Span # 2	2	47.800	-215.64	296.54	0.73
Span # 3	3	48.000	-222.93	367.89	0.61
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-232.22	367.89	0.63
Span # 2	2	36.200	118.47	118.80	1.00
Span # 3	3	48.000	-215.46	367.89	0.59
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-216.87	367.89	0.59
Span # 2	2	47.800	-265.82	296.54	0.90
Span # 3	3	48.000	-277.39	367.89	0.75
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	10.000	274.14	399.60	0.69
Span # 2	2	24.000	-263.59	367.89	0.72
Span # 3	3	48.000	-141.46	367.89	0.38
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	10.200	280.63	399.60	0.70
Span # 2	2	24.000	-248.11	367.89	0.67
Span # 3	3	61.800	229.76	399.60	0.57
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-304.88	367.89	0.83
Span # 2	2	24.000	-320.05	367.89	0.87
Span # 3	3	48.000	-195.92	367.89	0.53
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-289.52	367.89	0.79
Span # 2	2	47.800	-247.10	296.54	0.83

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Girder G3 (gridline E)

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 3	3	48.000	-257.85	367.89	0.70
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-160.88	367.89	0.44
Span # 2	2	47.800	-215.64	296.54	0.73
Span # 3	3	48.000	-222.93	367.89	0.61
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-232.22	367.89	0.63
Span # 2	2	36.200	118.47	118.80	1.00
Span # 3	3	48.000	-215.46	367.89	0.59
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-216.87	367.89	0.59
Span # 2	2	47.800	-265.82	296.54	0.90
Span # 3	3	48.000	-277.39	367.89	0.75
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	10.000	274.14	399.60	0.69
Span # 2	2	24.000	-263.59	367.89	0.72
Span # 3	3	48.000	-141.46	367.89	0.38
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	10.200	280.63	399.60	0.70
Span # 2	2	24.000	-248.11	367.89	0.67
Span # 3	3	61.800	229.76	399.60	0.57
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-304.88	367.89	0.83
Span # 2	2	24.000	-320.05	367.89	0.87
Span # 3	3	48.000	-195.92	367.89	0.53
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-289.52	367.89	0.79
Span # 2	2	47.800	-247.10	296.54	0.83
Span # 3	3	48.000	-257.85	367.89	0.70
+1.20D+1.60Lr+0.50L, LL Comb Run (*					
Span # 1	1	23.800	-171.43	367.89	0.47
Span # 2	2	47.800	-173.50	296.54	0.59
Span # 3	3	48.000	-180.35	367.89	0.49
+1.20D+1.60Lr+0.50L, LL Comb Run (*					
Span # 1	1	23.800	-193.73	367.89	0.53
Span # 2	2	47.800	-170.03	296.54	0.57
Span # 3	3	48.000	-178.02	367.89	0.48
+1.20D+1.60Lr+0.50L, LL Comb Run (*					
Span # 1	1	23.800	-188.93	367.89	0.51
Span # 2	2	47.800	-189.18	296.54	0.64
Span # 3	3	48.000	-197.37	367.89	0.54
+1.20D+1.60Lr+0.50L, LL Comb Run (L					
Span # 1	1	23.800	-198.93	367.89	0.54
Span # 2	2	24.000	-209.86	367.89	0.57
Span # 3	3	48.000	-154.89	367.89	0.42
+1.20D+1.60Lr+0.50L, LL Comb Run (L					
Span # 1	1	23.800	-194.14	367.89	0.53
Span # 2	2	47.800	-167.65	296.54	0.57
Span # 3	3	48.000	-174.25	367.89	0.47
+1.20D+1.60Lr+0.50L, LL Comb Run (L					
Span # 1	1	23.800	-216.43	367.89	0.59
Span # 2	2	24.000	-227.51	367.89	0.62
Span # 3	3	48.000	-171.91	367.89	0.47
+1.20D+1.60Lr+0.50L, LL Comb Run (L					
Span # 1	1	23.800	-211.63	367.89	0.58
Span # 2	2	47.800	-183.33	296.54	0.62
Span # 3	3	48.000	-191.27	367.89	0.52

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L)	1	0.0806	10.200	D+L, LL Comb Run (L*L)	-0.0029	24.600
D+L, LL Comb Run (L*L)	2	0.0027	24.600	D+L, LL Comb Run (L*L)	-0.0173	9.000
D+L, LL Comb Run (L*L)	3	0.0519	12.600		0.0000	9.000

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:
Project Notes :

Job # M11-040

Printed: 5 OCT 2011, 12:38PM

Concrete Slender Wall

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.30, Ver:6.11.8.1

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Existing 6" wall

General Information

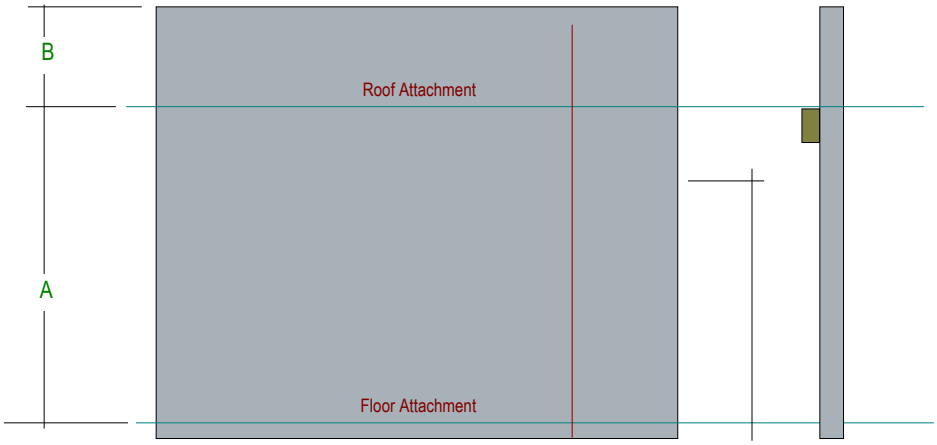
Calculations per ACI 318-08 Sec 14.8, IBC 2009, CBC 2010, ASCE 7-05

f'c : Concrete 28 day strength	=	3.50	ksi	Wall Thickness	6.0	in	Temp Diff across thickness = deg F
Fy : Rebar Yield	=	40.0	ksi	Rebar at wall center			Min Allow Out-of-Plane Defl Ratio = L / 150.0
Ec : Concrete Elastic Modulus	=	3,372.17	ksi	Rebar "d" distance	3.0	in	Minimum Vertical Steel % = 0.0020
λ : Lt Wt Conc Factor	=	1.0		Lower Level Rebar . . .			Using Stiffness Reduction Factor per ACI R.10.12.3
Fr : Rupture Modulus	=	295.80	psi	Bar Size	# 4		
Max % of ρ balanced	=	0.60		Bar Spacing	12.0	in	
Max Pu/Ag = f'c *	=	0.060					
Concrete Density	=	144.0	pcf				
Width of Design Strip	=	12.0	in				

One-Story Wall Dimensions

A Clear Height = 12.0 ft
B Parapet height = ft

Wall Support Condition: Top & Bottom Pinned



Vertical Loads

Vertical Uniform Loads . . . (Applied per foot of Strip Width)		DL : Dead Load	Lr : Roof Live Load	Lf : Floor Live Load	S : Snow Load	
Ledger Load	Eccentricity in	0.880	0.2560			k/ft
Concentric Load						k/ft

Lateral Loads

Full area WIND load 15.0 psf
Fp 1.0 = 34.0 psf

Wall Weight Seismic Load Input Method : Direct entry of Lateral Wall Weight
Seismic Wall Lateral Load 34.0 psf

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check	Maximum Bending Stress Ratio = **0.420**			
	+1.050D+1.280Lr+1.40E	Max Mu	0.8592 k-ft	Phi * Mn	2.046 k-ft
PASS	Service Deflection Check	Min. Defl. Ratio	9,242.33	Max Allow Ratio	150.0
	D + L + S + E/1.4	Max. Deflection	0.01558 in	Max. Allow. Defl.	0.960 in
PASS	Axial Load Check	Max Pu / Ag	26.631 psi	0.06 * f'c	210.0 psi
	+1.40D at 5.20 to 5.60				
PASS	Reinforcing Limit Check	Controlling As/bd	0.005556	As/bd = 0.50 rho bal	0.02598
	+1.40D				
FAIL	Minimum Moment Check	Mcracking	1.775 k-ft	Minimum Phi Mn	1.733 k-ft
	+1.40D	Maximum Reactions . . .	*for Load Combination....*		
		Top Horizontal	E Only		0.2040 k
		Base Horizontal	E Only		0.2040 k
		Vertical Reaction	D + L + Lr		2.0 k

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 5 OCT 2011, 12:38PM

Concrete Slender Wall

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.30, Ver:6.11.8.1

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Existing 6" wall

Design Maximum Combinations - Moments

Load Combination	Axial Load Pu k	0.06*f'c*b*t k	Moment Values Mcr k-ft	Mu k-ft	Phi	Phi Mn k-ft	As in^2	As Eff in^2	As Ratio	0.6 * rho bal
+1.40D at 5.20 to 5.60	1.917	15.120	1.77	0.00	0.88	2.08	0.200	0.248	0.0056	0.0260
+1.050D+1.280Lr+1.40E at 5.60 to 6.00	1.736	15.120	1.77	0.86	0.88	2.05	0.200	0.243	0.0056	0.0260
+0.90D+1.10E at 5.60 to 6.00	1.207	15.120	1.77	0.67	0.89	1.96	0.200	0.230	0.0056	0.0260

Design Maximum Combinations - Deflections

Load Combination	Axial Load Pu k	Moment Values Mcr k-ft	Mactual k-ft	Stiffness I gross in^4	I cracked in^4	I effective in^4	Deflections Deflection in	Defl. Ratio
D + L + Lr at 5.60 to 6.00	1.597	1.77	0.00	216.00	13.02	216.000	0.000	0.0
D + L + W at 5.60 to 6.00	1.341	1.77	0.27	216.00	12.73	216.000	0.010	14,963.8
D + L + W + S/2 at 5.60 to 6.00	1.341	1.77	0.27	216.00	12.73	216.000	0.010	14,963.8
D + L + S + W/2 at 5.60 to 6.00	1.341	1.77	0.14	216.00	12.73	216.000	0.005	29,927.6
D + L + S + E/1.4 at 5.60 to 6.00	1.341	1.77	0.44	216.00	12.73	216.000	0.016	9,242.3
D + 0.5(L+Lr) + 0.7W at 5.60 to 6.00	1.469	1.77	0.19	216.00	12.88	216.000	0.007	21,368.9
D + 0.5(L+Lr)+ 0.7E at 5.60 to 6.00	1.469	1.77	0.43	216.00	12.88	216.000	0.015	9,427.5

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	1.744 k
S Only	0.0 k	0.00 k	0.000 k
W Only	0.1 k	0.09 k	0.000 k
E Only	0.2 k	0.20 k	0.000 k
D + L + Lr	0.0 k	0.00 k	2.000 k
D + L + S	0.0 k	0.00 k	1.744 k
D + L + W + S/2	0.1 k	0.09 k	1.744 k
D + L + S + W/2	0.0 k	0.05 k	1.744 k
D + L + S + E/1.4	0.1 k	0.15 k	1.744 k

Program: PL-02 COLLECTOR STRESS ANALYSIS

Designation: Wall Line 1 & 5

Input Data:

Total Lateral Load:	104330	lbs.
Overall Collector Length:	121	ft.

Wall No.	Length (ft)	Begin (ft.)*	Wall No.	Length (ft)	Begin (ft.)*
1	7	1	6		
2	7	113	7		
3			8		
4			9		
5			10		

* Begin location is distance from left end of overall collector length to left end of shear wall.

Collector Load =	862.2	plf
Shear Wall Load =	7452.1	plf

Collector Loads:

Wall No.	Begin (lbs.)	End (lbs.)	Wall No.	Begin (lbs.)	End (lbs.)
1	862	-45267	6	0	0
2	45267	-862	7	0	0
3	0	0	8	0	0
4	0	0	9	0	0
5	0	0	10	0	0

Program: PL-02 COLLECTOR STRESS ANALYSIS

Designation: Wall Line A

Input Data:

Total Lateral Load:	104330	lbs.
Overall Collector Length:	97	ft.

Wall No.	Length (ft)	Begin (ft.)*	Wall No.	Length (ft)	Begin (ft.)*
1	8.5	25	6		
2	9.5	49	7		
3			8		
4			9		
5			10		

* Begin location is distance from left end of overall collector length to left end of shear wall.

Collector Load =	1075.6	plf
Shear Wall Load =	5796.1	plf

Collector Loads:

Wall No.	Begin (lbs.)	End (lbs.)	Wall No.	Begin (lbs.)	End (lbs.)
1	26889	-13235	6	0	0
2	3436	-41409	7	0	0
3	0	0	8	0	0
4	0	0	9	0	0
5	0	0	10	0	0

Program: PL-02 COLLECTOR STRESS ANALYSIS

Designation: Wall Line F

Input Data:

Total Lateral Load:	104330	lbs.
Overall Collector Length:	97	ft.

Wall No.	Length (ft)	Begin (ft.)*	Wall No.	Length (ft)	Begin (ft.)*
1	13	25	6		
2	7	89	7		
3			8		
4			9		
5			10		

* Begin location is distance from left end of overall collector length to left end of shear wall.

Collector Load =	1075.6	plf
Shear Wall Load =	5216.5	plf

Collector Loads:

Wall No.	Begin (lbs.)	End (lbs.)	Wall No.	Begin (lbs.)	End (lbs.)
1	26889	-26943	6	0	0
2	27911	-1076	7	0	0
3	0	0	8	0	0
4	0	0	9	0	0
5	0	0	10	0	0

Program: PC-03 CONCRETE SHEAR WALL

Designation: SW1.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	7	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	8	(3-10)
Number of Bars	2	
Right End Bar Size	8	(3-10)
Number of Bars	2	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	5.63		3.5	
2	6.3		3.5	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	52.16	12
2	1.3	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Double Reinforcing Curtain Required

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check:

ϕ = 0.85

V_u = 74.8 kips ϕV_n = 107.6 kips OK DCR = 0.70

Bending Check:

ϕ = 0.88

M_u left = 887.2 ft-kips ϕM_n = 364.1 ft-kips Overstressed! DCR = 2.44

M_u right = 887.2 ft-kips ϕM_n = 364.1 ft-kips Overstressed! DCR = 2.44

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	12.6	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.6	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.6	sq.in.	Supplied Number Of Bars =	2
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWA.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	8.5	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	9	(3-10)
Number of Bars	1	
Right End Bar Size	9	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	6.83		4.25	
2	7.7		4.25	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	47.12	12
2	1.6	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Single Reinforcing Curtain Allowed

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check: ϕ = 0.85

V_u =	68.2	kips	ϕV_n =	120.1	kips	OK	DCR =	0.57

Bending Check: ϕ = 0.88

M_u left =	805.1	ft-kips	ϕM_n =	285.4	ft-kips	Overstressed!	DCR =	2.82
M_u right =	805.1	ft-kips	ϕM_n =	285.4	ft-kips	Overstressed!	DCR =	2.82

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	15.3	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.7	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.0	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWA.2.2

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	9.5	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	10	(3-10)
Number of Bars	1	
Right End Bar Size	10	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	7.64		4.75	
2	8.6		4.75	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	57.21	12
2	1.8	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Double Reinforcing Curtain Required

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check:

ϕ = 0.85

V_u = 82.6 kips ϕV_n = 134.2 kips OK DCR = 0.62

Bending Check:

ϕ = 0.88

M_u left = 976.2 ft-kips ϕM_n = 406.2 ft-kips Overstressed! DCR = 2.40

M_u right = 976.2 ft-kips ϕM_n = 406.2 ft-kips Overstressed! DCR = 2.40

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	17.1	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.8	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.3	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWF.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	13	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	8	(3-10)
Number of Bars	2	
Right End Bar Size	8	(3-10)
Number of Bars	2	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	10.45		6.5	
2	11.7		6.5	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	77.32	12
2	2.5	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing IS Required			
Maximum Axial Load Check:	OK			

Shear Check:

ϕ = 0.85

V_u =	111.7	kips	ϕV_n =	183.7	kips	OK	DCR = 0.61

Bending Check:

ϕ = 0.88

M_u left =	1320.0	ft-kips	ϕM_n =	699.2	ft-kips	Overstressed!	DCR = 1.89
M_u right =	1320.0	ft-kips	ϕM_n =	699.2	ft-kips	Overstressed!	DCR = 1.89

Boundary Member Check:

Boundary Member Required: NO

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWA.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	8.5	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	9	(3-10)
Number of Bars	1	
Right End Bar Size	9	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	6.83		4.25	
2	7.7		4.25	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	47.12	12
2	1.6	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Single Reinforcing Curtain Allowed

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check: ϕ = 0.85

V_u =	68.2	kips	ϕV_n =	120.1	kips	OK	DCR =	0.57

Bending Check: ϕ = 0.88

M_u left =	805.1	ft-kips	ϕM_n =	285.4	ft-kips	Overstressed!	DCR =	2.82
M_u right =	805.1	ft-kips	ϕM_n =	285.4	ft-kips	Overstressed!	DCR =	2.82

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	15.3	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.7	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.0	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWA.2.2

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	9.5	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	10	(3-10)
Number of Bars	1	
Right End Bar Size	10	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	7.64		4.75	
2	8.6		4.75	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	57.21	12
2	1.8	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK

Double Curtain Reinforcing Check: Double Reinforcing Curtain Required

Hooked Shear Reinforcing Check: Hooked Shear Reinforcing IS Required

Maximum Axial Load Check: OK

Shear Check:

ϕ = 0.85

V_u = 82.6 kips ϕV_n = 134.2 kips OK DCR = 0.62

Bending Check:

ϕ = 0.88

M_u left = 976.2 ft-kips ϕM_n = 406.2 ft-kips Overstressed! DCR = 2.40

M_u right = 976.2 ft-kips ϕM_n = 406.2 ft-kips Overstressed! DCR = 2.40

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	17.1	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.8	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.3	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWF.2.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	13	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	8	(3-10)
Number of Bars	2	
Right End Bar Size	8	(3-10)
Number of Bars	2	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	10.45		6.5	
2	11.7		6.5	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	77.32	12
2	2.5	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing IS Required			
Maximum Axial Load Check:	OK			

Shear Check:

ϕ = 0.85

V_u =	111.7	kips	ϕV_n =	183.7	kips	OK	DCR = 0.61

Bending Check:

ϕ = 0.88

M_u left =	1320.0	ft-kips	ϕM_n =	699.2	ft-kips	Overstressed!	DCR = 1.89
M_u right =	1320.0	ft-kips	ϕM_n =	699.2	ft-kips	Overstressed!	DCR = 1.89

Boundary Member Check:

Boundary Member Required: NO

Program: PC-03 CONCRETE SHEAR WALL

Designation: SWF.2.2

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	7	ft.
Wall Thickness	6	in.
Unsupported Height	12	ft.
Horizontal Bars	4	(3-9)
Spacing	12	in.
Vertical Bars	4	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	9	(3-10)
Number of Bars	1	
Right End Bar Size	9	(3-10)
Number of Bars	1	
Distance From End	4	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	5.63		3.5	
2	6.3		3.5	
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	27.01	12
2	1.3	6
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0025	Actual:	0.0028	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK
Double Curtain Reinforcing Check:	Single Reinforcing Curtain Allowed			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing IS Required			
Maximum Axial Load Check:	OK			

Shear Check: ϕ = 0.85

V_u =	39.6	kips	ϕV_n =	107.6	kips	OK	DCR = 0.37

Bending Check: ϕ = 0.88

M_u left =	464.7	ft-kips	ϕM_n =	232.4	ft-kips	Overstressed!	DCR = 2.00
M_u right =	464.7	ft-kips	ϕM_n =	232.4	ft-kips	Overstressed!	DCR = 2.00

Boundary Member Check:

Boundary Member Required: YES

Minimum Boundary Member Length =	12.6	in.		
Minimum Boundary Member Thickness =	9.0	in.		
Minimum Boundary Member Steel =	0.6	sq.in.	Minimum Number Of Bars =	4
Actual Boundary Member Steel =	1.0	sq.in.	Supplied Number Of Bars =	1
Maximum Tie Spacing =	0.0	in.		

Length/Width Ratio Of Hoop Ties Shall Not Exceed 3

Cross Ties Or Hoops Shall Be Spaced 12 in. o.c. Maximum

Alternate Vertical Bars Shall Be Confined By Cross Tie Or Hoop Corner

Ties At Vertical Bar Splices Shall Be Spaced At 4 in. o.c. Maximum

Horizontal Wall Reinforcing Shall Be Hooked At Boundary Edge

Lap Splices Of Horizontal Reinforcing Not Allowed In Boundary Members

Program: PC-03 CONCRETE SHEAR WALL

Designation: SW1.1.1

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	49	ft.
Wall Thickness	12	in.
Unsupported Height	13	ft.
Horizontal Bars	5	(3-9)
Spacing	10	in.
Vertical Bars	6	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	6	(3-10)
Number of Bars	6	
Right End Bar Size	6	(3-10)
Number of Bars	6	
Distance From End	12	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	95.6		24.5	
2				
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	225.3	13
2	10.2	6.5
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0015	Actual:	0.0031	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0020	Actual:	0.0026	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	10	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing Not Required			
Maximum Axial Load Check:	OK			

Shear Check: ϕ = 0.85

V_u =	329.7	kips	ϕV_n =	1347.4	kips	OK	DCR = 0.24

Bending Check: ϕ = 0.89

M_u left =	4193.3	ft-kips	ϕM_n =	4503.2	ft-kips	OK	DCR = 0.93
M_u right =	4193.3	ft-kips	ϕM_n =	4503.2	ft-kips	OK	DCR = 0.93

Boundary Member Check:

Boundary Member Required: NO

Program: PC-03 CONCRETE SHEAR WALL

Designation: SW1.1.2

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	25	ft.
Wall Thickness	12	in.
Unsupported Height	13	ft.
Horizontal Bars	5	(3-9)
Spacing	10	in.
Vertical Bars	6	(3-9)
Spacing	12	in.
Single or Double Curtain	S	S/D
Left End Bar Size	6	(3-10)
Number of Bars	6	
Right End Bar Size	6	(3-10)
Number of Bars	6	
Distance From End	6	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	48.8		12.5	
2				
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	119.5	13
2	10.2	6.5
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0015	Actual:	0.0031	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0020	Actual:	0.0026	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	10	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing Not Required			
Maximum Axial Load Check:	OK			

Shear Check: ϕ = 0.85

V_u =	181.6	kips	ϕV_n =	687.4	kips	OK	DCR =	0.26

Bending Check: ϕ = 0.89

M_u left =	2267.7	ft-kips	ϕM_n =	2292.8	ft-kips	OK	DCR =	0.99
M_u right =	2267.7	ft-kips	ϕM_n =	2292.8	ft-kips	OK	DCR =	0.99

Boundary Member Check:

Boundary Member Required: NO

Program: PC-03 CONCRETE SHEAR WALL

Designation: SW5.1.5

1998 CBC (1985 UBC Sim.)

Input Data:

Wall Length	5.67	ft.
Wall Thickness	18	in.
Unsupported Height	5.75	ft.
Horizontal Bars	5	(3-9)
Spacing	12	in.
Vertical Bars	5	(3-9)
Spacing	12	in.
Single or Double Curtain	D	S/D
Left End Bar Size	6	(3-10)
Number of Bars	3	
Right End Bar Size	6	(3-10)
Number of Bars	3	
Distance From End	3	in.
Boundary Tie Bars		(3-6) if required.
Concrete Strength, f'c	3.5	ksi
Reinforcing Yield, Fy	40	ksi

Vertical Loads, kips: (Special = assembly, > 100 psf or garage)

Distance measured from left end of wall, ft.

Load No.	Dead, kips	Live, kips	Distance	Special (Y/N)
1	18.9		2.835	
2				
3				
4				
5				

Lateral Loads

Load No.	Force, kips	Height, ft.
1	27.4	5.75
2	4	2.875
3		
4		
5		
6		

Note: Program does not calculate wall self weight!

Code Minimum Checks:

Minimum Wall Vertical Reinforcing Ratio:	0.0012	Actual:	0.0029	OK
Minimum Wall Horizontal Reinforcing Ratio:	0.0020	Actual:	0.0029	OK
Maximum Vertical Reinforing Spacing:	18	Actual:	12	OK
Maximum Horizontal Reinforing Spacing:	18	Actual:	12	OK
Double Curtain Reinforcing Check:	Double Reinforcing Curtain Required			
Hooked Shear Reinforcing Check:	Hooked Shear Reinforcing Not Required			
Maximum Axial Load Check:	OK			

Shear Check:

ϕ = 0.85

V_u =	44.0	kips	ϕV_n =	243.4	kips	OK	DCR =	0.18

Bending Check:

ϕ = 0.89

M_u left =	236.7	ft-kips	ϕM_n =	252.6	ft-kips	OK	DCR =	0.94
M_u right =	236.7	ft-kips	ϕM_n =	252.6	ft-kips	OK	DCR =	0.94

Boundary Member Check:

Boundary Member Required: NO

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 Licensee : AHEARN & KNOX, INC

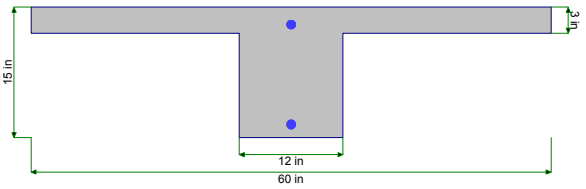
Description : Joist J-1 (betw. gl 2 & 3; assembly)

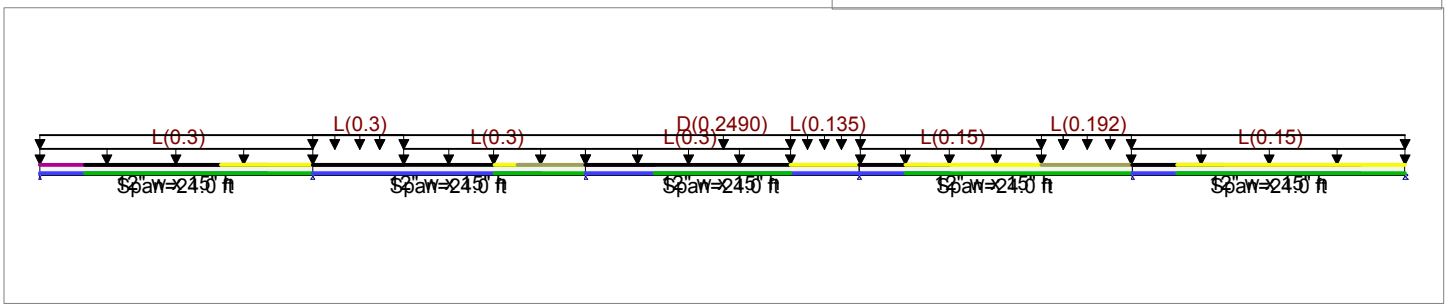
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c $^{1/2}$ * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup	=	2

Load Combination 2006 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 12.0 in, Total Height = 15.0 in, Top Flange Width = 60.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span

Span #2 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 16.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 to 24.0 ft in this span

Span #3 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 6.0 to 18.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 24.0 ft in this span

Span #4 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span

Span #5 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.0830
Uniform Load on ALL spans : D = 0.0830 ksf, Tributary Width = 3.0 ft

Load for Span Number 1
Uniform Load : L = 0.10 ksf, Tributary Width = 3.0 ft, (corridor LL)

Load for Span Number 2
Uniform Load : L = 0.10 ksf, Extent = 8.0 -->> 24.0 ft, Tributary Width = 3.0 ft, (assembly LL)
Uniform Load : L = 0.10 ksf, Extent = 0.0 -->> 8.0 ft, Tributary Width = 3.0 ft, (coridor)

Load for Span Number 3
Uniform Load : L = 0.10 ksf, Extent = 0.0 -->> 18.0 ft, Tributary Width = 3.0 ft, (assembly LL)
Uniform Load : L = 0.0450 ksf, Extent = 18.0 -->> 24.0 ft, Tributary Width = 3.0 ft, (office LL)

Load for Span Number 4
Uniform Load : L = 0.050 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 3.0 ft, (office LL)
Uniform Load : L = 0.0640 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 3.0 ft, (corridor LL)

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381

Licensee : AHEARN & KNOX, INC

Description : Joist J-1 (betw. gl 2 & 3; assembly)

Load for Span Number 5

Uniform Load : L = 0.050 ksf, Tributary Width = 3.0 ft, (office LL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.882** : 1

Section used for this span **Typical Section**

Mu : Applied -50.717 k-ft

Mn * Phi : Allowable 57.522 k-ft

Load Combination +1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (LL*L*)

Location of maximum on span 0.000 ft

Span # where maximum occurs Span # 2

Maximum Deflection

Max Downward L+Lr+S Deflection	0.111 in	Ratio =	2595
Max Upward L+Lr+S Deflection	-0.066 in	Ratio =	4357
Max Downward Total Deflection	0.154 in	Ratio =	1869
Max Upward Total Deflection	-0.041 in	Ratio =	6972

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5	Support 6
Overall MAXimum	5.442	15.052	13.998	11.326	11.528	4.012
D Only	2.359	6.762	5.819	5.819	6.762	2.359
L Only, LL Comb Run (****L)	0.004	-0.026	0.103	-0.388	2.347	1.559
L Only, LL Comb Run (***L*)	-0.014	0.081	-0.325	2.123	2.264	-0.194
L Only, LL Comb Run (***LL)	-0.009	0.056	-0.222	1.736	4.611	1.365
L Only, LL Comb Run (**L**)	0.090	-0.539	3.950	3.126	-0.501	0.083
L Only, LL Comb Run (**L*L)	0.094	-0.564	4.054	2.738	1.846	1.642
L Only, LL Comb Run (**LL*)	0.076	-0.457	3.625	5.249	1.763	-0.110
L Only, LL Comb Run (**LLL)	0.081	-0.483	3.728	4.861	4.110	1.449
L Only, LL Comb Run (*L***)	-0.353	3.919	4.125	-0.620	0.155	-0.026
L Only, LL Comb Run (*L**L)	-0.349	3.893	4.229	-1.008	2.502	1.533
L Only, LL Comb Run (*L*L*)	-0.367	4.000	3.800	1.503	2.419	-0.219
L Only, LL Comb Run (*L*LL)	-0.362	3.974	3.903	1.116	4.766	1.339
L Only, LL Comb Run (*LL**)	-0.263	3.380	8.076	2.505	-0.346	0.058
L Only, LL Comb Run (*LL*L)	-0.259	3.354	8.179	2.118	2.001	1.616
L Only, LL Comb Run (*LLL*)	-0.277	3.461	7.750	4.629	1.918	-0.136
L Only, LL Comb Run (*LLLL)	-0.273	3.436	7.854	4.241	4.265	1.423
L Only, LL Comb Run (L****)	3.118	4.694	-0.775	0.207	-0.052	0.009
L Only, LL Comb Run (L***L)	3.122	4.668	-0.672	-0.181	2.295	1.567
L Only, LL Comb Run (L**L*)	3.104	4.775	-1.101	2.330	2.212	-0.185
L Only, LL Comb Run (L**LL)	3.108	4.749	-0.997	1.943	4.559	1.374
L Only, LL Comb Run (L*L**)	3.207	4.155	3.175	3.332	-0.552	0.092
L Only, LL Comb Run (L*L*L)	3.212	4.129	3.279	2.945	1.795	1.651
L Only, LL Comb Run (L*LL*)	3.194	4.237	2.850	5.456	1.712	-0.102
L Only, LL Comb Run (L*LLL)	3.198	4.211	2.953	5.068	4.058	1.457
L Only, LL Comb Run (LL***)	2.765	8.612	3.350	-0.413	0.103	-0.017
L Only, LL Comb Run (LL**L)	2.769	8.587	3.454	-0.801	2.450	1.542
L Only, LL Comb Run (LL*L*)	2.751	8.694	3.025	1.710	2.367	-0.211
L Only, LL Comb Run (LL*LL)	2.755	8.668	3.128	1.322	4.714	1.348
L Only, LL Comb Run (LLL**)	2.854	8.074	7.301	2.712	-0.397	0.066
L Only, LL Comb Run (LLL*L)	2.859	8.048	7.404	2.325	1.950	1.625
L Only, LL Comb Run (LLLL*)	2.841	8.155	6.975	4.836	1.867	-0.127
L Only, LL Comb Run (LLLLL)	2.845	8.129	7.079	4.448	4.213	1.431
D+L, LL Comb Run (****L)	2.363	6.736	5.922	5.431	9.109	3.918
D+L, LL Comb Run (***L*)	2.345	6.844	5.493	7.942	9.026	2.165
D+L, LL Comb Run (***LL)	2.350	6.818	5.597	7.555	11.373	3.724
D+L, LL Comb Run (**L**)	2.449	6.224	9.769	8.944	6.262	2.442
D+L, LL Comb Run (**L*L)	2.453	6.198	9.872	8.557	8.609	4.001
D+L, LL Comb Run (**LL*)	2.435	6.305	9.444	11.068	8.526	2.249
D+L, LL Comb Run (**LLL)	2.439	6.279	9.547	10.680	10.872	3.808
D+L, LL Comb Run (*L***)	2.006	10.681	9.944	5.199	6.917	2.333
D+L, LL Comb Run (*L**L)	2.010	10.655	10.047	4.811	9.264	3.892
D+L, LL Comb Run (*L*L*)	1.992	10.762	9.619	7.322	9.181	2.140
D+L, LL Comb Run (*L*LL)	1.997	10.736	9.722	6.934	11.528	3.698
D+L, LL Comb Run (*LL**)	2.096	10.142	13.894	8.324	6.417	2.417
D+L, LL Comb Run (*LL*L)	2.100	10.117	13.998	7.937	8.764	3.975
D+L, LL Comb Run (*LLL*)	2.082	10.224	13.569	10.448	8.681	2.223
D+L, LL Comb Run (*LLLL)	2.086	10.198	13.672	10.060	11.027	3.782
D+L, LL Comb Run (L****)	5.390	11.653	4.904	6.063	6.701	2.369
D+L, LL Comb Run (L***L)	5.392	11.632	5.004	5.676	9.048	3.928
D+L, LL Comb Run (L**L*)	5.382	11.722	4.588	8.184	8.966	2.175
D+L, LL Comb Run (L**LL)	5.384	11.700	4.688	7.797	11.312	3.734

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Joist J-1 (betw. gl 2 & 3; assembly)

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5	Support 6
D+L, LL Comb Run (L*L**)	5.439	11.206	8.790	9.206	6.196	2.453
D+L, LL Comb Run (L*L*L)	5.442	11.185	8.890	8.819	8.543	4.012
D+L, LL Comb Run (L*LL*)	5.432	11.273	8.474	11.326	8.461	2.260
D+L, LL Comb Run (L*LLL)	5.434	11.252	8.574	10.940	10.808	3.818
D+L, LL Comb Run (LL***)	5.290	15.003	9.425	5.342	6.882	2.339
D+L, LL Comb Run (LL**L)	5.289	14.987	9.521	4.956	9.228	3.898
D+L, LL Comb Run (LL*L*)	5.290	15.052	9.122	7.460	9.147	2.145
D+L, LL Comb Run (LL*LL)	5.290	15.037	9.218	7.074	11.493	3.704
D+L, LL Comb Run (LLL**)	5.279	14.691	13.218	8.507	6.371	2.424
D+L, LL Comb Run (LLL*L)	5.278	14.676	13.314	8.122	8.717	3.983
D+L, LL Comb Run (LLLL*)	5.281	14.737	12.917	10.625	8.636	2.230
D+L, LL Comb Run (LLLLL)	5.280	14.723	13.012	10.239	10.983	3.789

Shear Stirrup Requirements

Between 0.00 to 19.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 20.40 to 27.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 27.60 to 45.00 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 45.60 to 50.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 51.00 to 70.20 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 70.80 to 71.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 72.00 to 95.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 96.00 to 96.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 97.20 to 119.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-43.98	57.52	0.76
Span # 2	2	24.000	-50.72	57.52	0.88
Span # 3	3	48.000	-42.92	57.52	0.75
Span # 4	4	72.000	-35.54	57.52	0.62
Span # 5	5	96.000	-35.80	57.52	0.62
+1.40D					
Span # 1	1	23.400	-18.16	57.52	0.32
Span # 2	2	24.000	-21.14	57.52	0.37
Span # 3	3	48.000	-15.85	57.52	0.28
Span # 4	4	95.400	-18.56	57.52	0.32
Span # 5	5	96.000	-21.14	57.52	0.37
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-15.40	57.52	0.27
Span # 2	2	24.000	-17.95	57.52	0.31
Span # 3	3	48.000	-14.25	57.52	0.25
Span # 4	4	95.400	-24.87	57.52	0.43
Span # 5	5	96.000	-27.38	57.52	0.48
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-16.07	57.52	0.28
Span # 2	2	24.000	-18.64	57.52	0.32
Span # 3	3	71.400	-19.05	57.52	0.33
Span # 4	4	84.000	16.39	35.90	0.46
Span # 5	5	96.000	-25.55	57.52	0.44
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-15.91	57.52	0.28
Span # 2	2	24.000	-18.47	57.52	0.32
Span # 3	3	71.400	-16.65	57.52	0.29
Span # 4	4	95.400	-30.37	57.52	0.53
Span # 5	5	96.000	-34.81	57.52	0.61
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.77	35.90	0.24
Span # 2	2	47.400	-24.96	57.52	0.43
Span # 3	3	48.000	-27.37	57.52	0.48
Span # 4	4	72.000	-26.40	57.52	0.46
Span # 5	5	96.000	-14.91	57.52	0.26
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.80	35.90	0.25
Span # 2	2	47.400	-25.60	57.52	0.45
Span # 3	3	48.000	-28.04	57.52	0.49
Span # 4	4	72.000	-23.92	57.52	0.42

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381
Licensee : AHEARN & KNOX, INC

Description : Joist J-1 (betw. gl 2 & 3; assembly)

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 2	2	24.000	-27.01	57.52	0.47
Span # 3	3	48.000	-21.20	57.52	0.37
Span # 4	4	95.400	-18.01	57.52	0.31
Span # 5	5	96.000	-20.22	57.52	0.35
+1.20D+0.50L+0.20S+E, LL Comb Run (					
Span # 1	1	23.400	-23.40	57.52	0.41
Span # 2	2	24.000	-27.23	57.52	0.47
Span # 3	3	48.000	-20.35	57.52	0.35
Span # 4	4	72.000	-19.21	57.52	0.33
Span # 5	5	96.000	-19.65	57.52	0.34
+1.20D+0.50L+0.20S+E, LL Comb Run (					
Span # 1	1	23.400	-23.35	57.52	0.41
Span # 2	2	24.000	-27.18	57.52	0.47
Span # 3	3	48.000	-20.55	57.52	0.36
Span # 4	4	95.400	-19.73	57.52	0.34
Span # 5	5	96.000	-22.54	57.52	0.39
+0.90D+1.60W+1.60H					
Span # 1	1	23.400	-11.67	57.52	0.20
Span # 2	2	24.000	-13.59	57.52	0.24
Span # 3	3	48.000	-10.19	57.52	0.18
Span # 4	4	95.400	-11.93	57.52	0.21
Span # 5	5	96.000	-13.59	57.52	0.24
+0.90D+E+1.60H					
Span # 1	1	23.400	-11.67	57.52	0.20
Span # 2	2	24.000	-13.59	57.52	0.24
Span # 3	3	48.000	-10.19	57.52	0.18
Span # 4	4	95.400	-11.93	57.52	0.21
Span # 5	5	96.000	-13.59	57.52	0.24

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*L)	1	0.1540	10.800	D+L, LL Comb Run (L*L*L)	-0.0109	25.200
D+L, LL Comb Run (*L*L*)	2	0.0641	13.200	D+L, LL Comb Run (L*L*L)	-0.0280	3.600
D+L, LL Comb Run (L*L*L)	3	0.0787	10.800	D+L, LL Comb Run (L*L*L)	-0.0060	25.200
D+L, LL Comb Run (*L*L*)	4	0.0427	10.800	D+L, LL Comb Run (L*L*L)	-0.0149	20.400
D+L, LL Comb Run (L*L*L)	5	0.0843	13.200		0.0000	20.400

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 Licensee : AHEARN & KNOX, INC

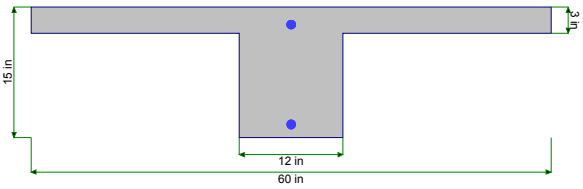
Description : Joist J-1 (betw. gl 1 & 2; corridor)

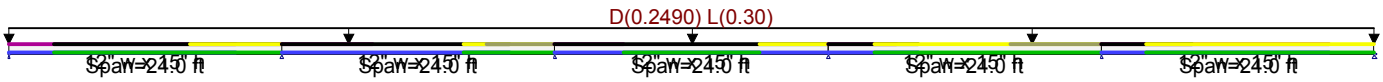
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c $^{1/2}$ * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup	=	2

Load Combination 2006 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 12.0 in, Total Height = 15.0 in, Top Flange Width = 60.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span

Span #2 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 16.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 to 24.0 ft in this span

Span #3 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 6.0 to 18.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 24.0 ft in this span

Span #4 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 2.0 in from Top, from 16.0 to 24.0 ft in this span

Span #5 Reinforcing....
- 1-#8 at 1.50 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 20.0 ft in this span
- 1-#8 at 2.0 in from Top, from 4.0 to 24.0 ft in this span
- 1-#8 at 1.50 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 8.0 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...

D = 0.0830, L = 0.10

Uniform Load on ALL spans : D = 0.0830, L = 0.10 ksf, Tributary Width = 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.890** : 1	Maximum Deflection	
Section used for this span	**Typical Section**	Max Downward L+Lr+S Deflection	0.113 in Ratio = 2549
Mu : Applied	-51.188 k-ft	Max Upward L+Lr+S Deflection	-0.067 in Ratio = 4301
Mn * Phi : Allowable	57.522 k-ft	Max Downward Total Deflection	0.156 in Ratio = 1846
Load Combination	1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*L*LL)	Max Upward Total Deflection	-0.044 in Ratio = 6573
Location of maximum on span	0.000 ft		
Span # where maximum occurs	Span # 5		

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381
Licensee : AHEARN & KNOX, INC

Description : Joist J-1 (betw. gl 1 & 2; corridor)

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5	Support 6
Overall MAXimum	5.448	15.098	14.275	14.275	15.107	5.448
D Only	2.359	6.762	5.819	5.819	6.762	2.359
L Only, LL Comb Run (****L)	0.009	-0.052	0.207	-0.775	4.694	3.118
L Only, LL Comb Run (***L*)	-0.026	0.155	-0.620	4.125	3.919	-0.353
L Only, LL Comb Run (***LL)	-0.017	0.103	-0.413	3.350	8.612	2.765
L Only, LL Comb Run (**L**)	0.095	-0.568	4.074	4.074	-0.568	0.095
L Only, LL Comb Run (**L*L)	0.103	-0.620	4.280	3.299	4.125	3.212
L Only, LL Comb Run (**LL*)	0.069	-0.413	3.454	8.199	3.350	-0.258
L Only, LL Comb Run (**LLL)	0.078	-0.465	3.660	7.424	8.044	2.859
L Only, LL Comb Run (*L***)	-0.353	3.919	4.125	-0.620	0.155	-0.026
L Only, LL Comb Run (*L**L)	-0.344	3.867	4.332	-1.395	4.849	3.092
L Only, LL Comb Run (*L*L*)	-0.379	4.074	3.505	3.505	4.074	-0.379
L Only, LL Comb Run (*L*LL)	-0.370	4.022	3.712	2.730	8.767	2.739
L Only, LL Comb Run (*LL**)	-0.258	3.350	8.199	3.454	-0.413	0.069
L Only, LL Comb Run (*LL*L)	-0.250	3.299	8.406	2.678	4.280	3.187
L Only, LL Comb Run (*LLL*)	-0.284	3.505	7.579	7.579	3.505	-0.284
L Only, LL Comb Run (*LLLL)	-0.276	3.454	7.786	6.804	8.199	2.833
L Only, LL Comb Run (L****)	3.118	4.694	-0.775	0.207	-0.052	0.009
L Only, LL Comb Run (L***L)	3.126	4.642	-0.568	-0.568	4.642	3.126
L Only, LL Comb Run (L**L*)	3.092	4.849	-1.395	4.332	3.867	-0.344
L Only, LL Comb Run (L**LL)	3.100	4.797	-1.189	3.557	8.561	2.773
L Only, LL Comb Run (L*L**)	3.212	4.125	3.299	4.280	-0.620	0.103
L Only, LL Comb Run (L*L*L)	3.221	4.074	3.505	3.505	4.074	3.221
L Only, LL Comb Run (L*LL*)	3.187	4.280	2.678	8.406	3.299	-0.250
L Only, LL Comb Run (L*LLL)	3.195	4.229	2.885	7.631	7.992	2.868
L Only, LL Comb Run (LL***)	2.765	8.612	3.350	-0.413	0.103	-0.017
L Only, LL Comb Run (LL**L)	2.773	8.561	3.557	-1.189	4.797	3.100
L Only, LL Comb Run (LL*L*)	2.739	8.767	2.730	3.712	4.022	-0.370
L Only, LL Comb Run (LL*LL)	2.747	8.716	2.937	2.937	8.716	2.747
L Only, LL Comb Run (LLL**)	2.859	8.044	7.424	3.660	-0.465	0.078
L Only, LL Comb Run (LLL*L)	2.868	7.992	7.631	2.885	4.229	3.195
L Only, LL Comb Run (LLLL*)	2.833	8.199	6.804	7.786	3.454	-0.276
L Only, LL Comb Run (LLLLL)	2.842	8.147	7.011	7.011	8.147	2.842
D+L, LL Comb Run (****L)	2.369	6.701	6.063	4.904	11.653	5.390
D+L, LL Comb Run (***L*)	2.333	6.917	5.199	9.944	10.681	2.006
D+L, LL Comb Run (***LL)	2.339	6.881	5.343	9.419	15.013	5.285
D+L, LL Comb Run (**L**)	2.454	6.194	9.892	9.892	6.194	2.454
D+L, LL Comb Run (**L*L)	2.465	6.128	10.155	8.909	11.181	5.442
D+L, LL Comb Run (**LL*)	2.428	6.349	9.272	14.018	10.113	2.101
D+L, LL Comb Run (**LLL)	2.436	6.302	9.459	13.325	14.684	5.273
D+L, LL Comb Run (*L***)	2.006	10.681	9.944	5.199	6.917	2.333
D+L, LL Comb Run (*L**L)	2.016	10.621	10.183	4.301	11.784	5.375
D+L, LL Comb Run (*L*L*)	1.980	10.836	9.324	9.324	10.836	1.980
D+L, LL Comb Run (*L*LL)	1.986	10.803	9.457	8.841	15.107	5.286
D+L, LL Comb Run (*LL**)	2.101	10.113	14.018	9.272	6.349	2.428
D+L, LL Comb Run (*LL*L)	2.111	10.048	14.275	8.308	11.309	5.428
D+L, LL Comb Run (*LLL*)	2.075	10.268	13.398	13.398	10.268	2.075
D+L, LL Comb Run (*LLLL)	2.082	10.224	13.573	12.751	14.774	5.276
D+L, LL Comb Run (L****)	5.390	11.653	4.904	6.063	6.701	2.369
D+L, LL Comb Run (L***L)	5.396	11.601	5.143	5.143	11.601	5.396
D+L, LL Comb Run (L**L*)	5.375	11.784	4.301	10.183	10.621	2.016
D+L, LL Comb Run (L**LL)	5.377	11.755	4.443	9.645	14.974	5.285
D+L, LL Comb Run (L*L**)	5.442	11.181	8.909	10.155	6.128	2.465
D+L, LL Comb Run (L*L*L)	5.448	11.125	9.166	9.166	11.125	5.448
D+L, LL Comb Run (L*LL*)	5.428	11.309	8.308	14.275	10.048	2.111
D+L, LL Comb Run (L*LLL)	5.432	11.270	8.493	13.569	14.642	5.274
D+L, LL Comb Run (LL***)	5.290	15.003	9.425	5.342	6.882	2.339
D+L, LL Comb Run (LL**L)	5.290	14.964	9.651	4.442	11.755	5.377
D+L, LL Comb Run (LL*L*)	5.290	15.098	8.846	9.457	10.803	1.986
D+L, LL Comb Run (LL*LL)	5.288	15.082	8.970	8.964	15.092	5.283
D+L, LL Comb Run (LLL**)	5.278	14.673	13.333	9.458	6.303	2.436
D+L, LL Comb Run (LLL*L)	5.279	14.631	13.576	8.491	11.271	5.432
D+L, LL Comb Run (LLLL*)	5.282	14.763	12.758	13.571	10.224	2.082
D+L, LL Comb Run (LLLLL)	5.279	14.740	12.922	12.914	14.751	5.274

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Joist J-1 (betw. gl 1 & 2; corridor)

Shear Stirrup Requirements

Between 0.00 to 19.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 20.40 to 27.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 27.60 to 45.00 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 45.60 to 51.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 51.60 to 68.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 69.00 to 74.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 75.00 to 92.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 93.00 to 99.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.4.5.1, use stirrups spaced at 6.000 in
Between 100.20 to 119.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-44.44	57.52	0.77
Span # 2	2	24.000	-51.19	57.52	0.89
Span # 3	3	48.000	-44.34	57.52	0.77
Span # 4	4	95.400	-44.93	57.52	0.78
Span # 5	5	96.000	-51.19	57.52	0.89
+1.40D					
Span # 1	1	23.400	-18.16	57.52	0.32
Span # 2	2	24.000	-21.14	57.52	0.37
Span # 3	3	48.000	-15.85	57.52	0.28
Span # 4	4	95.400	-18.56	57.52	0.32
Span # 5	5	96.000	-21.14	57.52	0.37
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-15.24	57.52	0.27
Span # 2	2	24.000	-17.79	57.52	0.31
Span # 3	3	48.000	-14.91	57.52	0.26
Span # 4	4	95.400	-33.84	57.52	0.59
Span # 5	5	##.###	23.10	35.90	0.64
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-16.53	57.52	0.29
Span # 2	2	24.000	-19.11	57.52	0.33
Span # 3	3	71.400	-25.90	57.52	0.45
Span # 4	4	84.000	26.00	35.90	0.72
Span # 5	5	96.000	-31.68	57.52	0.55
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-16.21	57.52	0.28
Span # 2	2	24.000	-18.78	57.52	0.33
Span # 3	3	71.400	-21.10	57.52	0.37
Span # 4	4	95.400	-44.06	57.52	0.77
Span # 5	5	96.000	-50.20	57.52	0.87
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.80	35.90	0.25
Span # 2	2	47.400	-25.70	57.52	0.45
Span # 3	3	48.000	-28.14	57.52	0.49
Span # 4	4	72.000	-28.14	57.52	0.49
Span # 5	5	96.000	-14.48	57.52	0.25
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.85	35.90	0.25
Span # 2	2	47.400	-26.98	57.52	0.47
Span # 3	3	48.000	-29.46	57.52	0.51
Span # 4	4	95.400	-30.66	57.52	0.53
Span # 5	5	##.###	23.65	35.90	0.66
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.65	35.90	0.24
Span # 2	2	47.400	-21.86	57.52	0.38
Span # 3	3	71.400	-37.08	57.52	0.64
Span # 4	4	72.000	-43.02	57.52	0.75
Span # 5	5	96.000	-28.04	57.52	0.49
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	3.600	8.70	35.90	0.24
Span # 2	2	47.400	-23.14	57.52	0.40
Span # 3	3	71.400	-32.28	57.52	0.56
Span # 4	4	95.400	-40.88	57.52	0.71
Span # 5	5	96.000	-46.56	57.52	0.81
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.400	-28.79	57.52	0.50
Span # 2	2	36.000	26.00	35.90	0.72

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Joist J-1 (betw. gl 1 & 2; corridor)

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
+0.90D+1.60W+1.60H					
Span # 1	1	23.400	-11.67	57.52	0.20
Span # 2	2	24.000	-13.59	57.52	0.24
Span # 3	3	48.000	-10.19	57.52	0.18
Span # 4	4	95.400	-11.93	57.52	0.21
Span # 5	5	96.000	-13.59	57.52	0.24
+0.90D+E+1.60H					
Span # 1	1	23.400	-11.67	57.52	0.20
Span # 2	2	24.000	-13.59	57.52	0.24
Span # 3	3	48.000	-10.19	57.52	0.18
Span # 4	4	95.400	-11.93	57.52	0.21
Span # 5	5	96.000	-13.59	57.52	0.24

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*L)	1	0.1560	10.800	D+L, LL Comb Run (L*L*L)	-0.0112	25.200
D+L, LL Comb Run (*L*L*)	2	0.0667	13.200	D+L, LL Comb Run (L*L*L)	-0.0289	3.600
D+L, LL Comb Run (L*L*L)	3	0.0869	13.200		0.0000	3.600
D+L, LL Comb Run (*L*L*)	4	0.0667	10.800	D+L, LL Comb Run (L*L*L)	-0.0289	20.400
D+L, LL Comb Run (L*L*L)	5	0.1560	13.200		0.0000	20.400

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

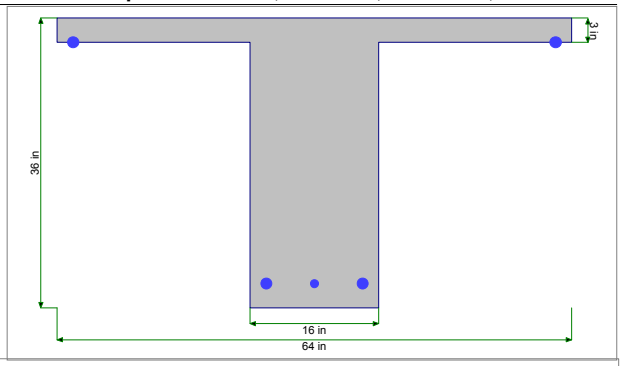
Description : Girder G1 (gridline B)

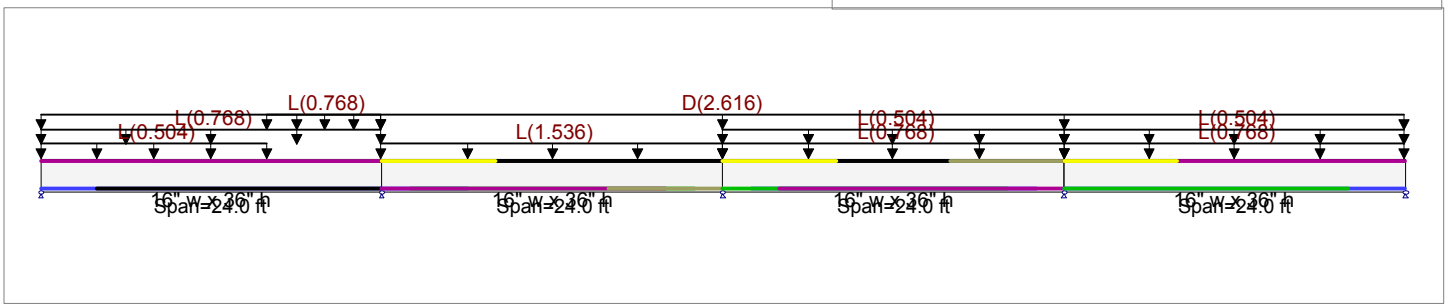
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	=	443.71 psi		Shear :	0.750
Ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup	=	2

Load Combination 2009 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 16.0 in, Total Height = 36.0 in, Top Flange Width = 64.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 8.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 4.0 to 24.0 ft in this span

Span #2 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 6.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 0.0 to 20.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 8.0 ft in this span
- 2-#8 at 3.0 in from Bottom, from 2.0 to 22.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#8 at 3.0 in from Bottom, from 16.0 to 24.0 ft in this span

Span #3 Reinforcing....
- 2-#8 at 3.0 in from Bottom, from 2.0 to 22.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 0.0 to 20.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#11 at 3.0 in from Top, from 16.0 to 24.0 ft in this span

Span #4 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 0.0 to 20.0 ft in this span
- 1-#11 at 3.0 in from Top, from 0.0 to 8.0 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.1090
Uniform Load on ALL spans : D = 0.1090 ksf, Tributary Width = 24.0 ft

Load for Span Number 1
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 12.0 ft, (Multiuse LL)
Uniform Load : L = 0.0640 ksf, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0640 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 12.0 ft, (corridor LL)

Load for Span Number 2
Uniform Load : L = 0.0640 ksf, Tributary Width = 24.0 ft, (corridor LL)

Load for Span Number 3
Uniform Load : L = 0.0640 ksf, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0420 ksf, Tributary Width = 12.0 ft, (Office LL)

Load for Span Number 4
Uniform Load : L = 0.0640 ksf, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0420 ksf, Tributary Width = 12.0 ft, (office LL)

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Girder G1 (gridline B)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.542** : 1
Section used for this span **Typical Section**
Mu : Applied -352.01 k-ft
Mn * Phi : Allowable 649.56 k-ft
Load Combination 1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (LL*L)
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 2

Maximum Deflection

Max Downward L+Lr+S Deflection	0.032 in	Ratio =	8907
Max Upward L+Lr+S Deflection	-0.037 in	Ratio =	7683
Max Downward Total Deflection	0.063 in	Ratio =	4605
Max Upward Total Deflection	-0.009 in	Ratio =	32410

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum	38.150	111.577	96.809	106.834	37.978
D Only	24.665	71.753	58.299	71.753	24.665
L Only, LL Comb Run (***L)	-0.136	0.818	-3.271	19.898	13.220
L Only, LL Comb Run (**L*)	0.409	-2.453	17.445	16.627	-1.499
L Only, LL Comb Run (**LL)	0.273	-1.635	14.174	36.525	11.721
L Only, LL Comb Run (*L**)	-1.810	20.078	21.065	-2.962	0.494
L Only, LL Comb Run (*L*L)	-1.947	20.895	17.794	16.935	13.713
L Only, LL Comb Run (*LL*)	-1.401	17.625	38.510	13.665	-1.005
L Only, LL Comb Run (*LLL)	-1.538	18.442	35.239	33.562	12.214
L Only, LL Comb Run (L***)	13.441	21.955	-3.480	0.870	-0.145
L Only, LL Comb Run (L**L)	13.304	22.772	-6.751	20.768	13.075
L Only, LL Comb Run (L*L*)	13.850	19.501	13.964	17.497	-1.644
L Only, LL Comb Run (L*LL)	13.713	20.319	10.693	37.395	11.576
L Only, LL Comb Run (LL**)	11.630	42.032	17.585	-2.092	0.349
L Only, LL Comb Run (LL*L)	11.494	42.850	14.314	17.806	13.568
L Only, LL Comb Run (LLL*)	12.039	39.579	35.029	14.535	-1.150
L Only, LL Comb Run (LLLL)	11.903	40.397	31.758	34.432	12.069
D+L, LL Comb Run (***L)	24.517	72.639	54.783	91.982	37.742
D+L, LL Comb Run (**L*)	25.074	69.300	75.744	88.380	23.166
D+L, LL Comb Run (**LL)	24.995	69.776	73.933	106.155	37.333
D+L, LL Comb Run (*L**)	22.855	91.831	79.365	68.791	25.159
D+L, LL Comb Run (*L*L)	22.712	92.768	75.451	89.640	37.956
D+L, LL Comb Run (*LL*)	23.264	89.378	96.809	85.418	23.660
D+L, LL Comb Run (*LLL)	23.153	90.040	94.223	104.307	37.332
D+L, LL Comb Run (L***)	38.005	93.947	54.631	72.681	24.510
D+L, LL Comb Run (L**L)	37.947	94.617	51.366	92.701	37.673
D+L, LL Comb Run (L*L*)	38.122	92.149	71.626	89.413	22.994
D+L, LL Comb Run (L*LL)	38.150	92.351	70.188	106.834	37.308
D+L, LL Comb Run (LL**)	37.585	110.898	77.866	69.199	25.091
D+L, LL Comb Run (LL*L)	37.550	111.577	74.239	89.824	37.978
D+L, LL Comb Run (LLL*)	37.593	109.348	94.685	85.976	23.567
D+L, LL Comb Run (LLLL)	37.622	109.663	92.523	104.501	37.387

Shear Stirrup Requirements

Between 0.00 to 4.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 5.40 to 13.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 14.40 to 20.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 21.00 to 25.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 10.000 in
Between 26.40 to 32.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 33.00 to 40.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 41.40 to 45.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 46.20 to 49.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 12.000 in
Between 50.40 to 54.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 55.20 to 64.20 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 64.80 to 69.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 69.60 to 74.40 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 10.000 in
Between 75.00 to 81.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 82.20 to 90.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 91.20 to 95.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-305.27	649.56	0.47
Span # 2	2	24.000	-352.01	649.56	0.54

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Girder G1 (gridline B)

Load Combination Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 4	4	72.000	-145.30	751.89	0.19

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*)	1	0.0625	10.800	D+L, LL Comb Run (L*L*)	-0.0033	25.200
D+L, LL Comb Run (*L*L)	2	0.0291	13.200		0.0000	25.200
D+L, LL Comb Run (L*L*)	3	0.0261	10.800	D+L, LL Comb Run (*L*L)	-0.0078	20.400
D+L, LL Comb Run (*L*L)	4	0.0608	13.200		0.0000	20.400

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 Licensee : AHEARN & KNOX, INC

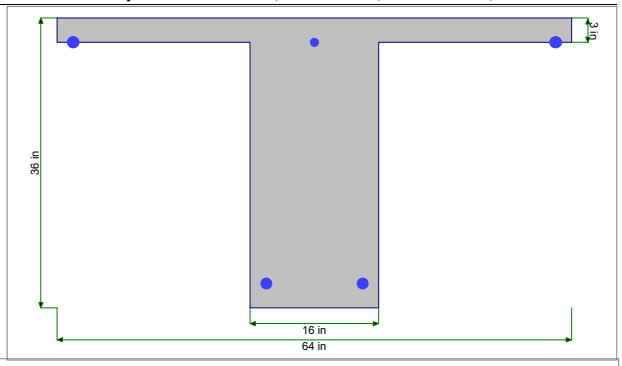
Description : Girder G2 (gridline C; fixed seating)

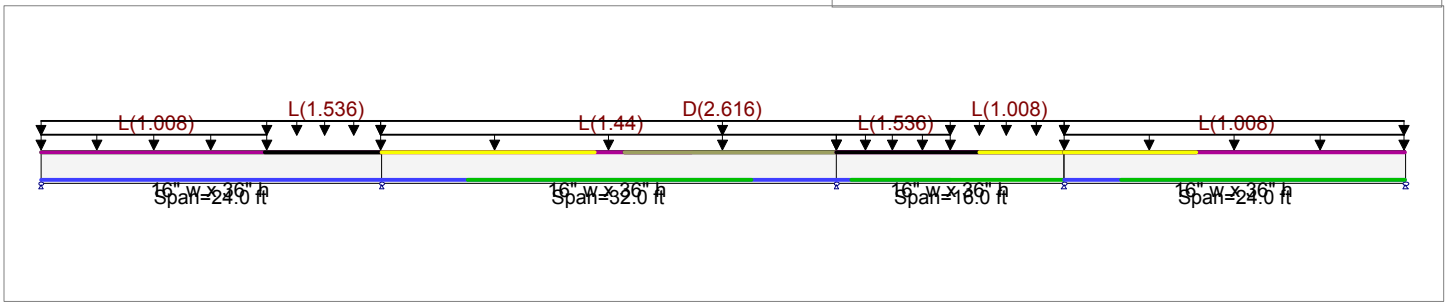
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size # =		# 3
			Number of Resisting Legs Per Stirrup =		2

Load Combination 2009 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 16.0 in, Total Height = 36.0 in, Top Flange Width = 64.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 15.750 to 24.0 ft in this span

Span #2 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 21.750 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 15.0 ft in this span
- 2-#8 at 3.0 in from Bottom, from 6.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 17.10 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 17.10 to 24.0 ft in this span

Span #3 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 8.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Top, from 10.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 1.0 to 16.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 16.0 ft in this span

Span #4 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 9.250 ft in this span
- 2-#8 at 3.0 in from Bottom, from 4.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 9.250 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.1090
Uniform Load on ALL spans : D = 0.1090 ksf, Tributary Width = 24.0 ft

Load for Span Number 1
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 24.0 ft, (Office LL)
Uniform Load : L = 0.0640 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (Corridor LL)

Load for Span Number 2
Uniform Load : L = 0.060 ksf, Tributary Width = 24.0 ft, (Assembly LL)

Load for Span Number 3
Uniform Load : L = 0.0640 ksf, Extent = 0.0 -->> 8.0 ft, Tributary Width = 24.0 ft, (Corridor LL)
Uniform Load : L = 0.0420 ksf, Extent = 8.0 -->> 16.0 ft, Tributary Width = 24.0 ft, (Office LL)

Load for Span Number 4
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (Office LL)

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:51PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Girder G2 (gridline C; fixed seating)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.841** : 1
Section used for this span **Typical Section**
Mu : Applied -369.18 k-ft
Mn * Phi : Allowable 439.06 k-ft
Load Combination 1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*LL*)
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 3

Maximum Deflection

Max Downward L+Lr+S Deflection	0.120 in	Ratio =	3204
Max Upward L+Lr+S Deflection	-0.073 in	Ratio =	5289
Max Downward Total Deflection	0.181 in	Ratio =	2118
Max Upward Total Deflection	-0.028 in	Ratio =	6791

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum	34.529	124.971	108.545	82.567	37.310
D Only	22.211	84.361	63.586	54.369	26.609
L Only, LL Comb Run (***L)	-0.099	0.434	-3.681	17.326	10.212
L Only, LL Comb Run (**L*)	0.154	-0.674	11.731	9.694	-0.553
L Only, LL Comb Run (**LL)	0.055	-0.240	8.050	27.020	9.659
L Only, LL Comb Run (*L**)	-3.190	25.474	31.350	-8.393	0.839
L Only, LL Comb Run (*L*L)	-3.289	25.908	27.669	8.933	11.052
L Only, LL Comb Run (*LL*)	-3.035	24.800	43.081	1.300	0.286
L Only, LL Comb Run (*LLL)	-3.135	25.234	39.400	18.626	10.499
L Only, LL Comb Run (L***)	11.130	18.971	-2.721	1.152	-0.115
L Only, LL Comb Run (L**L)	11.031	19.404	-6.402	18.478	10.097
L Only, LL Comb Run (L*L*)	11.284	18.297	9.010	10.845	-0.668
L Only, LL Comb Run (L*LL)	11.185	18.730	5.329	28.171	9.544
L Only, LL Comb Run (LL**)	7.940	44.445	28.628	-7.242	0.724
L Only, LL Comb Run (LL*L)	7.841	44.878	24.948	10.084	10.936
L Only, LL Comb Run (LLL*)	8.094	43.771	40.360	2.452	0.171
L Only, LL Comb Run (LLLL)	7.995	44.204	36.679	19.778	10.383
D+L, LL Comb Run (***L)	22.159	84.759	59.111	72.937	36.362
D+L, LL Comb Run (**L*)	22.294	83.829	75.204	64.110	26.051
D+L, LL Comb Run (**LL)	22.213	84.260	71.035	82.183	35.989
D+L, LL Comb Run (*L**)	19.060	108.859	98.305	43.131	27.860
D+L, LL Comb Run (*L*L)	18.992	109.053	94.061	61.991	37.310
D+L, LL Comb Run (*LL*)	19.140	108.723	108.545	53.993	27.166
D+L, LL Comb Run (*LLL)	19.095	108.770	105.038	71.975	36.883
D+L, LL Comb Run (L***)	34.487	101.063	62.618	54.820	26.564
D+L, LL Comb Run (L**L)	34.441	101.444	58.234	73.263	36.362
D+L, LL Comb Run (L*L*)	34.529	100.611	74.175	64.584	26.004
D+L, LL Comb Run (L*LL)	34.491	100.956	70.115	82.567	35.968
D+L, LL Comb Run (LL**)	31.840	124.675	97.656	43.696	27.765
D+L, LL Comb Run (LL*L)	31.741	124.971	93.294	62.569	37.249
D+L, LL Comb Run (LLL*)	31.974	124.310	108.401	54.186	27.113
D+L, LL Comb Run (LLLL)	31.909	124.446	104.758	72.205	36.858

Shear Stirrup Requirements

Between 0.00 to 3.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 4.20 to 13.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 14.40 to 19.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 20.40 to 28.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 5.000 in
Between 29.60 to 36.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 36.80 to 44.00 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 44.80 to 50.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 51.20 to 56.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 8.000 in
Between 57.20 to 61.20 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 61.60 to 68.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 69.20 to 71.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 72.00 to 72.60 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 12.000 in
Between 73.20 to 79.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 80.40 to 90.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 91.20 to 95.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-415.20	646.99	0.64
Span # 2	2	24.000	-463.37	576.91	0.80

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Girder G2 (gridline D)

Load Combination Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 4	4	85.800	121.61	474.11	0.26

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*)	1	0.0426	10.800	D+L, LL Comb Run (*L*L)	-0.0155	20.400
D+L, LL Comb Run (*L*L)	2	0.2161	17.600	D+L, LL Comb Run (*L*L)	-0.0085	32.800
D+L, LL Comb Run (*L*L)	3	0.0089	17.200	D+L, LL Comb Run (*L*L)	-0.0315	7.200
D+L, LL Comb Run (*L*L)	4	0.0744	13.200		0.0000	7.200

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 Licensee : AHEARN & KNOX, INC

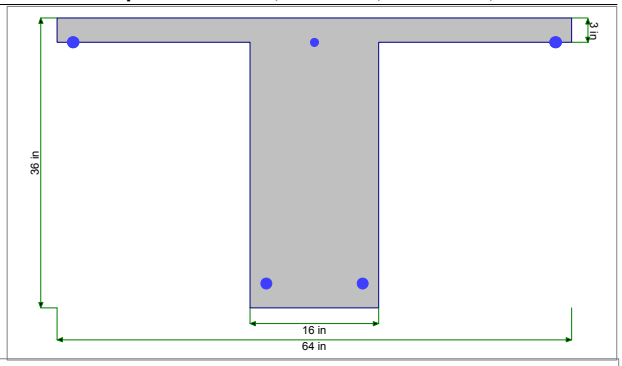
Description : Girder G2 (gridline D)

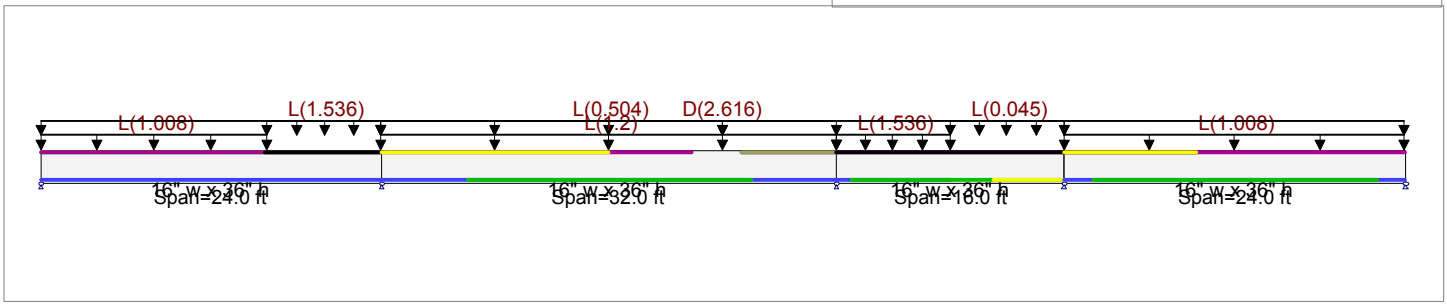
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size # =		# 3
			Number of Resisting Legs Per Stirrup =		2

Load Combination 2009 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 16.0 in, Total Height = 36.0 in, Top Flange Width = 64.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 15.750 to 24.0 ft in this span

Span #2 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 21.750 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 2-#8 at 3.0 in from Bottom, from 6.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 25.250 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 25.250 to 24.0 ft in this span

Span #3 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 8.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 11.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 1.0 to 16.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 16.0 ft in this span

Span #4 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 2-#8 at 3.0 in from Top, from 0.0 to 9.250 ft in this span
- 2-#8 at 3.0 in from Bottom, from 2.0 to 22.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 9.250 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.1090
Uniform Load on ALL spans : D = 0.1090 ksf, Tributary Width = 24.0 ft

Load for Span Number 1
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 24.0 ft, (Office LL)
Uniform Load : L = 0.0640 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (Corridor LL)

Load for Span Number 2
Uniform Load : L = 0.10 ksf, Tributary Width = 12.0 ft, (Assembly LL)
Uniform Load : L = 0.0420 ksf, Tributary Width = 12.0 ft, (office LL)

Load for Span Number 3
Uniform Load : L = 0.0640 ksf, Extent = 0.0 -->> 8.0 ft, Tributary Width = 24.0 ft, (Corridor LL)
Uniform Load : L = 0.0450 k/ft, Extent = 8.0 -->> 16.0 ft, Tributary Width = 1.0 ft, (Office LL)

Load for Span Number 4
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (Office LL)

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

Description : Girder G2 (gridline D)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.894** : 1
Section used for this span **Typical Section**
Mu : Applied -392.60 k-ft
Mn * Phi : Allowable 439.06 k-ft
Load Combination 1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*LL*)
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 3

Maximum Deflection

Max Downward L+Lr+S Deflection	0.155 in	Ratio =	2481
Max Upward L+Lr+S Deflection	-0.072 in	Ratio =	5354
Max Downward Total Deflection	0.216 in	Ratio =	1776
Max Upward Total Deflection	-0.032 in	Ratio =	6086

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5
Overall MAXimum	34.517	129.909	112.925	76.522	37.528
D Only	22.211	84.361	63.586	54.369	26.609
L Only, LL Comb Run (***L)	-0.099	0.434	-3.681	17.326	10.212
L Only, LL Comb Run (**L*)	0.108	-0.474	9.797	3.513	-0.296
L Only, LL Comb Run (**LL)	0.009	-0.041	6.117	20.839	9.916
L Only, LL Comb Run (*L**)	-3.774	30.144	37.097	-9.932	0.993
L Only, LL Comb Run (*L*L)	-3.873	30.578	33.416	7.394	11.205
L Only, LL Comb Run (*LL*)	-3.666	29.670	46.894	-6.420	0.697
L Only, LL Comb Run (*LLL)	-3.765	30.104	43.213	10.906	10.909
L Only, LL Comb Run (L***)	11.130	18.971	-2.721	1.152	-0.115
L Only, LL Comb Run (L**L)	11.031	19.404	-6.402	18.478	10.097
L Only, LL Comb Run (L*L*)	11.238	18.496	7.076	4.664	-0.411
L Only, LL Comb Run (L*LL)	11.139	18.930	3.395	21.990	9.801
L Only, LL Comb Run (LL**)	7.356	49.115	34.376	-8.780	0.878
L Only, LL Comb Run (LL*L)	7.257	49.549	30.695	8.546	11.090
L Only, LL Comb Run (LLL*)	7.464	48.640	44.173	-5.268	0.582
L Only, LL Comb Run (LLLL)	7.365	49.074	40.492	12.058	10.794
D+L, LL Comb Run (***L)	22.159	84.759	59.111	72.937	36.362
D+L, LL Comb Run (**L*)	22.263	83.999	73.294	57.918	26.309
D+L, LL Comb Run (**LL)	22.196	84.412	69.017	76.172	36.179
D+L, LL Comb Run (*L**)	18.191	114.026	103.990	41.343	28.113
D+L, LL Comb Run (*L*L)	18.140	114.157	99.805	60.226	37.528
D+L, LL Comb Run (*LL*)	18.258	113.845	112.925	45.582	27.703
D+L, LL Comb Run (*LLL)	18.228	113.884	109.103	64.048	37.240
D+L, LL Comb Run (L***)	34.487	101.063	62.618	54.820	26.564
D+L, LL Comb Run (L**L)	34.441	101.444	58.234	73.263	36.362
D+L, LL Comb Run (L*L*)	34.517	100.745	72.293	58.382	26.263
D+L, LL Comb Run (L*LL)	34.476	101.102	68.122	76.522	36.170
D+L, LL Comb Run (LL**)	30.957	129.726	103.781	41.580	28.037
D+L, LL Comb Run (LL*L)	30.883	129.909	99.567	60.443	37.470
D+L, LL Comb Run (LLL*)	31.029	129.570	112.628	45.862	27.638
D+L, LL Comb Run (LLLL)	30.985	129.618	108.922	64.170	37.225

Shear Stirrup Requirements

Between 0.00 to 3.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 4.20 to 13.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 14.40 to 19.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 20.40 to 29.60 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 4.000 in
Between 30.40 to 36.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 36.80 to 44.00 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 44.80 to 49.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 50.40 to 56.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 6.000 in
Between 57.20 to 61.20 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 61.60 to 69.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 70.00 to 71.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 72.00 to 72.00 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 12.000 in
Between 72.60 to 79.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 80.40 to 90.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 91.20 to 95.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.400	-437.09	646.99	0.68
Span # 2	2	24.000	-485.82	576.91	0.84

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Girder G2 (gridline D)

Load Combination Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 4	4	85.800	121.61	474.11	0.26

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L*)	1	0.0426	10.800	D+L, LL Comb Run (*L*L)	-0.0155	20.400
D+L, LL Comb Run (*L*L)	2	0.2161	17.600	D+L, LL Comb Run (*L*L)	-0.0085	32.800
D+L, LL Comb Run (*L*L)	3	0.0089	17.200	D+L, LL Comb Run (*L*L)	-0.0315	7.200
D+L, LL Comb Run (*L*L)	4	0.0744	13.200		0.0000	7.200

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Printed: 15 SEP 2011, 12:52PM

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 — Licensee : AHEARN & KNOX, INC

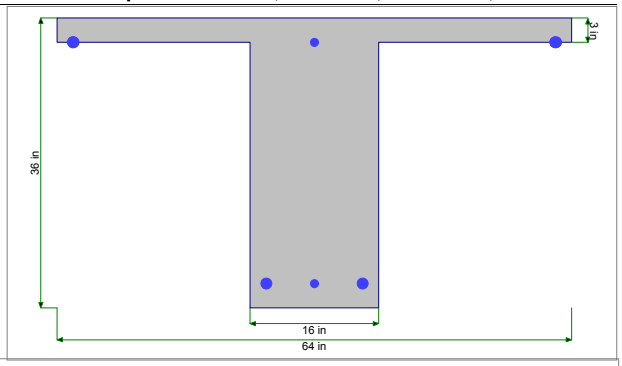
Description : Girder G3 (gridline E)

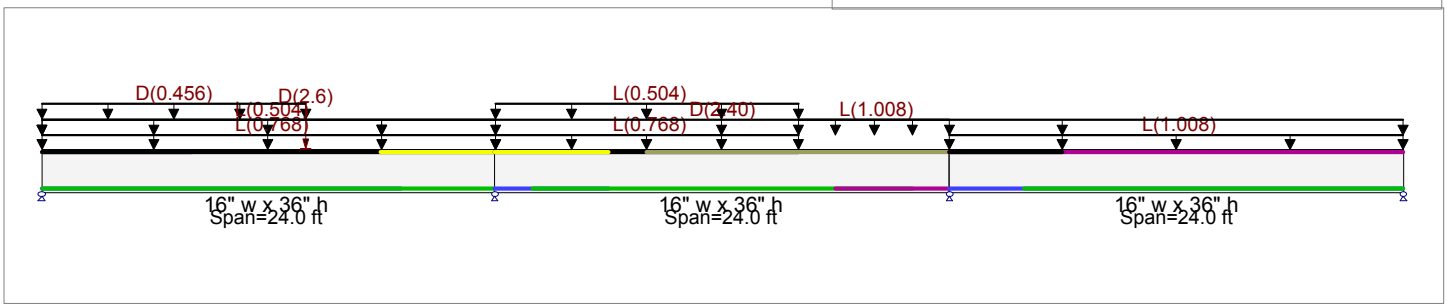
Material Properties

Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05

f'c	=	3.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c $^{1/2}$ * 7.50	=	443.71 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,372.17 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	40.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	# 3
			Number of Resisting Legs Per Stirrup	=	2

Load Combination 2009 IBC & ASCE 7-05





Cross Section & Reinforcing Details

Tee Section, Stem Width = 16.0 in, Total Height = 36.0 in, Top Flange Width = 64.0 in, Flange Thickness = 3.0 in

Span #1 Reinforcing....
- 1-#8 at 3.0 in from Bottom, from 0.0 to 19.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 8.0 ft in this span
- 1-#8 at 3.0 in from Top, from 18.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span

Span #2 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 6.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 18.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 6.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 2.0 to 22.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 2-#11 at 3.0 in from Top, from 8.0 to 24.0 ft in this span

Span #3 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Bottom, from 4.0 to 24.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 6.0 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...
D = 0.10
Uniform Load on ALL spans : D = 0.10 ksf, Tributary Width = 24.0 ft

Load for Span Number 1
Uniform Load : L = 0.0640 ksf, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0420 ksf, Tributary Width = 12.0 ft, (office LL)
Uniform Load : D = 0.0380 ksf, Extent = 0.0 -->> 14.0 ft, Tributary Width = 12.0 ft, (addl slab DL)
Point Load : D = 2.60 k @ 14.0 ft

Load for Span Number 2
Uniform Load : L = 0.0640 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 12.0 ft, (corridor LL)
Uniform Load : L = 0.0420 ksf, Extent = 16.0 -->> 24.0 ft, Tributary Width = 24.0 ft, (office LL)
Uniform Load : L = 0.0420 ksf, Extent = 0.0 -->> 16.0 ft, Tributary Width = 12.0 ft, (office LL)

Load for Span Number 3
Uniform Load : L = 0.0420 ksf, Tributary Width = 24.0 ft, (office LL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.997 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward L+Lr+S Deflection	0.043 in	Ratio = 6648
Mu : Applied		118.47 k-ft	Max Upward L+Lr+S Deflection	-0.041 in	Ratio = 6969
Mn * Phi : Allowable		118.80 k-ft	Max Downward Total Deflection	0.081 in	Ratio = 3572
Load Combination		1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*L*)	Max Upward Total Deflection	-0.017 in	Ratio = 16621
Location of maximum on span		12.200 ft			
Span # where maximum occurs		Span # 2			

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Job # M11-040

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381

Licensee : AHEARN & KNOX, INC

Description : Girder G3 (gridline E)

Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum	40.666	103.500	92.137	34.360
D Only	27.966	68.266	62.342	23.210
L Only, LL Comb Run (**L)	0.403	-2.419	15.725	10.483
L Only, LL Comb Run (*L*)	-1.470	16.434	14.870	-1.418
L Only, LL Comb Run (*LL)	-1.067	14.015	30.595	9.065
L Only, LL Comb Run (L**)	13.229	19.843	-3.053	0.509
L Only, LL Comb Run (L*L)	13.632	17.424	12.672	10.992
L Only, LL Comb Run (LL*)	11.758	36.278	11.817	-0.909
L Only, LL Comb Run (LLL)	12.162	33.858	27.542	9.574
D+L, LL Comb Run (**L)	28.375	65.818	78.111	33.673
D+L, LL Comb Run (*L*)	26.550	84.579	77.292	21.778
D+L, LL Comb Run (*LL)	26.815	82.807	92.137	32.632
D+L, LL Comb Run (L**)	40.558	89.547	58.323	23.883
D+L, LL Comb Run (L*L)	40.666	87.714	73.764	34.360
D+L, LL Comb Run (LL*)	40.193	103.500	74.841	22.194
D+L, LL Comb Run (LLL)	40.274	101.926	89.845	32.876

Shear Stirrup Requirements

Between 0.00 to 5.40 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 5.60 to 13.80 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 14.00 to 20.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 20.20 to 25.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 9.000 in
Between 26.00 to 31.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 31.80 to 40.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 40.80 to 46.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 46.80 to 49.80 ft, PhiVc < Vu, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 12.000 in
Between 50.00 to 56.60 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in
Between 56.80 to 67.40 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd, use stirrups spaced at 0.000 in
Between 67.60 to 71.80 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 11.5.6.3, use stirrups spaced at 11.000 in

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	23.800	-304.88	367.89	0.83
Span # 2	2	36.200	118.47	118.80	1.00
Span # 3	3	48.000	-277.39	367.89	0.75
+1.40D					
Span # 1	1	23.800	-205.60	367.89	0.56
Span # 2	2	47.800	-180.07	296.54	0.61
Span # 3	3	48.000	-187.83	367.89	0.51
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-160.88	367.89	0.44
Span # 2	2	47.800	-215.64	296.54	0.73
Span # 3	3	48.000	-222.93	367.89	0.61
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-232.22	367.89	0.63
Span # 2	2	36.200	118.47	118.80	1.00
Span # 3	3	48.000	-215.46	367.89	0.59
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-216.87	367.89	0.59
Span # 2	2	47.800	-265.82	296.54	0.90
Span # 3	3	48.000	-277.39	367.89	0.75
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	10.000	274.14	399.60	0.69
Span # 2	2	24.000	-263.59	367.89	0.72
Span # 3	3	48.000	-141.46	367.89	0.38
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	10.200	280.63	399.60	0.70
Span # 2	2	24.000	-248.11	367.89	0.67
Span # 3	3	61.800	229.76	399.60	0.57
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-304.88	367.89	0.83
Span # 2	2	24.000	-320.05	367.89	0.87
Span # 3	3	48.000	-195.92	367.89	0.53
+1.20D+0.50Lr+1.60L+1.60H, LL Comb					
Span # 1	1	23.800	-289.52	367.89	0.79
Span # 2	2	47.800	-247.10	296.54	0.83

1505 Meridian Avenue, Suite B
San Jose, CA 95125
(408) 978-1970
WWW.AKHSE.COM

Title : Cupertino City Hall Job # M11-040
Dsgnr:
Project Desc.:

Project Notes :

Concrete Beam

File: P:\Cupertino\M11-040 City Hall Analysis\Calcs\m11-040 enercalc.ec6
ENERCALC, INC. 1983-2011, Build:6.11.9.9, Ver:6.11.9.9

Lic. # : KW-06003381 **Licensee : AHEARN & KNOX, INC**

Description : Girder G3 (gridline E)

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 3	3	48.000	-257.85	367.89	0.70
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-160.88	367.89	0.44
Span # 2	2	47.800	-215.64	296.54	0.73
Span # 3	3	48.000	-222.93	367.89	0.61
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-232.22	367.89	0.63
Span # 2	2	36.200	118.47	118.80	1.00
Span # 3	3	48.000	-215.46	367.89	0.59
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-216.87	367.89	0.59
Span # 2	2	47.800	-265.82	296.54	0.90
Span # 3	3	48.000	-277.39	367.89	0.75
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	10.000	274.14	399.60	0.69
Span # 2	2	24.000	-263.59	367.89	0.72
Span # 3	3	48.000	-141.46	367.89	0.38
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	10.200	280.63	399.60	0.70
Span # 2	2	24.000	-248.11	367.89	0.67
Span # 3	3	61.800	229.76	399.60	0.57
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-304.88	367.89	0.83
Span # 2	2	24.000	-320.05	367.89	0.87
Span # 3	3	48.000	-195.92	367.89	0.53
+1.20D+1.60L+0.50S+1.60H, LL Comb R					
Span # 1	1	23.800	-289.52	367.89	0.79
Span # 2	2	47.800	-247.10	296.54	0.83
Span # 3	3	48.000	-257.85	367.89	0.70
+1.20D+1.60Lr+0.50L, LL Comb Run (*					
Span # 1	1	23.800	-171.43	367.89	0.47
Span # 2	2	47.800	-173.50	296.54	0.59
Span # 3	3	48.000	-180.35	367.89	0.49
+1.20D+1.60Lr+0.50L, LL Comb Run (*					
Span # 1	1	23.800	-193.73	367.89	0.53
Span # 2	2	47.800	-170.03	296.54	0.57
Span # 3	3	48.000	-178.02	367.89	0.48
+1.20D+1.60Lr+0.50L, LL Comb Run (*					
Span # 1	1	23.800	-188.93	367.89	0.51
Span # 2	2	47.800	-189.18	296.54	0.64
Span # 3	3	48.000	-197.37	367.89	0.54
+1.20D+1.60Lr+0.50L, LL Comb Run (L					
Span # 1	1	23.800	-198.93	367.89	0.54
Span # 2	2	24.000	-209.86	367.89	0.57
Span # 3	3	48.000	-154.89	367.89	0.42
+1.20D+1.60Lr+0.50L, LL Comb Run (L					
Span # 1	1	23.800	-194.14	367.89	0.53
Span # 2	2	47.800	-167.65	296.54	0.57
Span # 3	3	48.000	-174.25	367.89	0.47
+1.20D+1.60Lr+0.50L, LL Comb Run (L					
Span # 1	1	23.800	-216.43	367.89	0.59
Span # 2	2	24.000	-227.51	367.89	0.62
Span # 3	3	48.000	-171.91	367.89	0.47
+1.20D+1.60Lr+0.50L, LL Comb Run (L					
Span # 1	1	23.800	-211.63	367.89	0.58
Span # 2	2	47.800	-183.33	296.54	0.62
Span # 3	3	48.000	-191.27	367.89	0.52

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L, LL Comb Run (L*L)	1	0.0806	10.200	D+L, LL Comb Run (L*L)	-0.0029	24.600
D+L, LL Comb Run (L*L)	2	0.0027	24.600	D+L, LL Comb Run (L*L)	-0.0173	9.000
D+L, LL Comb Run (L*L)	3	0.0519	12.600		0.0000	9.000