# 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**

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**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.

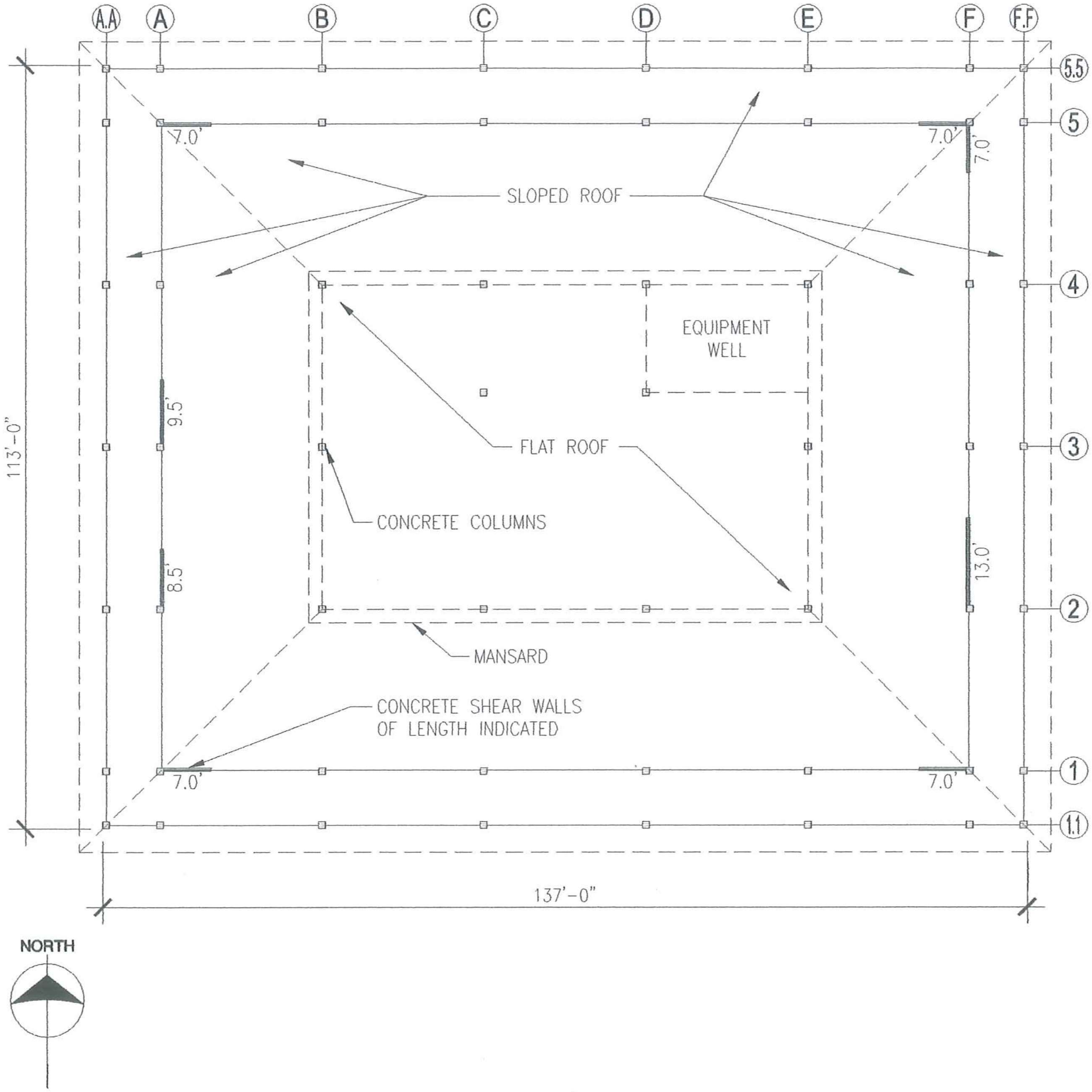
Mr. Greene received a preliminary report from Bill Knox in November of 2005, which became the basis for a staff report by Mr. Greene to the Director of Public Works in December of 2005. Mr. Knox later produced a re-cap of the report, in April of 2006 and transmitted that re-cap to Gene Sugimura. Mr.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

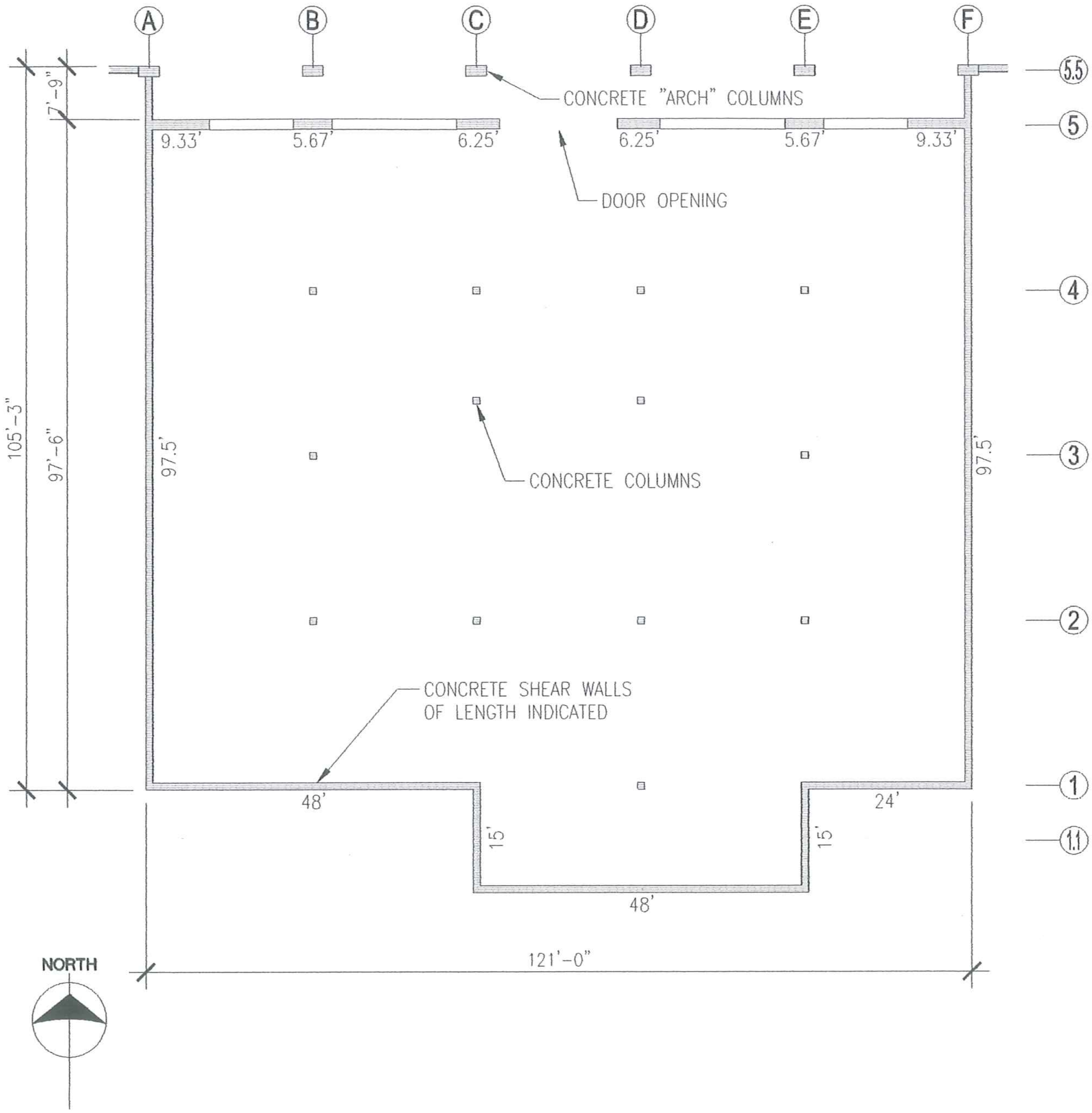
# Key Plans

## 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 x 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 x 1.5 x W = **0.21 x W at Upper Level**
Lower Level: $F_{EQ}$ = 0.19 x 1.5 x 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"$\phi$.) 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 x 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 x 1.5 x W = **0.21 x W at Upper Level**
Lower Level: $F_{EQ}$ = 0.19 x 1.5 x W = **0.28 x W at Lower Level**

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

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

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# 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: | |

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TIM D. HYDE
No. S3260
EXP. 3/31/13
STRUCTURAL
STATE OF CALIFORNIA

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

**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**

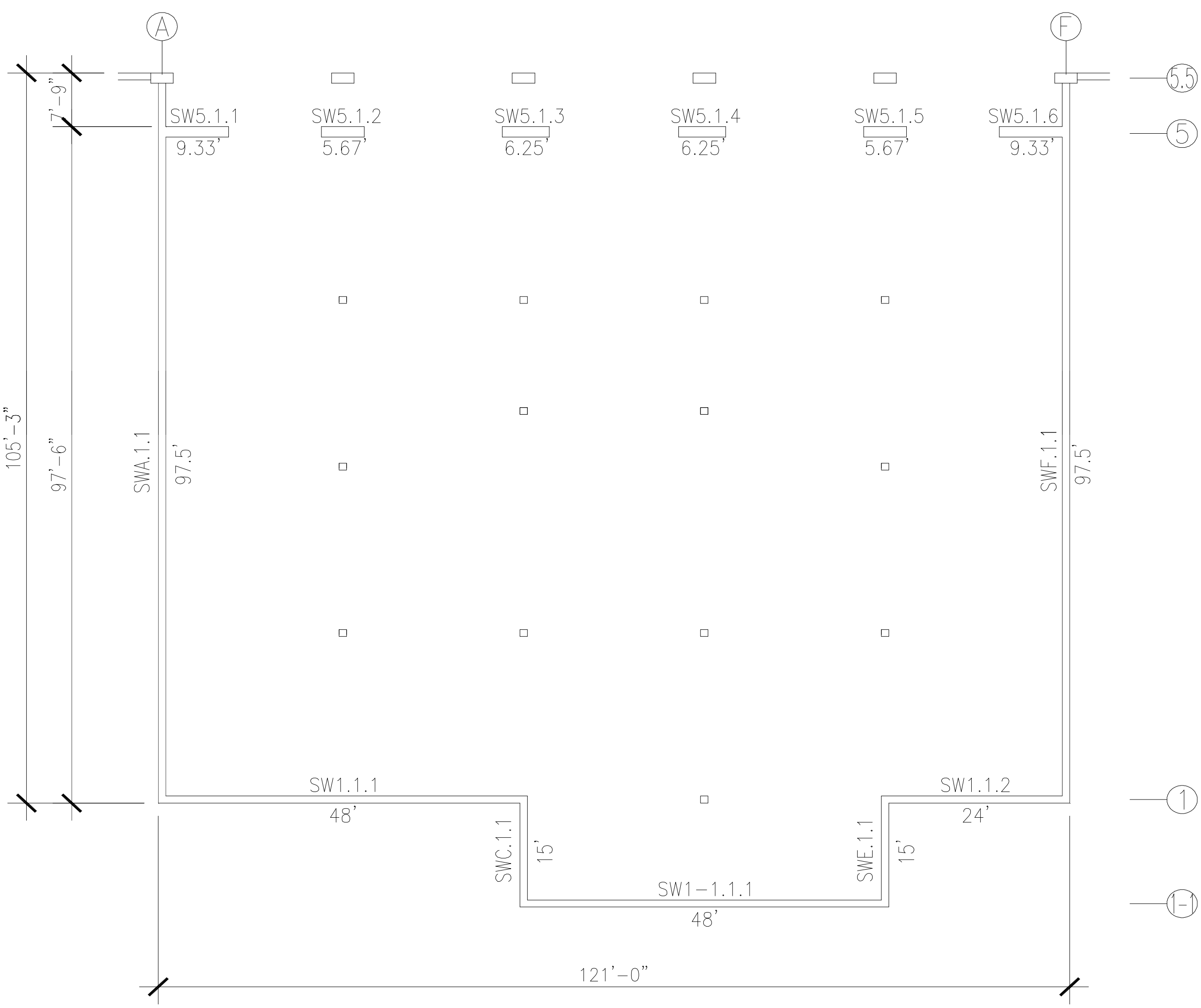
| | | |
|---|---|---|
| Office Areas | 50 | psf |
| Corridors & Lobbies | 100 | psf |
| Assembly, Open | 100 | psf |

| **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 |

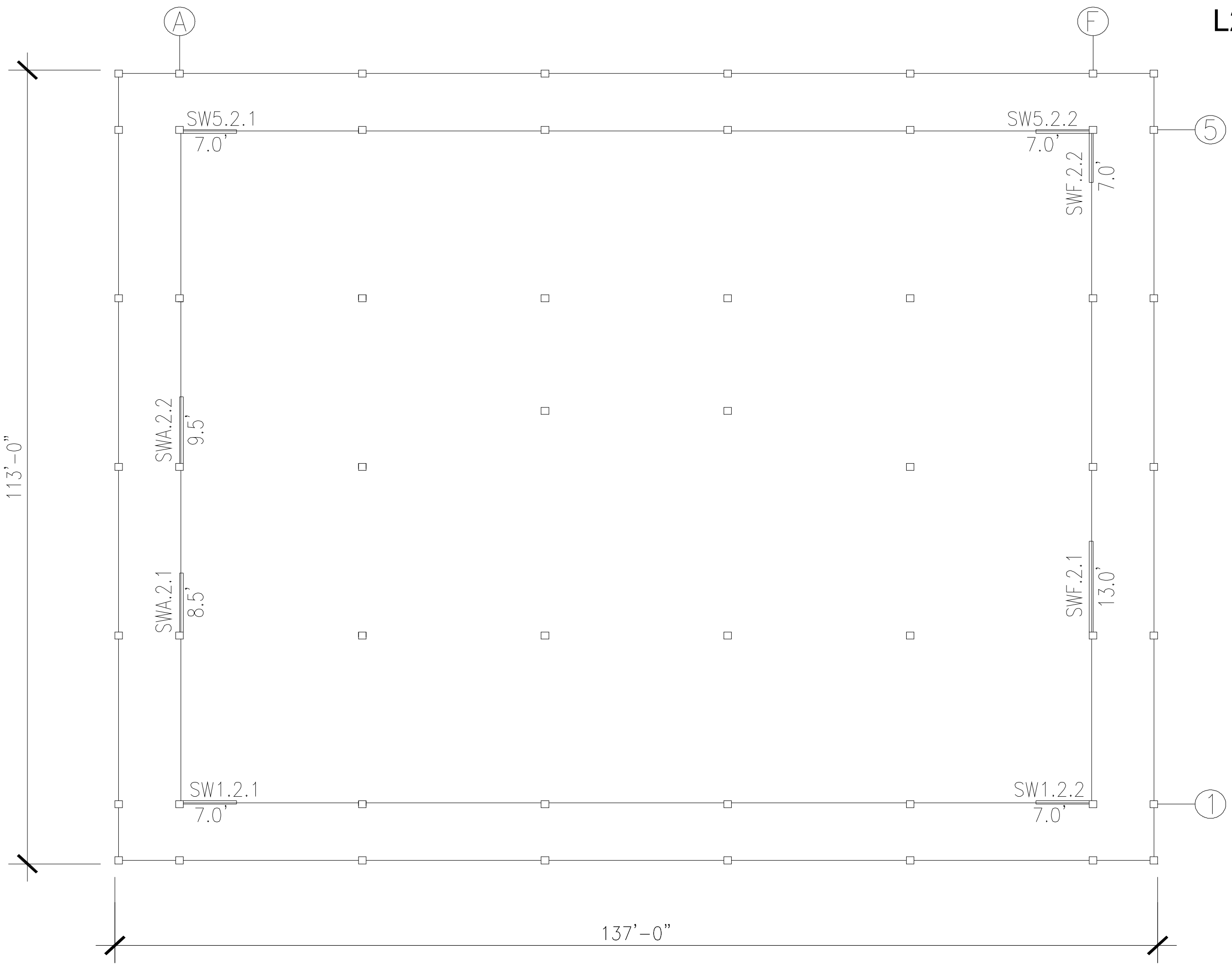
The building was originally designed in 1964 as a non-essential facitlity. The upper level of the building lateral system consists of existing wood roof diaphragm and concrete shear walls. The lower level consists of a concrete joist floor system over concrete walls & columns. The lower portion was originally 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 |

$\boxed{V = 0.210\ W}$

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 |

$\boxed{V = 0.279\ W}$

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

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

$\boxed{F_p = 0.450\ W_p}$

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

Max. $F_{px} = 0.30ZIw_{px}$ = 0.450 $w_{px}$
Min. $F_{px} = 0.14ZIw_{px}$ = 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 | $w_{px}$ |

$\boxed{F_{px} = 0.210\ w_{px}}$

**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 $W_{N\text{-}S}$ = V / L = 4.38 kip/ft.
Longitudinal (E-W) load $W_{E\text{-}W}$ = 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 Overstressed. Therefore, Diaphragm Is NOT Adequate.** |
|---|
| **DEFICIENCIES: 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"$\phi$ 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"$\phi$ 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** |

<u>Line F</u>

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!** | | | |

<u>Line 1 (Line 5 Same)</u>

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 <u>Overstressed</u>. Therefore, Beam Splice Connection Is <u>NOT</u> Adequate.** |
| **<u>DEFICIENCIES</u>: 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 <u>Overstressed</u>. Therefore, Beam Anchorage Is <u>NOT</u> Adequate.** |
| **<u>DEFICIENCIES</u>: 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 > $\phi M_n$ = 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.: $\phi$ = 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

$\phi V_n$ = 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 <u>Overstressed</u> in In-Plane bending. Therefore, Walls Are <u>NOT</u> Adequate.** |
|---|
| **<u>DEFICIENCIES</u>: 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 > $\phi M_n$ = 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.: $\phi$ = 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

$\phi V_n$ = 192.6 kips > $V_u$ = 74.8 kips **OK**

| |
|---|
| **Existing Concrete Shear Wall Along Line A Is Highly Overstressed in In-Plane bending.**<br>**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 > $\phi M_n$ = 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 > $\phi M_n$ = 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 > $\phi M_n$ = 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.**<br>**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 > $\phi M_n$ = 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 < $\phi M_n$ = 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 $L = L_o(0.25+15/\mathrm{sqrt}(K_{LL}A_T))$

| | | |
|---|---|---|
| $K_{LL}$ = | 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 |
| 65000 | 24193 | no | 12.4 | 11 | 12 | 11 |
| 60000 | 17526 | no | 12.4 | 11 | 16 | 11 |
| 50000 | 4193 | no | 12.4 | 11 | 69 | 11 |
| 40000 | 0 | no | 12.4 | 11 | not req'd | 11 |
| 30000 | 0 | no | 12.4 | 11 | not req'd | 11 |

Per Enercalc Analysis:
Vu max = 78730 lbs
$\phi$Vn for ties @ 12" = 65000 lbs
max overstress = 1.21

Deflection limitations

<u>1964 UBC & 2010 CBC</u>

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).** |
|---|
| **<u>DEFICIENCIES:</u> Shear Reinforcement Spacing is Too Large at Some Locations** |

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Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040



# 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 |
| Fy : Rebar Yield | = | 40.0 | ksi |
| Ec : Concrete Elastic Modulus | = | 3,372.17 | ksi |
| $\lambda$ : Lt Wt Conc Factor | = | 1.0 | |
| Fr : Rupture Modulus | = | 295.80 | psi |
| Max % of $\rho$ balanced | = | 0.60 | |
| Max Pu/Ag = f'c * | = | 0.060 | |
| Concrete Density | = | 144.0 | pcf |
| Width of Design Strip | = | 12.0 | in |

| | | | |
|---|---|---|---|
| Wall Thickness | | 6.0 | in |
| Rebar at wall center | | | |
| Rebar "d" distance | | 3.0 | in |
| Lower Level Rebar . . . | | | |
| Bar Size | # | 4 | |
| Bar Spacing | | 12.0 | in |

| | | | |
|---|---|---|---|
| Temp Diff across thickness | = | | deg F |
| Min Allow Out-of-Plane Defl Ratio | = | L / 150.0 | |
| Minimum Vertical Steel % | = | 0.0020 | |

Using Stiffness Reduction Factor per ACI R.10.12.3

## 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)*

| | Eccentricity | | DL : Dead Load | Lr : Roof Live Load | Lf : Floor Live Load | S : Snow Load | |
|---|---|---|---|---|---|---|---|
| Ledger Load | | 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 |

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# Concrete Slender Wall



Description : Existing 6" wall

## Design Maximum Combinations - Moments

| Load Combination | Axial Load Pu (k) | Axial Load 0.06*f'c*b*t (k) | 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) | Moment Values Mactual (k-ft) | Stiffness I gross (in^4) | Stiffness I cracked (in^4) | Stiffness I effective (in^4) | Deflections Deflection (in) | Deflections 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: $\phi$ = 0.85

$V_u$ = 74.8 kips — $\phi V_n$ = 107.6 kips — OK — DCR = 0.70

Bending Check: $\phi$ = 0.88

$M_u$ left = 887.2 ft-kips — $\phi M_n$ = 364.1 ft-kips — Overstressed! — DCR = 2.44

$M_u$ right = 887.2 ft-kips — $\phi M_n$ = 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: $\phi$ = 0.85

$V_u$ = 68.2 kips $\phi V_n$ = 120.1 kips OK DCR = 0.57

Bending Check: $\phi$ = 0.88

$M_u$ left = 805.1 ft-kips $\phi M_n$ = 285.4 ft-kips Overstressed! DCR = 2.82

$M_u$ right = 805.1 ft-kips $\phi M_n$ = 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: $\phi$ = 0.85

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $V_u$ = | 82.6 | kips | $\phi V_n$ = | 134.2 | kips | OK | DCR = | 0.62 |

Bending Check: $\phi$ = 0.88

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $M_u$ left = | 976.2 | ft-kips | $\phi M_n$ = | 406.2 | ft-kips | Overstressed! | DCR = | 2.40 |
| $M_u$ right = | 976.2 | ft-kips | $\phi M_n$ = | 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: $\phi$ = 0.85

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $V_u$ = | 111.7 | kips | $\phi V_n$ = | 183.7 | kips | OK | DCR = | 0.61 |

Bending Check: $\phi$ = 0.88

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $M_u$ left = | 1320.0 | ft-kips | $\phi M_n$ = | 699.2 | ft-kips | Overstressed! | DCR = | 1.89 |
| $M_u$ right = | 1320.0 | ft-kips | $\phi M_n$ = | 699.2 | ft-kips | Overstressed! | DCR = | 1.89 |

Boundary Member Check:

Boundary Member Required: NO

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## 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: $\phi$ = 0.85

$V_u$ = 68.2 kips $\phi V_n$ = 120.1 kips OK DCR = 0.57

Bending Check: $\phi$ = 0.88

$M_u$ left = 805.1 ft-kips $\phi M_n$ = 285.4 ft-kips Overstressed! DCR = 2.82

$M_u$ right = 805.1 ft-kips $\phi M_n$ = 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: $\phi$ = 0.85

$V_u$ = 82.6 kips — $\phi V_n$ = 134.2 kips — OK — DCR = 0.62

Bending Check: $\phi$ = 0.88

$M_u$ left = 976.2 ft-kips — $\phi M_n$ = 406.2 ft-kips — Overstressed! — DCR = 2.40

$M_u$ right = 976.2 ft-kips — $\phi M_n$ = 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: $\phi$ = 0.85

$V_u$ = 111.7 kips — $\phi V_n$ = 183.7 kips — OK — DCR = 0.61

Bending Check: $\phi$ = 0.88

$M_u$ left = 1320.0 ft-kips — $\phi M_n$ = 699.2 ft-kips — Overstressed! — DCR = 1.89

$M_u$ right = 1320.0 ft-kips — $\phi M_n$ = 699.2 ft-kips — Overstressed! — DCR = 1.89

Boundary Member Check:

Boundary Member Required: NO

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# 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: $\phi$ = 0.85

$V_u$ = 39.6 kips — $\phi V_n$ = 107.6 kips — OK — DCR = 0.37

Bending Check: $\phi$ = 0.88

$M_u$ left = 464.7 ft-kips — $\phi M_n$ = 232.4 ft-kips — Overstressed! — DCR = 2.00

$M_u$ right = 464.7 ft-kips — $\phi M_n$ = 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: $\phi$ = 0.85

$V_u$ = 329.7 kips $\phi V_n$ = 1347.4 kips OK DCR = 0.24

Bending Check: $\phi$ = 0.89

$M_u$ left = 4193.3 ft-kips $\phi M_n$ = 4503.2 ft-kips OK DCR = 0.93

$M_u$ right = 4193.3 ft-kips $\phi M_n$ = 4503.2 ft-kips OK DCR = 0.93

Boundary Member Check:

Boundary Member Required: NO

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## 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: $\phi$ = 0.85

$V_u$ = 181.6 kips $\phi V_n$ = 687.4 kips OK DCR = 0.26

Bending Check: $\phi$ = 0.89

$M_u$ left = 2267.7 ft-kips $\phi M_n$ = 2292.8 ft-kips OK DCR = 0.99

$M_u$ right = 2267.7 ft-kips $\phi M_n$ = 2292.8 ft-kips OK DCR = 0.99

Boundary Member Check:

Boundary Member Required: NO

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## 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: $\phi$ = 0.85

$V_u$ = 44.0 kips $\phi V_n$ = 243.4 kips OK DCR = 0.18

Bending Check: $\phi$ = 0.89

$M_u$ left = 236.7 ft-kips $\phi M_n$ = 252.6 ft-kips OK DCR = 0.94

$M_u$ right = 236.7 ft-kips $\phi M_n$ = 252.6 ft-kips OK DCR = 0.94

Boundary Member Check:

Boundary Member Required: NO

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1505 Meridian Avenue, Suite B
San Jose, CA 95125
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Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040

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**

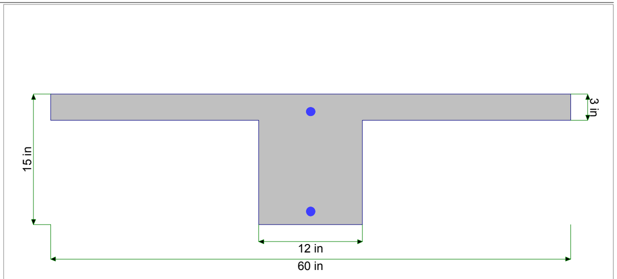
$f'c$ = 3.50 ksi
$fr = f'c^{1/2} * 7.50$ = 443.71 psi
$\psi$ Density = 145.0 pcf
$\lambda$ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi

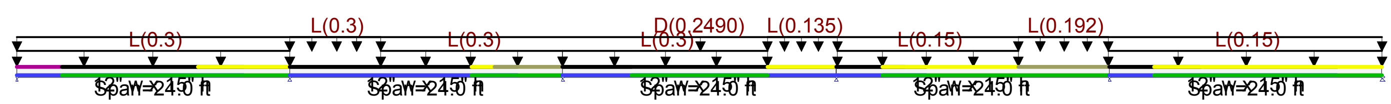
fy - Main Rebar = 40.0 ksi
E - Main Rebar = 29,000.0 ksi

$\phi$ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850

Fy - Stirrups 40.0 ksi
E - Stirrups = 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 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 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 2.0 in from Top, from 16.0 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 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 8.0 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 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 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 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 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 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 6.0 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 4.0 to 24.0 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 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 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 4.0 to 24.0 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)

# Concrete Beam



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
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WWW.AKHSE.COM

Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040

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)

## 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 |

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Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040

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 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 |

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## Concrete Beam



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

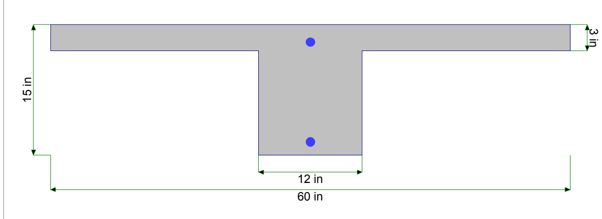
Description : Joist J-1 (betw. gl 1 & 2; corridor)

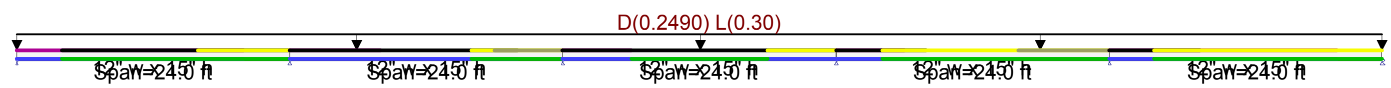
### Material Properties

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

| | | | | | |
|---|---|---|---|---|---|
| f'c | = | 3.50 ksi | $\phi$ Phi Values | Flexure : | 0.90 |
| fr = $f'c^{1/2}$ * 7.50 | = | 443.71 psi | | Shear : | 0.750 |
| $\psi$ Density | = | 145.0 pcf | $\beta_1$ | = | 0.850 |
| $\lambda$ 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 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 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 2.0 in from Top, from 16.0 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 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 8.0 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 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 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 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 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 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 6.0 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 4.0 to 24.0 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 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 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 4.0 to 24.0 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 | |
| Section used for this span | **Typical Section** | |
| Mu : Applied | -51.188 | k-ft |
| Mn * Phi : Allowable | 57.522 | k-ft |
| Load Combination | +1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*L*LL) | |
| Location of maximum on span | 0.000 | ft |
| Span # where maximum occurs | Span # 5 | |

| Maximum Deflection | | | | |
|---|---|---|---|---|
| Max Downward L+Lr+S Deflection | 0.113 | in | Ratio = | 2549 |
| Max Upward L+Lr+S Deflection | -0.067 | in | Ratio = | 4301 |
| Max Downward Total Deflection | 0.156 | in | Ratio = | 1846 |
| Max Upward Total Deflection | -0.044 | in | Ratio = | 6573 |

## Concrete Beam



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 |

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# Concrete Beam



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 |

## Concrete Beam



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 |

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# Concrete Beam

**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**

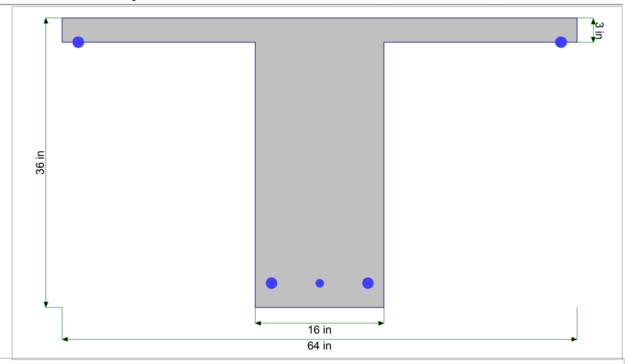
f'c = 3.50 ksi
fr = f'c$^{1/2}$ * 7.50 = 443.71 psi
$\psi$ Density = 145.0 pcf
$\lambda$ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi

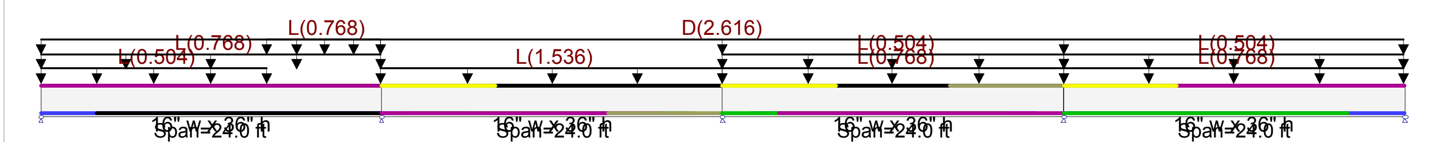
fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi

$\phi$ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850

Fy - Stirrups 40.0 ksi
E - Stirrups = 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)

# Concrete Beam



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 |

# Concrete Beam



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 |

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Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040

Printed: 15 SEP 2011, 12:51PM
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

# Concrete Beam

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

Description : Girder G2 (gridline C; fixed seating)

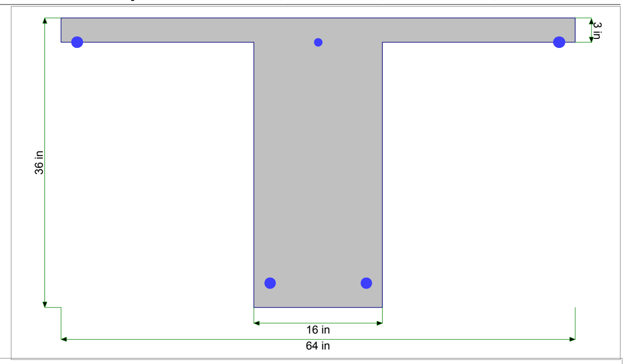
## Material Properties

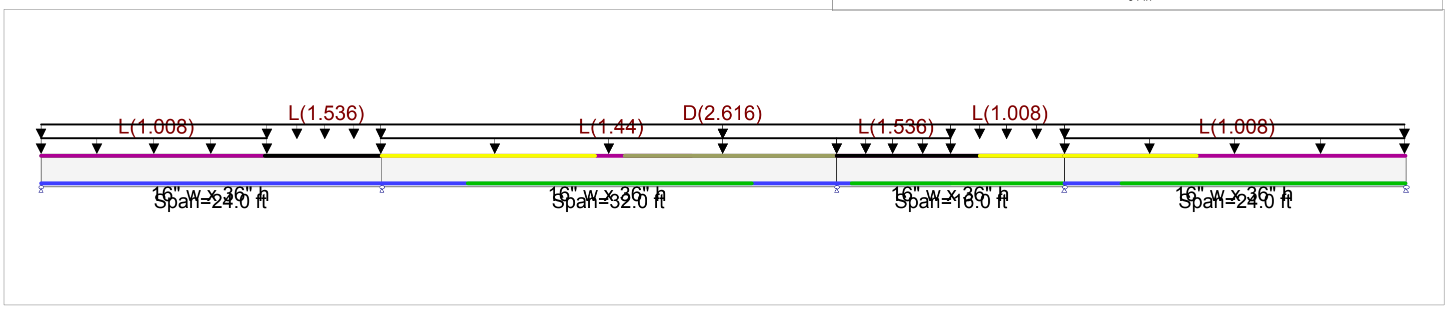
**Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05**

f'c = 3.50 ksi
fr = $f'c^{1/2} * 7.50$ = 443.71 psi
ψ Density = 145.0 pcf
λ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi
fy - Main Rebar = 40.0 ksi
E - Main Rebar = 29,000.0 ksi

φ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850
Fy - Stirrups 40.0 ksi
E - Stirrups = 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
- 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 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-#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 0.0 to 21.750 ft in this span
- 2-#11 at 3.0 in from Top, from 17.10 to 24.0 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 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 Bottom, from 1.0 to 16.0 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 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

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

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# Concrete Beam



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 | | | Bending Stress Results ( k-ft ) | | |
|---|---|---|---|---|---|
| Segment Length | Span # | Location (ft) in Span | 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 |

# Concrete Beam



Description : Girder G2 (gridline D)

| Load Combination<br>Segment Length | Span # | Location (ft)<br>in Span | Bending Stress Results ( k-ft )<br>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 |

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Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040



# Concrete Beam

**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**

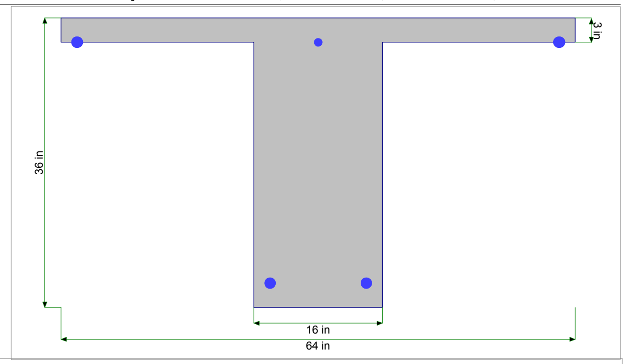
f'c = 3.50 ksi
fr = $f'c^{1/2}$ * 7.50 = 443.71 psi
$\psi$ Density = 145.0 pcf
$\lambda$ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi

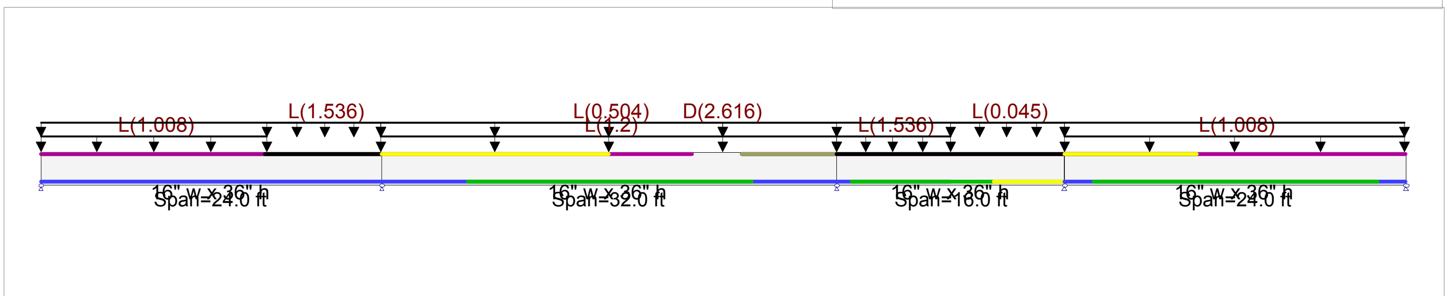
fy - Main Rebar = 40.0 ksi
E - Main Rebar = 29,000.0 ksi

$\phi$ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850

Fy - Stirrups 40.0 ksi
E - Stirrups = 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)

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# Concrete Beam





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 | | | Bending Stress Results ( k-ft ) | | |
|---|---|---|---|---|---|
| Segment Length | Span # | Location (ft) in Span | 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 |

# Concrete Beam



Description :  Girder G2 (gridline D)

| Load Combination<br>Segment Length | Span # | Location (ft)<br>in Span | Bending Stress Results ( k-ft )<br>Mu : Max | <br>Phi*Mnx | <br>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 |

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Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040

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)

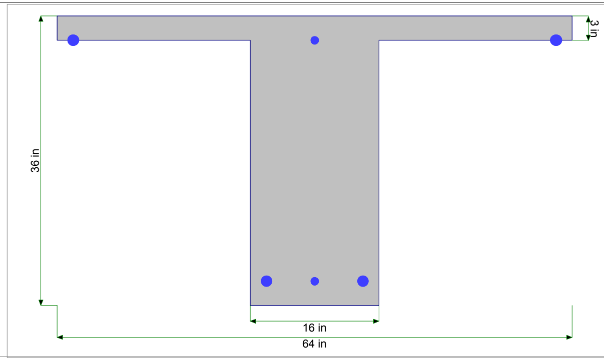
## Material Properties

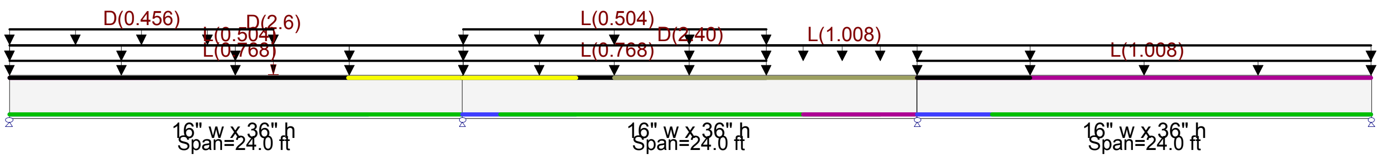
**Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05**

f'c = 3.50 ksi
fr = $f'c^{1/2}$ * 7.50 = 443.71 psi
ψ Density = 145.0 pcf
λ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi
fy - Main Rebar = 40.0 ksi
E - Main Rebar = 29,000.0 ksi

φ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850
Fy - Stirrups 40.0 ksi
E - Stirrups = 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 Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 8.0 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 18.0 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 2.0 to 22.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 18.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 6.0 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
- 1-#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 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
Section used for this span **Typical Section**
Mu : Applied 118.47 k-ft
Mn * Phi : Allowable 118.80 k-ft
Load Combination +1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*L*)
Location of maximum on span 12.200 ft
Span # where maximum occurs Span # 2

| Maximum Deflection | | | |
|---|---|---|---|
| Max Downward L+Lr+S Deflection | 0.043 in | Ratio = | 6648 |
| Max Upward L+Lr+S Deflection | -0.041 in | Ratio = | 6969 |
| Max Downward Total Deflection | 0.081 in | Ratio = | 3572 |
| Max Upward Total Deflection | -0.017 in | Ratio = | 16621 |

# Concrete Beam



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 |

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## Concrete Beam



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 |

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# Concrete Slender Wall



**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 |
| Fy : Rebar Yield | = | 40.0 | ksi |
| Ec : Concrete Elastic Modulus | = | 3,372.17 | ksi |
| $\lambda$ : Lt Wt Conc Factor | = | 1.0 | |
| Fr : Rupture Modulus | = | 295.80 | psi |
| Max % of $\rho$ balanced | = | 0.60 | |
| Max Pu/Ag = f'c * | = | 0.060 | |
| Concrete Density | = | 144.0 | pcf |
| Width of Design Strip | = | 12.0 | in |

| | | | |
|---|---|---|---|
| Wall Thickness | | 6.0 | in |
| Rebar at wall center | | | |
| Rebar "d" distance | | 3.0 | in |
| Lower Level Rebar . . . | | | |
| Bar Size | # | 4 | |
| Bar Spacing | | 12.0 | in |

| | | | |
|---|---|---|---|
| Temp Diff across thickness | = | | deg F |
| Min Allow Out-of-Plane Defl Ratio | = L / | 150.0 | |
| Minimum Vertical Steel % | = | 0.0020 | |

Using Stiffness Reduction Factor per ACI R.10.12.3

## 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)*

| | Eccentricity | | DL : Dead Load | Lr : Roof Live Load | Lf : Floor Live Load | S : Snow Load | |
|---|---|---|---|---|---|---|---|
| Ledger Load | | 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 . . . | | |
|---|---|---|---|---|---|---|---|
| | | Maximum Bending Stress Ratio = | **0.420** | | | | |
| PASS | Moment Capacity Check +1.050D+1.280Lr+1.40E | Max Mu | 0.8592 | k-ft | Phi * Mn | 2.046 | k-ft |
| PASS | Service Deflection Check D + L + S + E/1.4 | Min. Defl. Ratio | 9,242.33 | | Max Allow Ratio | 150.0 | |
| | | Max. Deflection | 0.01558 | in | Max. Allow. Defl. | 0.960 | in |
| PASS | Axial Load Check +1.40D at 5.20 to 5.60 | Max Pu / Ag | 26.631 | psi | 0.06 * f'c | 210.0 | psi |
| PASS | Reinforcing Limit Check +1.40D | Controlling As/bd | 0.005556 | | As/bd = 0.50 rho bal | 0.02598 | |
| FAIL | Minimum Moment Check +1.40D | Mcracking | 1.775 | k-ft | Minimum Phi Mn | 1.733 | k-ft |

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 |

# Concrete Slender Wall



Description : Existing 6" wall

## Design Maximum Combinations - Moments

| Load Combination | Axial Load Pu (k) | Axial Load 0.06*f'c*b*t (k) | 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) | Moment Values Mactual (k-ft) | Stiffness I gross (in^4) | Stiffness I cracked (in^4) | Stiffness I effective (in^4) | Deflections Deflection (in) | Deflections 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: $\phi$ = 0.85

$V_u$ = 74.8 kips — $\phi V_n$ = 107.6 kips — OK — DCR = 0.70

Bending Check: $\phi$ = 0.88

$M_u$ left = 887.2 ft-kips — $\phi M_n$ = 364.1 ft-kips — Overstressed! — DCR = 2.44

$M_u$ right = 887.2 ft-kips — $\phi M_n$ = 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: $\phi$ = 0.85

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $V_u$ = | 68.2 | kips | $\phi V_n$ = | 120.1 | kips | OK | DCR = | 0.57 |

Bending Check: $\phi$ = 0.88

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $M_u$ left = | 805.1 | ft-kips | $\phi M_n$ = | 285.4 | ft-kips | Overstressed! | DCR = | 2.82 |
| $M_u$ right = | 805.1 | ft-kips | $\phi M_n$ = | 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: $\phi$ = 0.85

$V_u$ = 82.6 kips $\phi V_n$ = 134.2 kips OK DCR = 0.62

Bending Check: $\phi$ = 0.88

$M_u$ left = 976.2 ft-kips $\phi M_n$ = 406.2 ft-kips Overstressed! DCR = 2.40

$M_u$ right = 976.2 ft-kips $\phi M_n$ = 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: $\phi$ = 0.85

$V_u$ = 111.7 kips — $\phi V_n$ = 183.7 kips — OK — DCR = 0.61

Bending Check: $\phi$ = 0.88

$M_u$ left = 1320.0 ft-kips — $\phi M_n$ = 699.2 ft-kips — Overstressed! — DCR = 1.89

$M_u$ right = 1320.0 ft-kips — $\phi M_n$ = 699.2 ft-kips — Overstressed! — DCR = 1.89

Boundary Member Check:

Boundary Member Required: NO

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**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: $\phi$ = 0.85

$V_u$ = 68.2 kips — $\phi V_n$ = 120.1 kips — OK — DCR = 0.57

Bending Check: $\phi$ = 0.88

$M_u$ left = 805.1 ft-kips — $\phi M_n$ = 285.4 ft-kips — Overstressed! — DCR = 2.82

$M_u$ right = 805.1 ft-kips — $\phi M_n$ = 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: $\phi$ = 0.85

$V_u$ = 82.6 kips $\phi V_n$ = 134.2 kips OK DCR = 0.62

Bending Check: $\phi$ = 0.88

$M_u$ left = 976.2 ft-kips $\phi M_n$ = 406.2 ft-kips Overstressed! DCR = 2.40

$M_u$ right = 976.2 ft-kips $\phi M_n$ = 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: $\phi$ = 0.85

$V_u$ = 111.7 kips — $\phi V_n$ = 183.7 kips — OK — DCR = 0.61

Bending Check: $\phi$ = 0.88

$M_u$ left = 1320.0 ft-kips — $\phi M_n$ = 699.2 ft-kips — Overstressed! — DCR = 1.89

$M_u$ right = 1320.0 ft-kips — $\phi M_n$ = 699.2 ft-kips — Overstressed! — DCR = 1.89

Boundary Member Check:

Boundary Member Required: NO

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**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: $\phi$ = 0.85

$V_u$ = 39.6 kips $\phi V_n$ = 107.6 kips OK DCR = 0.37

Bending Check: $\phi$ = 0.88

$M_u$ left = 464.7 ft-kips $\phi M_n$ = 232.4 ft-kips Overstressed! DCR = 2.00

$M_u$ right = 464.7 ft-kips $\phi M_n$ = 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: $\phi$ = 0.85

$V_u$ = 329.7 kips $\phi V_n$ = 1347.4 kips OK DCR = 0.24

Bending Check: $\phi$ = 0.89

$M_u$ left = 4193.3 ft-kips $\phi M_n$ = 4503.2 ft-kips OK DCR = 0.93

$M_u$ right = 4193.3 ft-kips $\phi M_n$ = 4503.2 ft-kips OK DCR = 0.93

Boundary Member Check:

Boundary Member Required: NO

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**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: $\phi$ = 0.85

$V_u$ = 181.6 kips $\phi V_n$ = 687.4 kips OK DCR = 0.26

Bending Check: $\phi$ = 0.89

$M_u$ left = 2267.7 ft-kips $\phi M_n$ = 2292.8 ft-kips OK DCR = 0.99

$M_u$ right = 2267.7 ft-kips $\phi M_n$ = 2292.8 ft-kips OK DCR = 0.99

Boundary Member Check:

Boundary Member Required: NO

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## 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!

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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: $\phi$ = 0.85

$V_u$ = 44.0 kips — $\phi V_n$ = 243.4 kips — OK — DCR = 0.18

Bending Check: $\phi$ = 0.89

$M_u$ left = 236.7 ft-kips — $\phi M_n$ = 252.6 ft-kips — OK — DCR = 0.94

$M_u$ right = 236.7 ft-kips — $\phi M_n$ = 252.6 ft-kips — OK — DCR = 0.94

Boundary Member Check:

Boundary Member Required: NO

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# Concrete Beam



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

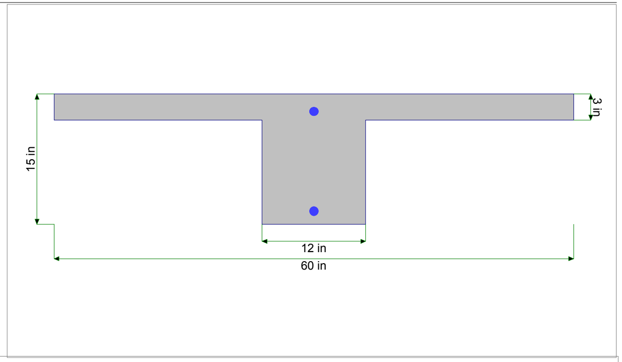
## Material Properties

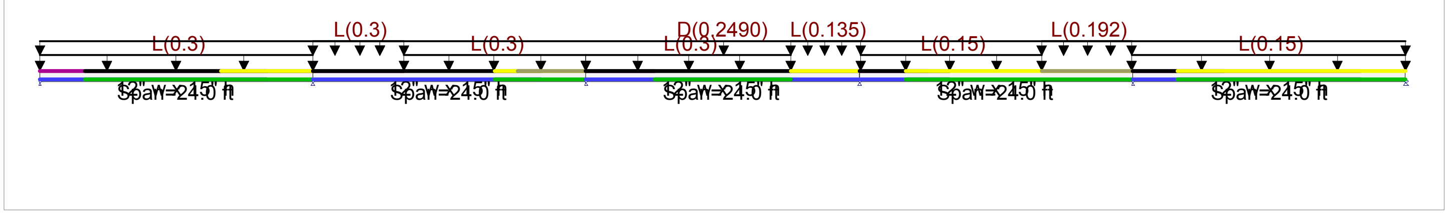
**Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05**

f'c = 3.50 ksi
fr = $f'c^{1/2}$ * 7.50 = 443.71 psi
ψ Density = 145.0 pcf
λ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi
fy - Main Rebar = 40.0 ksi
E - Main Rebar = 29,000.0 ksi

φ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850
Fy - Stirrups 40.0 ksi
E - Stirrups = 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 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 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 2.0 in from Top, from 16.0 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 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 8.0 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 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 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 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 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 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 6.0 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 4.0 to 24.0 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 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 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 4.0 to 24.0 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)

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## Concrete Beam



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 | Maximum Deflection | | |
| Section used for this span | **Typical Section** | Max Downward L+Lr+S Deflection | 0.111 in | Ratio = 2595 |
| Mu : Applied | -50.717 k-ft | Max Upward L+Lr+S Deflection | -0.066 in | Ratio = 4357 |
| Mn * Phi : Allowable | 57.522 k-ft | Max Downward Total Deflection | 0.154 in | Ratio = 1869 |
| Load Combination | +1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (LL*L*) | Max Upward Total Deflection | -0.041 in | Ratio = 6972 |
| Location of maximum on span | 0.000 ft | | | |
| Span # where maximum occurs | Span # 2 | | | |

### 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 |

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# Concrete Beam



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 |

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## Concrete Beam



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 |

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Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040

Printed: 15 SEP 2011, 12:51PM
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

# Concrete Beam

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

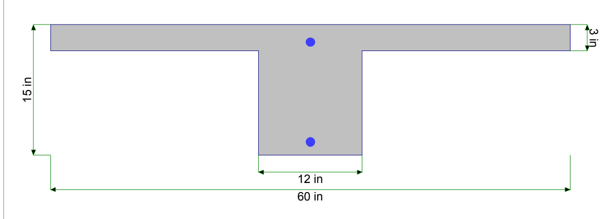
Description : Joist J-1 (betw. gl 1 & 2; corridor)

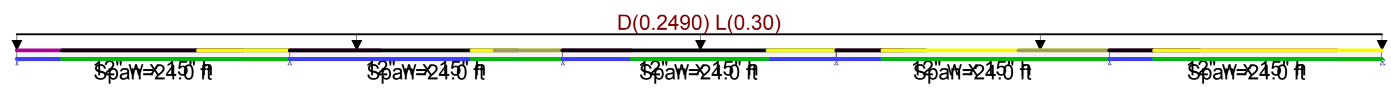
## Material Properties

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

| | | | | |
|---|---|---|---|---|
| f'c | = | 3.50 ksi | $\phi$ Phi Values Flexure : | 0.90 |
| $fr = f'c^{1/2} * 7.50$ | = | 443.71 psi | Shear : | 0.750 |
| $\psi$ Density | = | 145.0 pcf | $\beta_1$ = | 0.850 |
| $\lambda$ 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 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 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 2.0 in from Top, from 16.0 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 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 8.0 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 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 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 6.0 ft in this span
- 1-#8 at 2.0 in from Top, from 0.0 to 24.0 ft in this span
- 1-#8 at 2.0 in from Top, from 18.0 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 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 6.0 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 4.0 to 24.0 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 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 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 4.0 to 24.0 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 |
| Section used for this span | **Typical Section** |
| Mu : Applied | -51.188 k-ft |
| Mn * Phi : Allowable | 57.522 k-ft |
| Load Combination | +1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*L*LL) |
| Location of maximum on span | 0.000 ft |
| Span # where maximum occurs | Span # 5 |

| Maximum Deflection | | |
|---|---|---|
| Max Downward L+Lr+S Deflection | 0.113 in | Ratio = 2549 |
| Max Upward L+Lr+S Deflection | -0.067 in | Ratio = 4301 |
| Max Downward Total Deflection | 0.156 in | Ratio = 1846 |
| Max Upward Total Deflection | -0.044 in | Ratio = 6573 |

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Project Notes :

Job # M11-040



## 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 |

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Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040



## Concrete Beam



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 |

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# Concrete Beam



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 |

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# Concrete Beam



Description : Girder G1 (gridline B)

## Material Properties

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

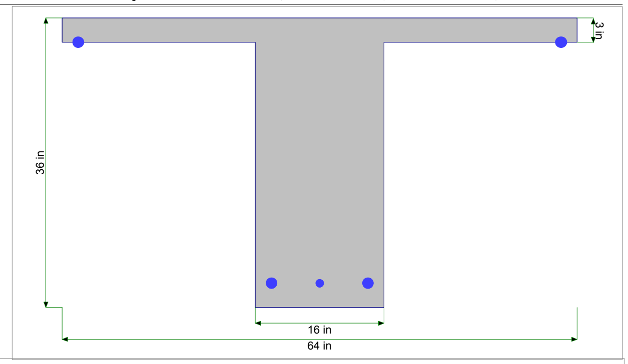
f'c = 3.50 ksi
$fr = f'c^{1/2} * 7.50$ = 443.71 psi
$\psi$ Density = 145.0 pcf
$\lambda$ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi

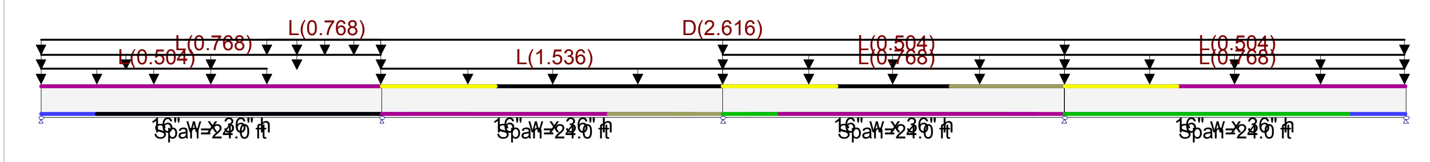
fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi

$\phi$ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850

Fy - Stirrups 40.0 ksi
E - Stirrups = 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-#8 at 3.0 in from Top, from 8.0 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

Span #2 Reinforcing....
- 2-#11 at 3.0 in from Bottom, from 0.0 to 6.0 ft in this span
- 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 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
- 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 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 0.0 to 20.0 ft in this span
- 1-#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 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-#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
- 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 0.0 to 8.0 ft in this span
- 1-#8 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)

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Job # M11-040



# Concrete Beam



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 |

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# Concrete Beam



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 |

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# Concrete Beam

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

Description : Girder G2 (gridline C; fixed seating)

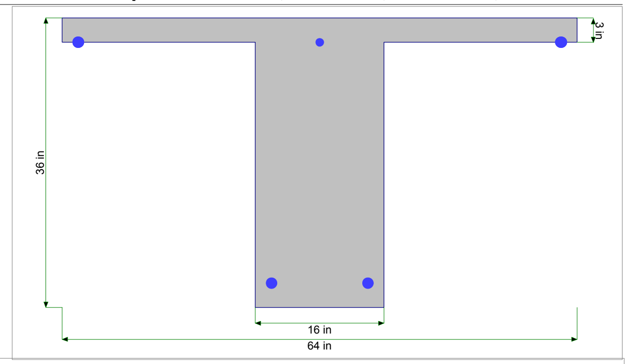
## Material Properties

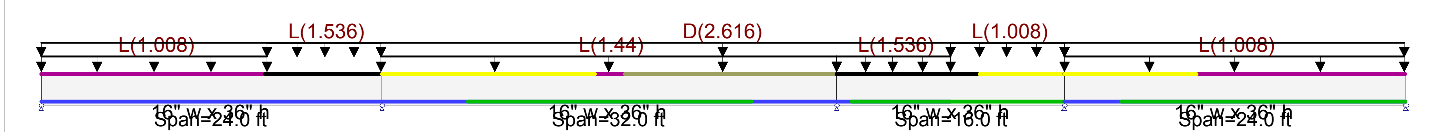
**Calculations per ACI 318-08, IBC 2009, CBC 2010, ASCE 7-05**

f'c = 3.50 ksi
fr = $f'c^{1/2}$ * 7.50 = 443.71 psi
ψ Density = 145.0 pcf
λ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi
fy - Main Rebar = 40.0 ksi
E - Main Rebar = 29,000.0 ksi

φ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850
Fy - Stirrups 40.0 ksi
E - Stirrups = 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
- 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 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-#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 0.0 to 21.750 ft in this span
- 2-#11 at 3.0 in from Top, from 17.10 to 24.0 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 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 Bottom, from 1.0 to 16.0 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 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

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

# Concrete Beam



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 | | | Bending Stress Results ( k-ft ) | | |
|---|---|---|---|---|---|
| Segment Length | Span # | Location (ft) in Span | 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 |

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# Concrete Beam



Description :  Girder G2 (gridline D)

| Load Combination | | | Bending Stress Results ( k-ft ) | | |
|---|---|---|---|---|---|
| Segment Length | Span # | Location (ft) in Span | 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 |

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# Concrete Beam

**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

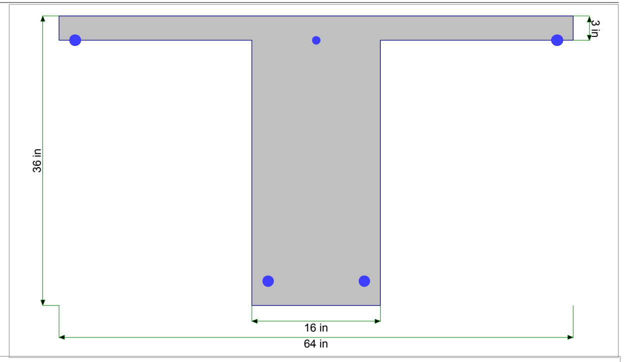
f'c = 3.50 ksi
$fr = f'c^{1/2} * 7.50$ = 443.71 psi
ψ Density = 145.0 pcf
λ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi

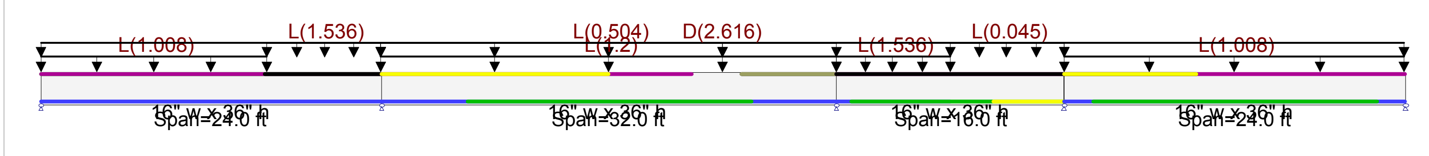
fy - Main Rebar = 40.0 ksi
E - Main Rebar = 29,000.0 ksi

ϕ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850

Fy - Stirrups 40.0 ksi
E - Stirrups = 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
- 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 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-#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 0.0 to 21.750 ft in this span
- 2-#11 at 3.0 in from Top, from 25.250 to 24.0 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 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 Bottom, from 1.0 to 16.0 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 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

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

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# Concrete Beam





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 |

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# Concrete Beam



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 |

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# Concrete Beam





Description : Girder G3 (gridline E)

## Material Properties

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

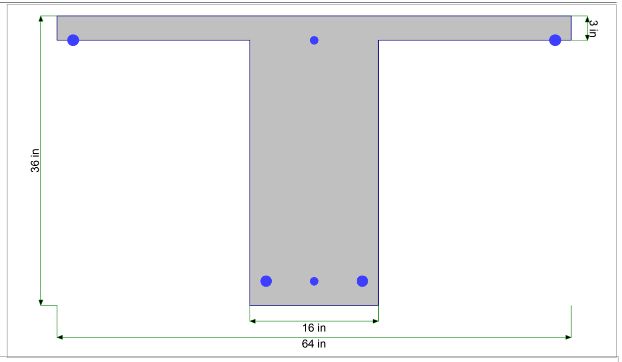
f'c = 3.50 ksi
fr = $f'c^{1/2}$ * 7.50 = 443.71 psi
ψ Density = 145.0 pcf
λ LtWt Factor = 1.0
Elastic Modulus = 3,372.17 ksi

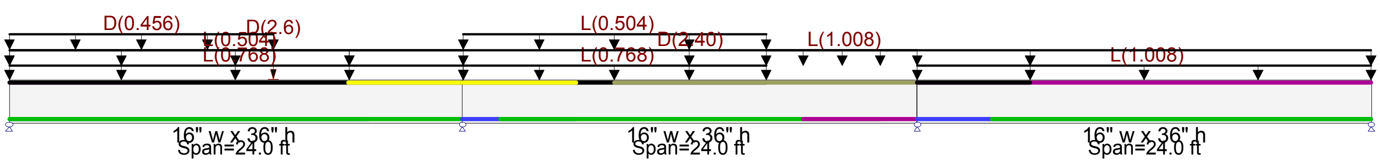
fy - Main Rebar = 40.0 ksi
E - Main Rebar = 29,000.0 ksi

φ Phi Values Flexure : 0.90
Shear : 0.750
$\beta_1$ = 0.850

Fy - Stirrups 40.0 ksi
E - Stirrups = 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 Bottom, from 0.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 8.0 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 18.0 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 2.0 to 22.0 ft in this span
- 2-#11 at 3.0 in from Bottom, from 18.0 to 24.0 ft in this span
- 2-#11 at 3.0 in from Top, from 0.0 to 16.0 ft in this span
- 1-#8 at 3.0 in from Top, from 0.0 to 6.0 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
- 1-#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 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
Section used for this span **Typical Section**
Mu : Applied 118.47 k-ft
Mn * Phi : Allowable 118.80 k-ft
Load Combination +1.20D+0.50Lr+1.60L+1.60H, LL Comb Run (*L*)
Location of maximum on span 12.200 ft
Span # where maximum occurs Span # 2

Maximum Deflection

| | | | |
|---|---|---|---|
| Max Downward L+Lr+S Deflection | 0.043 in | Ratio = | 6648 |
| Max Upward L+Lr+S Deflection | -0.041 in | Ratio = | 6969 |
| Max Downward Total Deflection | 0.081 in | Ratio = | 3572 |
| Max Upward Total Deflection | -0.017 in | Ratio = | 16621 |

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Title : Cupertino City Hall
Dsgnr:
Project Desc.:

Project Notes :

Job # M11-040



# Concrete Beam



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 |

## Concrete Beam

**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 |