

Sierra Bonita: Innovative Use of Long Span Metal Deck Slabs and Shored Construction

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INTRODUCTION

Sierra Bonita is a mixed use building providing affordable housing for people living with disabilities. The five story building is located in West Hollywood, CA and was completed in 2010. A 6 inch reinforced metal deck slab system spans 20 feet between supporting steel framing. The supporting framing consists of 43 foot long beams and 60 foot long girders supported by columns at the building perimeter and at the four corners of the central building atrium.

Occupant induced floor vibrations as well as service and creep deflections were taken into account in the design process. Finite element modeling using SAP2000 was used to predict the response of the structure from walking induced vibrations. Testing was later performed during construction in conjunction with The Pennsylvania State University to confirm the acceptability of the structural response. Service and creep deflections were mitigated using slab reinforcement, camber, wood joist shores and loose steel shores. This paper presents our approach to the design of this system, including modeling techniques; use of construction sequencing and shoring; and a comparison of in service vs. design results.

STRUCTURAL SYSTEM SELECTION

In pursuit of the project goals, a structural system with a shallow floor depth and a minimal wall thickness or column size would be required. The system would also need to accommodate the retail and parking occupancies on the lower level. In residential buildings, these goals are typically achieved with a concrete flat plate. A flat plate system, however, would likely have required a transfer level at the second floor and the additional weight would also increase the seismic base shear of the building and thus the cost.

In California, low rise residential buildings typically consist of wood framing supported by bearing walls. While this system provides a fairly shallow floor depth at low cost and weight, it would also require a transfer podium structure to accommodate the lower level occupancy. A hybrid steel frame/wood joist solution combined the benefits of the wood bearing wall system with the flexibility of a steel system. This system, however, provided a modest 7'-8" ceiling height and would not have ideal fire performance characteristics.

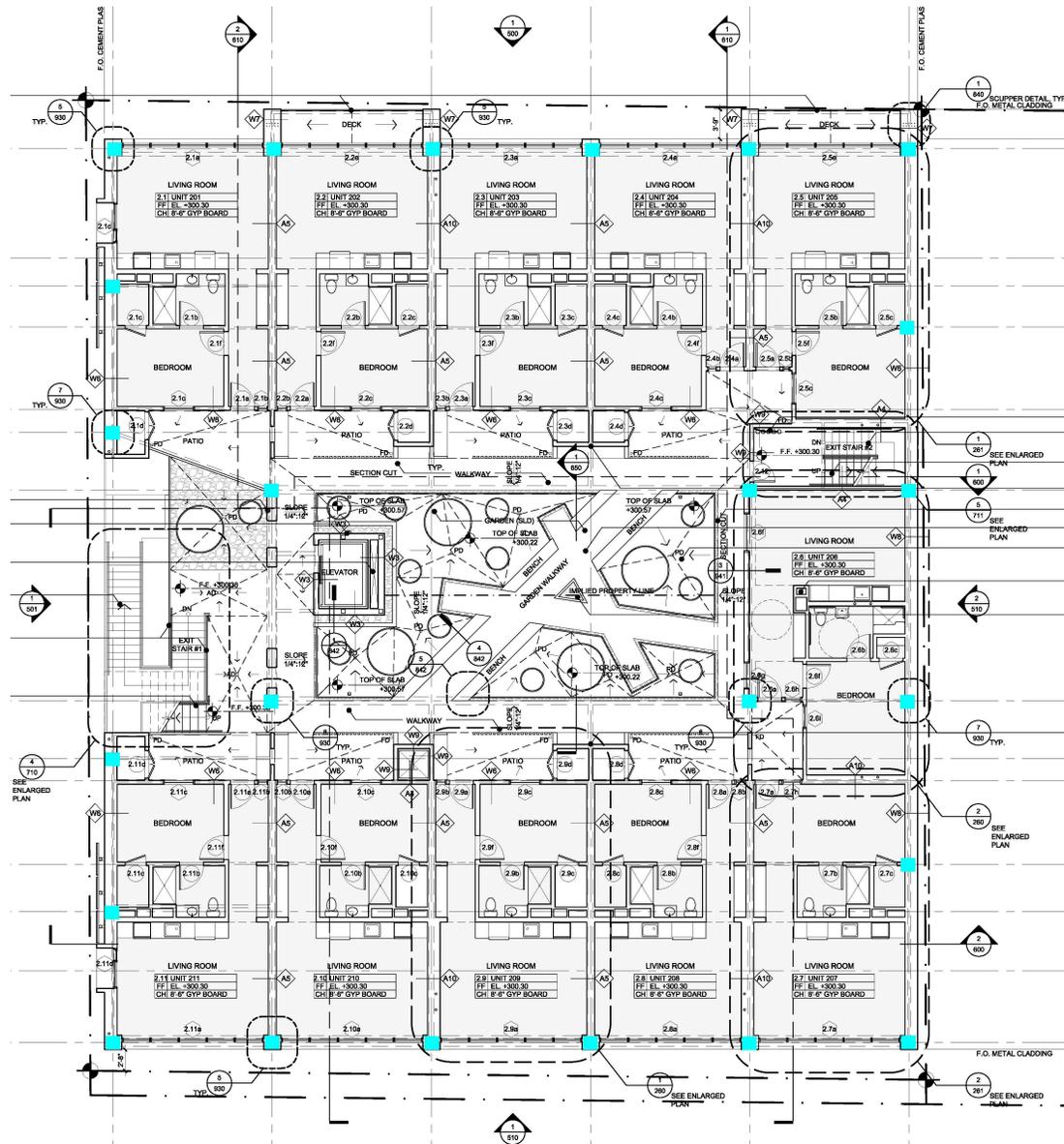


Figure 1
 Typical Residential Floor Layout

An ideal solution, combining the flexibility and light weight of steel framing with the shallow floor depth of a concrete flat plate was decided upon. Steel beams were located in line with the demising partitions between adjacent apartments. The beams span between the building perimeter and the girders around the central atrium. Four corner columns define the central atrium space resulting in a structural system with 20 foot slab spans, 43 foot beam spans and 60 foot girder spans. This system required particularly special attention to deflection and vibration performance.

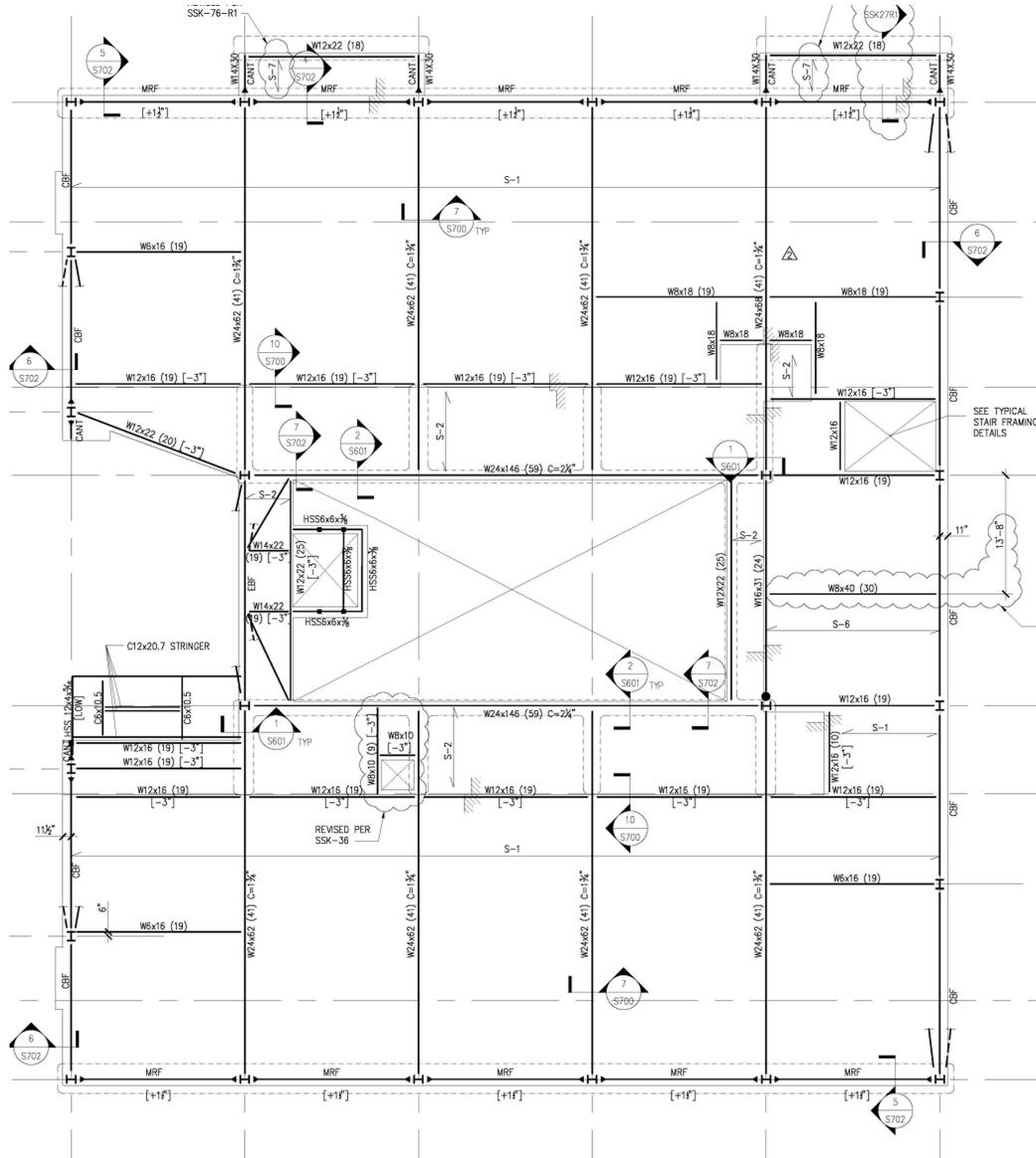


Figure 2
Typical Residential Floor Structural Framing Plan

LONG SPAN SLABS

As a result of the apartment layout, the typical one way slab span of the structure is 20 feet. This is beyond the limits in use for conventional metal deck slabs and required a different design approach. Following ACI 318 guidelines would have required a maximum slab thickness of 10". In previous projects it was found that the performance of reinforced metal deck slabs outperformed this empirical guide line. Performance of metal deck slabs were found to be satisfactory up to a span to depth ratio of $L/43$. Building on this experience, we selected a span ratio of $L/40$ for the Sierra Bonita Project.

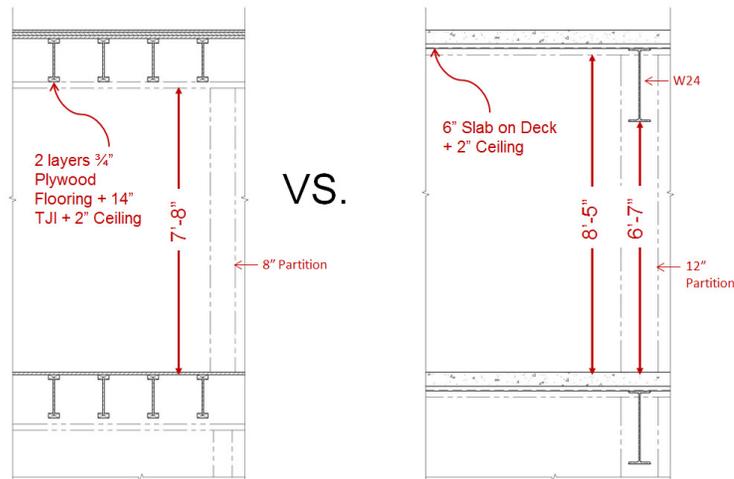


Figure 3
Comparison of Steel/Wood Hybrid System vs. Long Span Slab System

To determine the deflection characteristics of the slab, two methods were used. Initial hand calculations were performed using the effective moment of inertia method in ACI 318. Long term creep deflections were accounted for with a standard multiplier of 2 on the long term dead load. Modulus of rupture of the concrete was assumed to be $7.5\sqrt{f'c}$. Using this method, long term dead load deflection plus live load deflection was found to be $L/339$ and total deflection including long term dead load deflection was found to be $L/226$. Although these results exceed the recommended deflection limitation of $L/360$ and $L/240$, the findings were preliminary and were used for input into a more accurate finite element model that would include the contribution of the framing members and steel reinforcement.

Following the completion of the hand calculations, a SAFE finite element model was constructed that would accurately capture the stiffness contribution of the other structural elements. Several iterations were performed, adjusting the top and bottom steel and concrete strength until the predicted total deflections were within the long term live and total load deflection limits of $L/360$ and $L/240$ respectively. The contribution of the metal deck was modeled as an equivalent layer of reinforcing steel at the mid depth of the metal deck.

Once deflections were found to be in accordance with the above limitations the slab reinforcing was checked for strength. Using 6000psi concrete, the 1 1/2" 20 gauge metal deck was found to have sufficient strength to function as the bottom reinforcement of the slab. A nominal amount of top reinforcing consisting of #5@12" was required for the negative moment over the beams. End spans required additional reinforcing of #6@6". Because UL fire tests only cover metal deck spans up to 17 feet, bottom reinforcing was added to supplement the metal deck.

SHORING AND ADDITIONAL DEFLECTION CONSIDERATIONS

Typically, concrete flat plate construction requires significant amounts of form work and shoring. Additional time and man power is also required to strip form work and re-shore the slabs as work progresses up the building. In lieu of continuous vertical shoring, a method of “self-shoring” and loose shores provided a procedure that would minimize the deflection of the slab under the wet weight of concrete, increase the levelness of the slab pour and maintain a constant thickness of concrete in the slab. By eliminating the majority of posts and/or screw jacks, work men were able to proceed with further trades on the floor immediately below a concrete pour.

Shoring of the slab consisted of two layers of wood framing spanning between the bottom flanges of the beams. Shims of varying thickness were installed between the top layer of framing and the metal deck to provide a slight slab camber. The bottom layer of framing consisted of off the shelf LVL lumber. Loose Shores at beams and girders were also used in combination with camber to control the deflection of the framing. The gap between the shore and the framing member was set equivalent to the non-composite deflection from the weight of the wet concrete. This allowed the contractor to achieve a level floor with minimal post-composite dead load deflections.



Figure 4: Self Shoring System In Place

VIBRATION DESIGN AND FIELD RESULTS

Due to the long span of the beams, girders and slabs, vibration response of the structure was an important consideration. To determine the vibration response of the structure two methods were used. The first method follows the recommendations in

AISC Design Guide 11 Chapter 4 and the second method estimates the vibration response using a finite element model.

Design Guide 11 estimates the natural frequency and peak acceleration of the floor system by combining a weighted average response of the different “panel” modes of a structure. Typically, the participating panels used in this analysis are

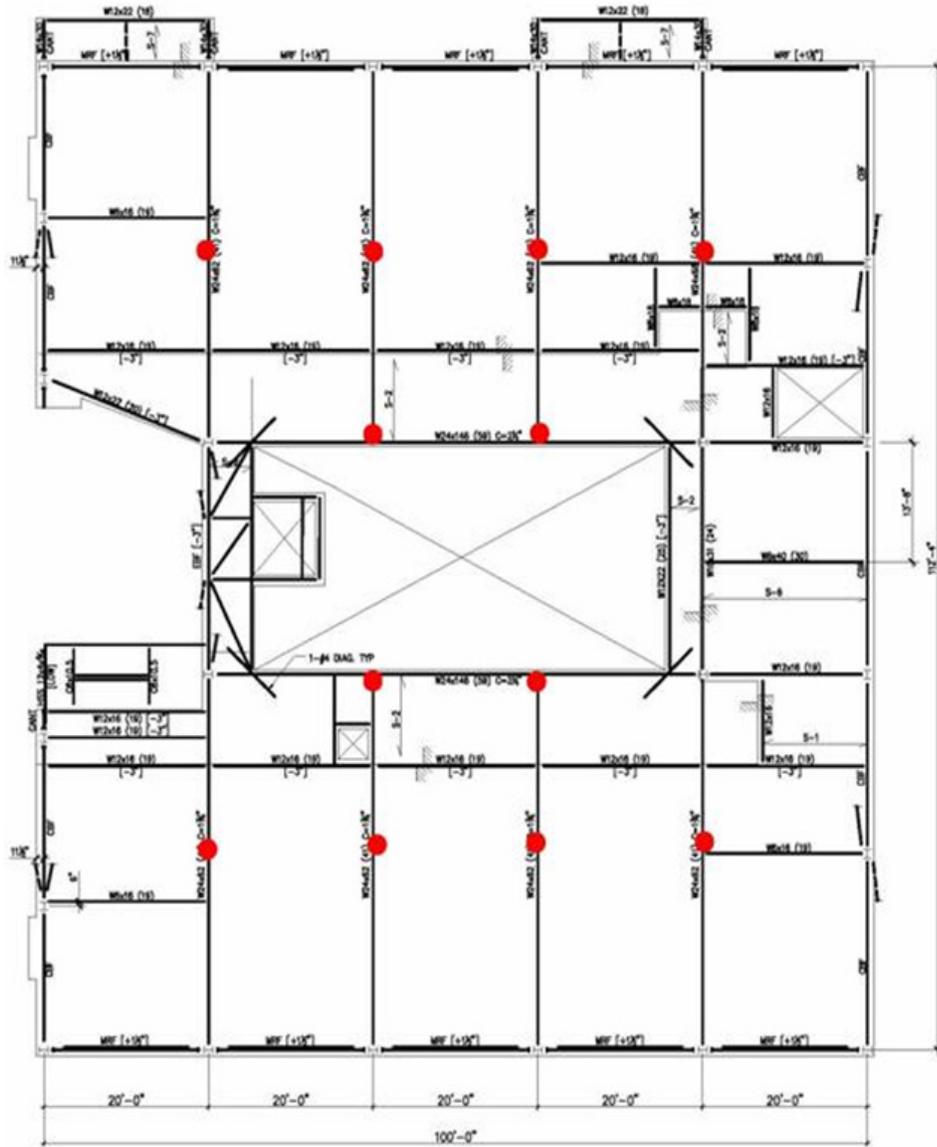


Figure 5: Loose Shore Locations

limited to the beams and girders. In this case, however, due to the long span of the slab, the contribution of the slab panel mode to the overall system response was included. The slab mode is calculated in the same manner as the joist panel mode presented in design guide 11. The results indicated a fairly low frequency for the

typical floor at approximately 2.5Hz. A low frequency response was anticipated due to the long spans of the structural system. The acceleration response was within the maximum recommended acceleration limit of 0.5%g for residential occupancy. It was 0.43%g.

Finite element modeling using SAP was also conducted to verify the results derived from DG 11 hand calculations. The typical floor plate of the entire building with columns was modeled in SAP. Slab stiffness parameters were modified to reflect the stiffness contributions of the steel reinforcing and cracked concrete. To determine the floor vibration response, a forcing function for a single 160 pound person was used. A forcing function with 15 foot fall pulses was used to approximate the input of a person walking across the length of the apartment. For simplicity, the input force was located in the middle of the typical apartment in the middle of the framing bay. This would be the most flexible area of the floor plate. A time history analysis was conducted on all force inputs and the acceleration response spectrums were plotted. The resulting peak acceleration of the time history analysis was 0.34%g with 5% damping. This was deemed to be an acceptable level of vibration response and was within the ISO curve in Design Guide 11. Additional areas were analyzed in a similar manner to confirm these results.

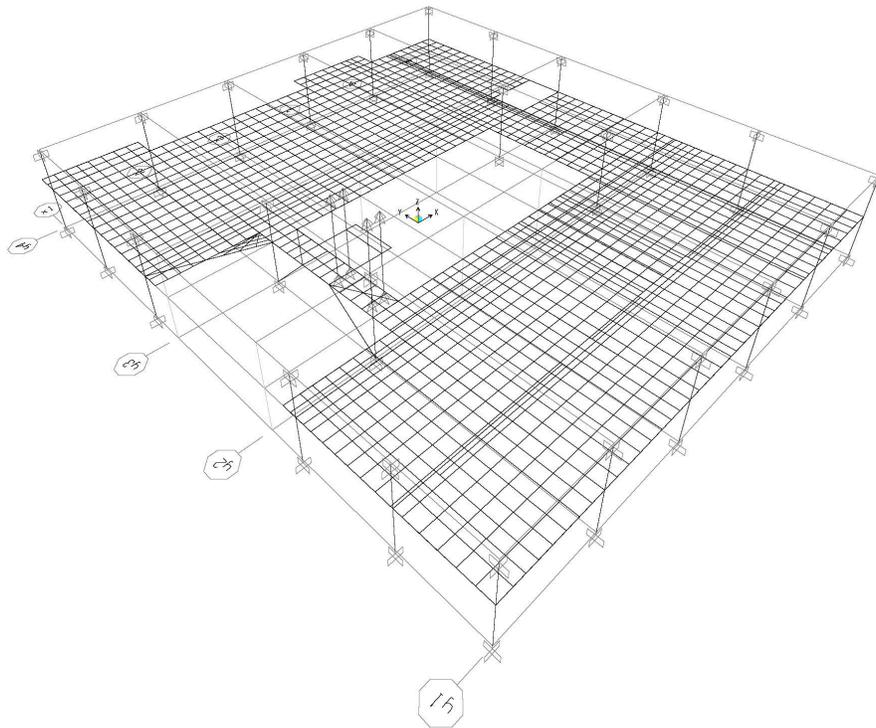


Figure 6
SAP Finite Element Model

During construction, in conjunction with The Pennsylvania State University, we visited the site to conduct vibration testing of the in place floor system. Testing was conducted at an end bay and an interior bay as well as on different floors within

the building. An array of accelerometers was located within the framing bay being tested and heel-drop and walking tests were performed. The field measurements indicated a floor system that would not be expected to have objectionable levels of floor vibration due to walking. The damping affect of the partition walls were found to be larger than what was used for the finite element model. The findings of this testing is being presented in a separate paper.

CONCLUSIONS

Residential construction requires that special attention be paid to deflection and the vibration response of structures. For this building an innovative structural system consisting of long span reinforced concrete slabs and beams was used. Occupant induced floor vibrations as well as service and creep deflections were taken into account in the design process. Finite element modeling using SAP and SAFE was used to predict the response of the structure from walking induced vibrations and long term deflections. Testing was later performed during construction in conjunction with Penn State to confirm the acceptability of the structural vibration response. Service and creep deflections were found to be acceptable, because of the added stiffness contribution of the reinforcing steel. A combination of camber, wood joist “self” shores and loose steel shores to achieve a level finished floor with minimal post composite dead load deflection.

REFERENCES

Hanagan, L.M., Raebel, C.H., Trethewey, M.W. Dynamic Measurements of In-Place Steel Floors to Assess Vibration Performance. *Journal of Performance of Constructed Facilities*. August 2003.

Murray, T. A., Allen, D., and Ungar, E (1997). *AISC Design Guide 11: Floor Vibration Due to Human Activity*, AISC.

Tipping, Eldon. "Quality Construction of Suspended Floors." *Concrete Construction*. February 2006

American Concrete Institute. (2005). *Building Code Requirements for Structural Concrete*

American Institute of Steel Construction. (2005). *Steel Construction Manual*

American Society of Civil Engineers. (1991). *ASCE 3-91: Standard for the Structural Design of Composite Slabs*