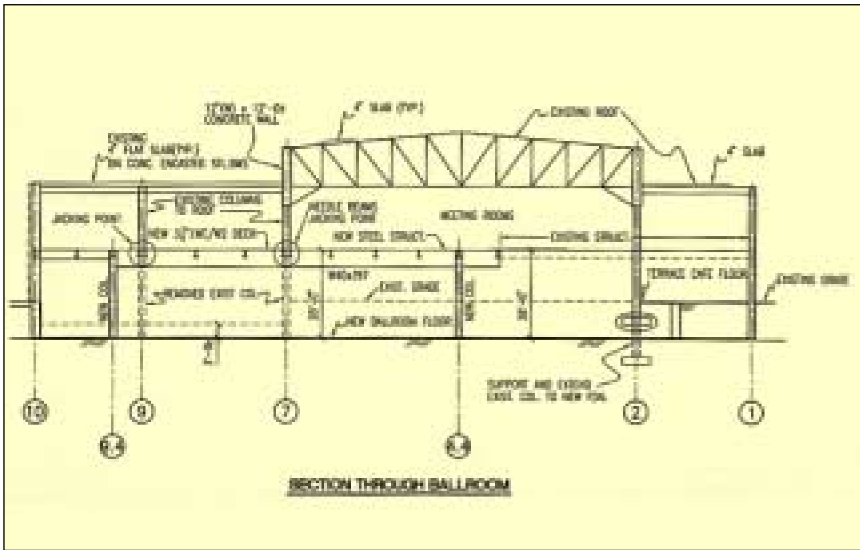
Public Market to Ballroom

A Cinderella Transformation in Sacramento

By D. Scott Kennedy, P. Eng.



The well-known California architect, Julia Morgan, designed the Public Market Building in Sacramento in 1920. The Public Market Building lies adjacent to the new Sheraton Grand Hotel in Sacramento and will house the ballroom and meeting rooms for the hotel. This article addresses the ways and means by which the transfer of the existing roof structure column loads to the new second floor level was economically, safely and simply achieved.

The problem facing the development team was how to achieve the hotel's requirement of an 80′ wide, 28′ high ballroom within the confines of the existing structure. Bruce Rusher, Chief Estimator of Hensel Phelps Construction Company, brought in Scott Kennedy of Kensco Engineering Ltd., Vancouver, B.C., to devise a workable construction scheme and budget for this portion of the project and work with the design team to develop a system which would most economically meet the goals of the construction team.

The initial concept reviewed by Kensco and Hensel Phelps showed the steel grid of the second floor directly supporting twenty-four roof columns of the existing structure, the shoring planned during construction. This approach required extensive temporary shoring to the existing structure during construction at a cost of several hundred thousand dollars. Bruce and I suggested to the design team that an entirely different approach be taken. We reasoned that if the entire second floor of the structure could be built in advance of the removal of the lower floor columns, then the second floor could be used as shoring.

We proposed that needle beams, which would be free to deflect under the weight of the concrete floor, be placed on either side of the existing roof columns. Once the floor was poured, the load from the twenty-four roof columns could then be transferred to the composite beams and girders and secured prior to the lower columns being removed. The second floor would in effect become the shoring. We proposed that a simple jacking system could be developed by using four 1 ¼″ diameter A325 bolts per column (see detail). Initially there was some concern that bolt loads would be unreliable and difficult to gauge. It was pointed out that in other similar situations extensive hydraulic jacking systems had been used with load cells and complex temporary supports. We suggested that the bolts be tested for torque versus load to establish the bolt load in kips. Signet Testing of Sacramento confirmed a straight-

Interior of the Public market building showing the remaining structure after demolition of the existing second level and basement.

Interior view after installation of new second floor steel and steel deck.

Columns cleaned and painted with intumescent paint. Bases have been grouted and the shoes and bolts hidden under a concrete cover.

line relationship between torque and load.

Detailed drawings of the original structure were not available, so Brian Hill, Project Manager with Hensel Phelps, arranged an on-site survey to establish the dead weight and construction type of the existing roof structure. At this stage, input came from Gregg Haskell of Haskell Engineering, who was responsible for all temporary on-site work and shoring, and by Scott Koehler of Koehler Steel, the fabricator and erector. They approved our proposed scheme and enthusiastically suggested modifications.

We discovered that the existing roof structure consisted of a four to six inch thick one way slab supported by simple concrete encased steel beams and girders with 75′ steel roof trusses over the central raised area (see section). A 12″ wide by 12′ deep poured in place concrete wall with window openings surrounded the entire roof area and provided a rigid box around this portion of the structure. The calculated deflections due to the tributary dead load of the roof structure at the second floor jacking points varied from 2 ¼″ to ¼″, so it was necessary to pre-load each jacking point to this load prior to removing the supporting columns. Failure to pre-load columns would cause the stiff box-like roof structure to bridge between "hard" jacking points sharing only a portion of the load with softer jacking points in proportion to their relative stiffness. We recognized this would seriously overstress those columns at "hard" jacking points only designed to take the tributary roof load.

Once the existing structure had been surveyed, the point loads from the roof could then be calculated at each column. The maximum loads were 84 kips. To transfer this load to the second floor beams, a torque of only 270 lb./ft. per bolt was required. We were pleased that our original assumption was correct in that only a small torque on four bolts would be required to support 84,000 pounds per column.

The cambers for all the beams and girders affected by the roof column loads were then calculated. We calculated the cambers as the sum of the initial deflection due to the concrete floor dead load plus the composite deflection due to the

Julia Morgan (1872 - 1957) was a truly remarkable woman. Decades ahead of her time, she pursued an undergraduate degree in civil engineering and then traveled to Paris in 1896 to study architecture at the École National des Beaux Arts. Her most famous structures are Hearst Castle and The Fairmont Hotel in San Francisco; however, she also designed several residential buildings in the Berkeley area where she favored a California style of architecture, including exposed beams, extensive use of shingles and earth tones and horizontal lines that blended into the landscape. These buildings, with distinct arts and crafts attributes, could easily have been created today. With Julia Morgan looking over our shoulders, we heeded the plea of the City of Sacramento Preservation Department to "please look after our building."

roof column dead load. This camber, if correctly applied, would give us a flat floor once all dead loads were in place. Kensco used Ram Steel beam design in calculating the cambers applying the concrete load as a construction dead load and the roof point loads as a dead load to the composite beams and girders.

Some load points were torqued three times to achieve their final load and deflection as their initially applied load was relieved by beam or girder deflections caused by the application of loads in other locations. The layout of the column loads and the deflections caused were studied to establish the most effective order of jacking. The maximum calculated deflection due to roof loads was 2 ¼″ at column #14. The initial deflection at this point due to the dead weight of concrete was 1 ¼″, giving a total calculated deflection of 3 ½″. Hardip Pannu of Middlebrook and Louie cautioned that in his experience the calculated deflection did not occur as predicted and that we should use a lesser (75%) camber. His advice proved correct.

On the day that the jacking program was due to commence, we arrived at the site with some fears that unexpected problems could occur. We anticipated that the jacking could take up to four days; however it took four ironworkers only four hours to torque the bolts on all twenty-four columns to their required loads. The columns were welded off and secured before the removal of the lower column sections. No measurable deflections were recorded upon cutting loose and removing the lower columns. The whole operation cost only $8,700 in materials and labor, a complete success for all concerned!

In conclusion, although we used sophisticated structural design software, lasers, load cells and hydraulic load cells, the key to the success of our project was the application of high school physics to a simple bolting function. The result was an inexpensive, safe, controlled solution to what at first appeared to be a difficult expensive construction problem.

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Structural Engineer, Shoring & Temporary Work:
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Software:
RAM Steel