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SEISMIC REHABILITATION OF A LARGE MANUFACTURING PLANT

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ABSTRACT

The objective of this paper is to discuss the seismic rehabilitation design and construction of a major manufacturing building located in California. The seismic rehabilitation of this building is of particular interest since it is a large timber building with over 760,000 square feet under a single, multi-level high-bay roof, and containing over 2 million boardfeet of lumber. The methods employed in conducting the seismic analysis included constructing two-dimensional computer models of the structure and performing static lateral force analyses of trussed moment frames in one direction, and chevron braced frames in the other direction. In order to reduce excessive stress levels relative to current building code requirements and desired seismic performance, a retrofit strengthening design was developed. The goal of the retrofit was provide structural life-safety in the event of strong ground shaking and avoid significant business interruption during the construction. Construction took place during the second and third shifts and sometimes on weekends, without any significant disruption to the manufacturing process.

KEYWORDS

Seismic Rehabilitation, Manufacturing Plant, Heavy Timber Truss, Moment Frames, Braced Frames, Two-Dimensional Modeling, Life-Safety, Lumber Grading, Minimal Business Interruption

SEISMIC REHABILITATION OF A LARGE MANUFACTURING PLANT

Description

The building is a one-story, multi-level high-bay, heavy timber structure. Built in the 1940s, the main manufacturing area is approximately 1,600 feet long and 350 feet wide. Attached to and on each side of the main manufacturing area, in the longitudinal direction, there is a one-story wood framed office area. The building occupies approximately 760,000 square feet of floor space. It is also divided into four separate structures to minimize thermal stresses and allow for expansion. There are a total of 84 transverse timber moment frames divided into four bays spanning up to 150 feet (see Figure 1). The structural system consists of trussed moment frames in the transverse direction and braced frames in the longitudinal direction. The building was originally designed to the 1940 Uniform Building Code (UBC).

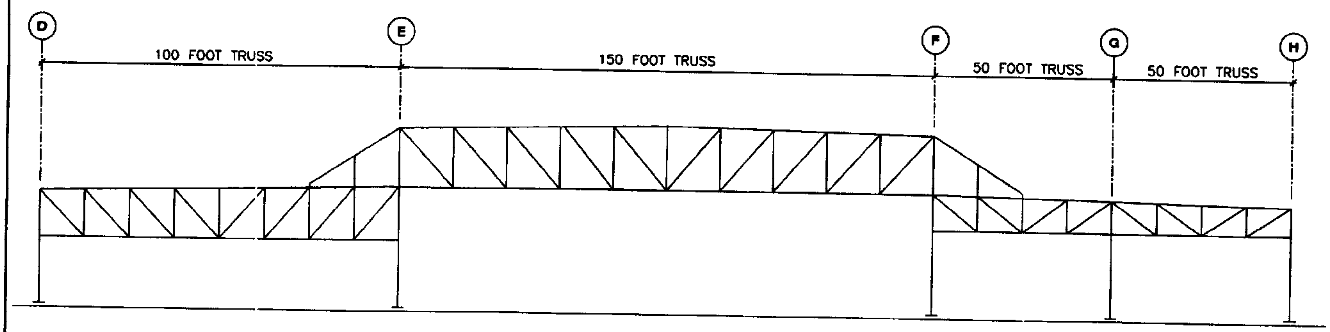

Figure 1 - Elevation of Truss

Analysis

The purpose of the project was to evaluate the building's primary vertical and lateral load carrying systems, and to develop an economical retrofit to correct identified deficiencies. The retrofit design was in accordance with the 1991 UBC which was the minimum requirement of the local building department.

Ten of the eighty-four truss lines were randomly sampled to evaluate the actual gravity loads and code prescribed live loads. Upon completion of the initial site investigation, member condition survey, and stress analyses for each truss, it became apparent that actual applied loads (i.e. roof material and structure weights, and weight of piping, electrical, HVAC, etc.) and member stresses varied by only a few percent from truss to truss. Further, it was found that the various timber members were very similar in size and grade from truss to truss. Therefore, only eight of the eighty-four truss frames were analyzed (a total of eight truss configurations existed). The longitudinal bracing was also very similar and only two of the four braced frames were analyzed. Once deficiencies were identified, retrofits for individual members were developed.

In order to establish allowable stresses based on the existing timber conditions, a timber consultant was engaged to review member conditions for ten randomly sampled trusses and twenty individual truss members. Members had their paint removed and were closely reviewed in order to establish an approximate "grade" (e.g. Select Structural). The grade was then used to determine allowable stresses. It was found from the grading process that most truss members and columns, with a few exceptions, were in very good condition, and qualified for a very high grade, and thus high allowable stresses.

The demand/capacity analyses, (i.e., problem identification process) for vertical and lateral loads based on the aforementioned allowable stresses indicated that most of the truss members conformed to the 1991 UBC and did not require retrofit. The exceptions were that all building columns and just a few truss members were deficient for the seismic load case.

Model Development Two-dimensional transverse and longitudinal models were developed to accurately emulate actual load conditions. In order to have as accurate a model as possible, an in-depth site investigation was performed to determine the actual dead loads on the building since mechanical and electrical systems have been modified and added over the last 50 years. A random sample of ten of the eighty-four truss lines were visually evaluated to determine the actual gravity loads and code prescribed live loads. The types of systems surveyed during the site investigation consisted of: roof material, structure weights, mechanical platforms, catwalks, weight of piping, electrical conduit, mechanical units, suspended ceilings, and partition walls.

Once the bounding load conditions were determined, they were inputted into the computer models. The following load cases were used as a basis for the strengthening design:

Case 1: Dead Load + Live Load

Case 2: Dead Load + Seismic Load (+x or +y direction)

Case 3: Dead Load + Seismic Load (-x or -y direction)

Design Code The code for which strengthening procedures were developed for this building was the 1991 UBC. The two major differences in the original design versus the 1991 UBC are as follows:

- Roof Live Load = 12 psf (originally designed for 20 psf)
- Design Base Shear, $V = \frac{ZICW}{Rw} = 0.22W$ where:

Z = 0.4 (Seismic Zone Factor)

I = 1.0 (Importance Factor)

C = 2.75 (Numerical Coefficient)

Rw = 5 (Numerical Coefficient)

W = Total Dead Load

(The original seismic design utilized a base shear of 0.08 times the total dead load plus half the live load)

Methodology for Determination of Allowable Timber Stresses Original design drawings listed the following information for timber elements:

Douglas Fir, Dry Locations

Tension and Bending = 1200 psi

Compression = 1100 psi

Young's Modulus (E) = 1,600,000 psi

Horizontal Shear = 100 psi (for ≥ 5" thick)

120 psi (for < 5" thick)

The above specifications translated into a grade of No. 1 per the 1940 UBC.

An evaluation of the existing wood members throughout the structure was performed to either verify the information provided on the original drawings or to furnish new grades based on a present day assessment. Ultimately, allowable stresses for each of the different member types was obtained. Member types included top chords, bottom chords, diagonal truss members, vertical truss members, and columns. The timber grade assessment was performed as follows:

1. A random sample of twenty members (four of each member type) was chosen for close inspection. The samples included members from each of the various

types of transverse trusses. Selected members were stripped of paint to facilitate a more thorough evaluation of individual characteristics.

2. A timber expert was engaged to survey the stripped members and assign a "best possible observed" grade based upon existing conditions.

3. Timber experts extrapolated information (from Item 2 above) to the remaining structure, thus establishing grades for each member type.

Recommended Grades and Allowable Stresses Based on the lumber grades received from the timber experts, allowable stresses were taken from the 1991 UBC. This was considered acceptable since grading procedures and criteria had remained essentially the same over the last 50 years. Thus, it was fair to say that a member conforming to a lumber grade of No. 1 in 1940s would have been graded as No. 1 today as well. Although grading procedures have stayed constant over the years, allowable stresses associated with those grades have changed considerably. Thus, an element graded No. 1 would be assigned different allowable stresses per the 1940 UBC then the 1991 UBC. The following table lists member types, recommended grades (per timber evaluations), and allowable stresses per the 1991 UBC.

Member Type	Member Size	Recommended Grade	Allowable Stresses Compr.	Allowable Stresses Tension	Allowable Stresses Bending
Top and Bottom Chords					
	(6x)	Select Struct.	1100 psi	1100 psi	1600 psi
	(3x, 4x)	Select Struct.	1400 psi	1200 psi	1800 psi
Diagonals					
	(6x, 8x)	Select Struct.	1100 psi	1100 psi	1600 psi
	(3x, 4x)	Select Struct.	1400 psi	1200 psi	1800 psi
Verticals					
	(6x, 8x)	No. 2	475 psi	475 psi	700 psi
	(3x)	No. 2	1050 psi	650 psi	1250 psi
Columns					
	(14x)	Select Struct.	1150 psi	1000 psi	1500 psi

Timber Connector Capacities Almost all of the connections employ the use of 4" diameter split ring timber connectors with 7/8" diameter bolts as indicated on the original construction drawings. However, no mention was made as to the type and/or the allowable capacities of the split rings. Therefore, capacities were obtained from the Wood Handbook (U.S. Department of Agriculture, September 1935), as referenced in the 1940 UBC. Split ring data in the Wood Handbook was based on 4" diameter split rings used in concert with 3/4" diameter bolts. Tuchscherer Split Ring (type of connector) capacities were used as they provided the most conservative values:

p = 12,000 lbs., safe load (parallel to grain)

q = 8400 lbs., safe load (perpendicular to grain)

Split ring values per the National Design Specification (National Forest Products Association, 1991) for wood, as referenced by the 1991 UBC, are as follows:

p = 10,000 lbs., safe load (parallel to grain)

q = 6960 lbs., safe load (perpendicular to grain)

Because it is the diameter of the split ring that determines the capacity of the connection, and because the above loads (for both 1940 and 1991) represent maximum split ring capacities, the use of a 7/8" diameter bolt (in-situ) instead of a 3/4" diameter bolt would not provide a stronger connection because the split ring will still fail at the same critical load. Thus, the values listed above were considered good approximations of the capacities of the split rings.

Findings

The above noted load cases were applied to both the transverse and longitudinal frame computer models to establish the structural integrity of the configurations based on the 1991 UBC. Member forces, as determined by the computer analysis, were compared to member capacities to locate overstresses. Demand-capacity ratios (D/C) were calculated for each member based on the recommended allowable stresses as provided by the timber experts. The following table summarizes the results for each of the three load cases:

Level of Overstress	Number of Overstressed Transverse Truss Members		
	Case #1	Case #2	Case #3
Up to 30%	9	5	4
30% to 100%	3	5	9
Over 100%	3	3	1
Total	15	13	14

No significant member overstresses were encountered in the longitudinal frame analysis.

In addition, member forces were transferred into connections to determine if allowable capacities were sufficient. Load Case #2 was used to determine the connection overstresses for the transverse truss. One bottom chord-to-diagonal connection was found to be 15% overstressed. Three chord-to-column connections were overstressed as much as 122% and two chord-to-column connections exhibited overstresses as high as 55%.

In the longitudinal frame, several top purlin-to-column connections (which do not appear to incorporate split rings) exhibited overstresses up to 156% due to Load Case #2. One strut-to-column connection, with a bent steel plate, on was also found to be overstressed by nearly 50%.

Retrofit Design

In light of the above described results, a seismic retrofit of the structure was developed to correct identified deficiencies. The major deficiencies, the column overstresses, were corrected by adding new steel columns and foundation reinforcement adjacent to existing interior columns that support the 150 foot span trusses. These new steel columns served two purposes: they reduced the stresses in the columns supporting the 150' span to an acceptable level, and due to their stiffness, drew enough load from the outer columns that supported the other trusses to mitigate overstresses in them as well. The corrective measures for the few truss members found deficient consist largely of applying (via nailing, bolting or gluing) new lumber between existing members to stiffen and strengthen them.

Typical Column Retrofit The major deficiencies discovered in the transverse trusses were that the columns were extremely overstressed due mainly to bending forces induced by lateral loads (Load Cases #2 and #3). The philosophy behind the seismic strengthening was to stiffen interior columns, thus dragging more load from the exterior columns, reduce deflections to an acceptable level, and strengthen any interior truss members/connections such that capacities exceeded induced loads.

Stiffening interior columns (along lines E and F) was accomplished by installing two steel wide flange columns, one on either side of each of the existing 14x22 wood columns. Loads were transferred from the existing trusses into new columns via steel tube/steel plate bolted and welded connections as shown in Figure 2. As these were the only main connections to the columns, no significant vertical load (aside from the new steel column self weight) was supported by the new columns. Lateral loads would travel the length of the new columns to the bottom where they will be taken out through shear at the base.

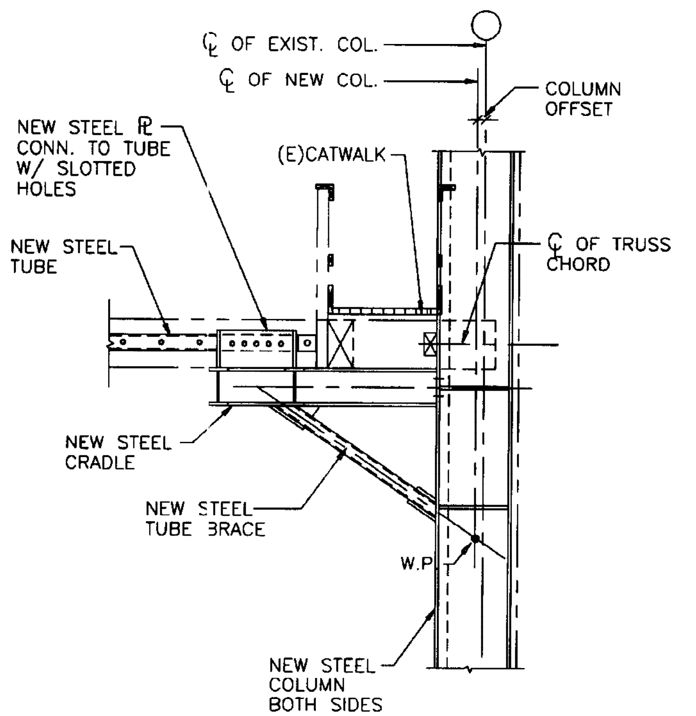

Figure 2 - Column to Truss Connection

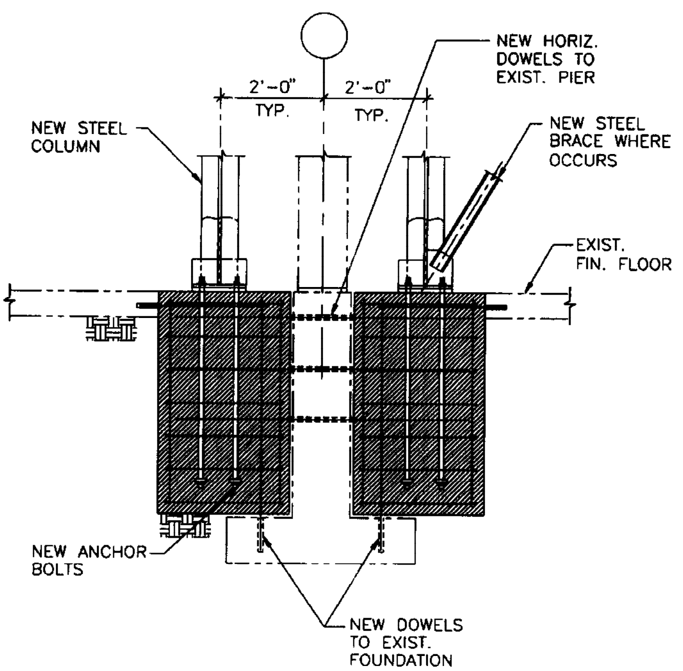

Figure 3 - Column to Foundation Detail

The new columns extend to the finish floor and attach to a new concrete pier (atop the existing footing) via embedded anchor bolts and steel plates as shown in Figure 3. Additional reinforced concrete was installed around the base of the columns surrounding the existing pier. New reinforcing bars were doweled into the existing pier and footing to encourage monolithic action.

The new columns required modification of the longitudinal braced frames as well. It would have been very time consuming and cost prohibitive, after the installation of new steel columns, to try and reuse the existing wood diagonal and strut members. Therefore, new steel members were installed, in place of the existing wood, and attached directly to the new steel columns. The two steel columns were linked together at various existing strut levels in order to transfer the longitudinal lateral loads to the longitudinal braced frames.

Specialized Column Retrofit Of the 168 column locations, 16 are located either at an end bay or adjacent to an expansion joint. In any case, these locations were not conducive to the installation of two new steel wide flange columns due to a lack of space between either two existing adjacent columns or an existing column and the outside wall. Instead, two narrower built-up steel plate and angle columns were erected on either side of the existing wood columns. The strengthening concept was the same as for the two wide flange columns, that is, to resist the lateral forces induced on the truss. As for the typical scheme, this procedure included the installation of additional reinforced concrete around the existing pier (atop the existing footing).

Interior Truss Member Retrofit Interior truss members that required strengthening were typically limited to the vertical or diagonal truss members. The strengthening of these members was typically accomplished by installing additional wood members between existing members to increase stiffness and cross-sectional area of the member as shown in Figure 4. As these members are predominantly subjected to axial loads only, an increase in cross-sectional area, with adequate connections, typically solved any tensile or compressive overstress problems.

For the few members overstressed in bending, as were the vertical members in the transition zones, installing steel plates on the sides of the member to increase the moment of inertia was used to alleviate overstresses.

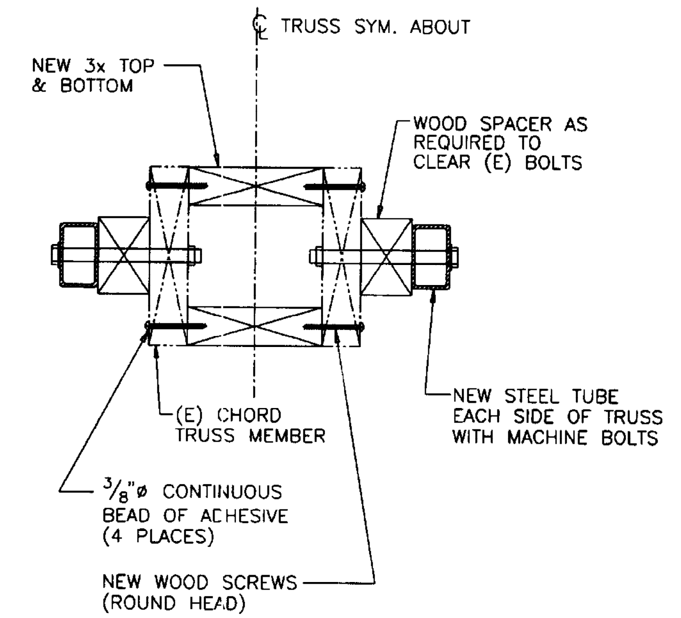

Figure 4- Member Strengthening Detail

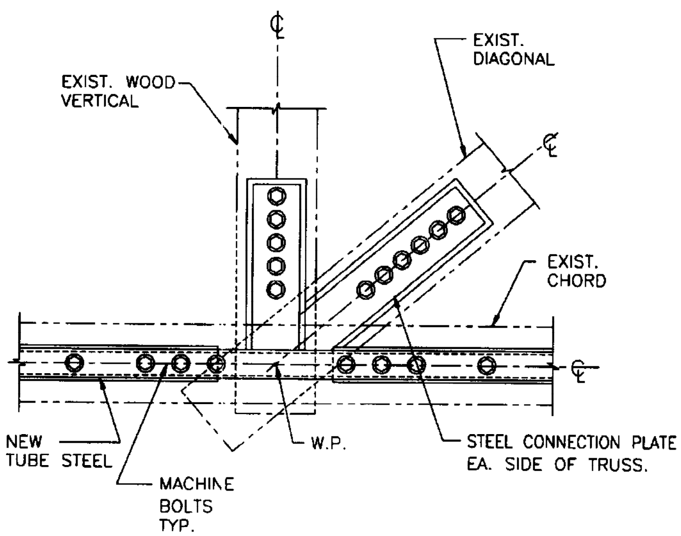

Figure 5- Connection Strengthening Detail

Connection Retrofit The bulk of the connection overstresses as previously listed occurred at chord-to-column and chord-to-strut connections. To transfer loads effectively into the new steel columns, new connections between the chords and the columns were developed. As shown in Figure 2, the new column-to-truss connection transfers lateral loads from the existing chords directly into the new steel columns, thus bypassing the original chord-to-wood column connection. This portion of the retrofit eliminated the overstress problem at the chord-to-column connection. Chord-to-web member connections which were overstressed were typically strengthened by the addition of steel plates designed to transfer member loads to the new tube steel chord members (see Figure 5).

Similarly, for longitudinal strut-to-column connections, the problem was eliminated with the new steel column and brace strengthening plan. As stated earlier, the existing longitudinal wood struts and diagonals were replaced with new steel members. The connection of steel struts/braces to steel columns were adequately designed such that no overstresses occurred.

Retrofit Induced Overstresses and Strengthening Procedures In any structure, loads tend to flow toward and be resisted by the stiffest elements assuming there is an acceptable path for the loads to take. For this building, with the addition of the steel columns, interior column lines E and F became the stiffest elements of each truss. As such, forces tended to flow toward these interior columns by dragging loads along members that were likely not designed to resist forces of such magnitudes. In effect, forcing loads along under-designed members resulted in an overstress in some members. Retrofit of the newly created drag lines required a combination of the typical member strengthening with the installation of additional wood members and steel tubes to increase member stiffness and cross-sectional area (see Figure 4).

Retrofit Construction

In order to avoid any significant disruption to the manufacturing process during construction, the retrofit took place during second and third shifts and sometimes on weekends. Major mechanical or electrical relocations which required shut-down of a significant portion of the facility were performed on weekends and holidays.

Some of the constructibility problems faced by the retrofit contractor were:

- Difficulty during the initial fit-up of new steel members with the existing timber structure. Although strengthening details allowed for varied dimension and location of wood members, field measurement and verification of existing conditions proved to be critical.
- Due to limited access, several new steel columns needed to be spliced in order to facilitate erection.
- Site was located on an old dump site where methane gas was present in the soil and required monitoring and venting during footing excavations.
- Existing concrete piers required bracing in order to allow overhead cranes to operate during the footing excavation.
- Manufacturing equipment stands had to be temporarily supported around columns were new foundations were installed. In order to keep manufacturing lines operating, external supports were introduced during retrofit.
- Floor trenches housing electrical conduit and compressed air lines were too close to selected columns and had to be rerouted or eliminated.
- Although shown on the retrofit drawings, various critical mechanical and electrical equipment had to be relocated during construction as they interfered with installation procedures.

Conclusion

The seismic rehabilitation of this major vintage manufacturing plant has shown that such structures can effectively upgraded to substantially reduce earthquake risk without adversely affecting critical, ongoing manufacturing operations. Innovative seismic retrofit systems can be employed which maximizes the existing strength of the structure while utilizing very efficient supplementary structural steel strengthening systems. It is also clear that a partnering relationship between the owner, engineer, and contractor is very important for the success of a project of this magnitude. Proactive communications between the design team, construction group and manufacturing personnel was critical in scheduling and phasing the work to avoid disruption to the manufacturing operations.