STRUCTURAL STEEL PLATED SHEAR WALLS

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SUMMARY

This paper is a presentation of the design criteria, procedure and details of construction used to achieve a structural steel shear wall system for a fifteen-story hospital building built in San Francisco. Introductions to the project include a description, discussion on the rational for selecting the system and a synopsis of the design procedure. The paper concludes with a report on the actual construction of the project, which is nearing completion.

INTRODUCTION

In 1976, the design of a fifteen-story teaching hospital was completed, using structural steel shear walls as the predominant earthquake resisting element. The project, the H.C. Moffitt Hospital Modernization, is located approximately five miles east of the San Andreas Fault, and is sited over a natural indentation into the Franciscan bedrock, overlain by thin layers of clay and sand.

Regulations in effect at the time of design required this facility to be designed by a "dynamic" analysis based on ground motions related to the probable maximum seismic event postulated for a sixty-year period. It was also required by the governing agencies that the maximum credible earthquake be considered. Final design was based on the credible earthquake (8+ Richter magnitude) criteria.

STEEL PLATED SHEAR WALLS

The building has a very irregular shape. The main building raises from a 180 by 216 foot rectangular shape at the ground and first floors (Figure 1) to the nursing tower configuration at the eighth floor (see Figure 2). The transition between the first floor and the tower occurs within the space north of the tower between lines 5 and 7, where there is a six floor transition with each floor generally stepping to the west. Above the seventh floor, the tower, which houses the nursing rooms, rises uniformly to the top of the building. The tower is 76 by 240 feet in dimension including a 26 foot cantilever over an existing building at the east end.

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The lateral resisting system consists of shear walls at lines A, E and J in the north-south direction and lines 2, 4 and 7.8 in the east-west direction. Resistance is also gained by using the concrete walls in the elevator core between lines J and M. The steel plated walls are the transverse walls at lines A (above the sixth floor), E and J and the longitudinal wall at line 4.

California law requires that all hospitals be designed under state jurisdiction using the state-developed design criteria, the Hospital Code. The intent of state control and the provision of the Hospital Code is to minimize damage in major earthquakes to hospital facilities so that they can continue to operate. In order to meet the Code, criteria were developed and implemented that included a "dual system" framing scheme using concrete and steel plate shear walls with a complete ductile moment resisting space frame; a seismic analysis following the response spectrum technique using a site specific response spectrum developed by the Geotechnical Consultants; and continuous full time field inspection as a means of quality control during construction.

The selection of the shear walls and "two-bit" frame system instead of a 100% moment resisting frame was based on several factors: First, meeting the intent of the Hospital Code dictates a rigid system to restrict story-to-story drift and protect non-structural elements within the building. Second, the 13 feet story height dictated by the adjacent existing hospital together with the many ducts, pipes and other mechanical and electrical items in the ceiling space left insufficient space above the ceilings for horizontal framing members of the depth required for the needed rigidity in a moment resisting frame. A braced frame system was investigated but with the space requirements and difficulty of connections for such high lateral forces, it proved to be impractical. This left shear walls as the only practical solution.

The 25% moment resisting frame was included in the preliminary design because it was required by all Codes in force at the time and was required by the early drafts of the Hospital Regulations. Although the requirement was deleted from the final Regulations, the 25% frame was retained in the final design to provide the added safety of a second line of defense. The added cost was minimal.

The dynamic analysis was based on response spectra with peak accelerations of 180% of gravity. Prior to converting the elastic response spectrum into design data, reduction factors were assigned. These amounted to a 20% reduction factor for the duration of loading, redundancy and other miscellaneous effects. In addition, a reduction factor was used to reduce theoretical elastic response as permitted by the regulations. For this, a ductility factor, or mu, of 3 was used with the equivalent energy factor of $\sqrt{2\mu-1}$. This resulted in a combined reduction to 36% of the theoretical elastic response. After taking all the reductions, the analysis resulted in base shears of 0.263 x gravity and 0.288 x gravity in the north-south and east-west direction, respectively.

The combination of force level and allowable stresses would have required shear wall thicknesses of over 4 feet if the walls were of reinforced concrete only. This would have been unacceptable architecturally and the added weight would have increased the design forces substantially. Because of this, it became necessary to introduce solid structural steel plate into the principal walls to resist the high shears. The plates are enclosed in concrete to provide stiffening against plate buckling.

In general, each of the major shear walls on lines A, E, J and 4 exhibit two predominant patterns of openings. Below the seventh floor, the openings are randomly located, due to the great variation of space usage. Above this level, the opening patterns are very regular, almost cutting any one wall into a series of linked shear walls.

Each steel plated wall is composed of several elements including columns, welded girders, steel wall plates and trim members as shown on the isometric section Figure 3. The columns (not shown) are either standard WF sections (W14x730) or special built-up H columns, weighing up to 1,042 pounds per foot. Plate girders are generally 49" deep, with either 9 or 14 inch flanges. Wall plates are structural steel plates extending vertically between plate girders and horizontally between columns, trim members, etc. Wall plates range in thickness between 3/8" and 1". Trim members are either steel plates or rolled WF sections. Major trim plates, in general, are used to bound major openings and are centered on the wall plates. Minor trim plates are generally used to bound mechanical openings in girder webs. Whenever possible, a single trim plate was used at each edge of the opening. This allowed the horizontal and vertical trim plates to be placed on opposite sides of the web, thus avoiding intersecting plates.

This building has a 13'-0" story height. Combined with this is the mechanical, plumbing and electrical requirements to route the ducts, pipes, etc., through the spandrels above the corridor space. For this reason, a built-up girder with a depth of 49 inches was selected. This allowed for the sills and heads of the corridor openings to be trimmed with the girder flanges.

The wall plates were sized to take the total shear forces in the wall. The concrete cover, varying from five to fifteen inches on each side of the plate, provided stiffness to the structure and was reinforced to the extent that it would be compatible with the steel in terms of strain. The concrete was also used to stiffen the steel plate. Number 3 ties at two feet on center each way were used to tie together the concrete walls at each side of the plate. Figure 4 illustrates a section through the plated wall.

Both the architectural openings and the penetrations through the girders were trimmed. The major trim plates at the architectural openings were connected through the girder flanges using full penetration butt welds. Whenever possible at the girder web penetrations, the horizontal and vertical trim plates were placed on the opposite sides of the web, thus avoiding intersecting plates. In areas where girder penetrations took most of the web, a vierendell truss made up of wide flange members was employed.

As with any tall shear walled structure, overturning became a critical factor in the analysis, especially when considering the high shear strength of the steel walls. The high overturning forces from the analysis resulted in tremendous uplift forces at the ends of the principal shear walls. For example, at the south ends of the shear wall on line E, overturning forces amounted to 6100 kips with net uplift of 3700 kips after deducting dead load. Net uplift at the north end of the same wall mounted to 4000 kips. Comparable forces occur at the ends of the walls on lines J and 4. To resist these forces, 5 foot diameter caissons extend as much as 35 feet into bedrock. Up to 25 number 18 bars are required to resist the tension in the caissons. To anchor the building columns to the caissons for tension, the structural steel sections extend down into the 5 foot caissons as much as 15 feet with welded shear lugs to transfer the force. Spiral ties at 4 inch pitch confine the concrete in the lap zone.

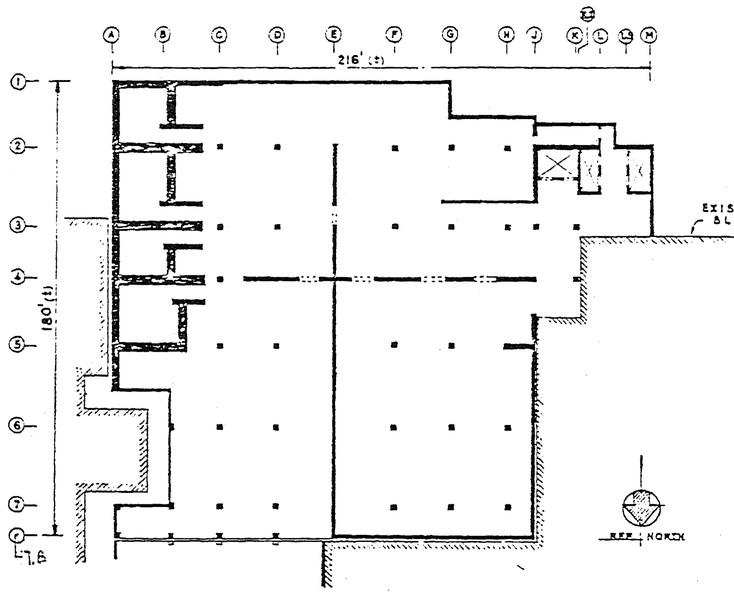
The construction of the plated shear wall system was done much like any framed structure. The columns and plate girders within the wall were erected as part of the steel frame. After the frame had been aligned, the steel plates were lowered into place, attached to the girders and boundary columns with stitch bolts. The metal deck flooring was normally in place prior to the lowering of the wall plates, thus giving the workmen a platform from which to work.

The wall plates were shipped to the site in one piece, except for a few of the larger pieces. The largest column-to-column spacing in the building is 32'-4". Most bays have full height openings (between girders at adjacent floors) for doors or corridors, thus allowing the wall panels to be a reasonable length for shipping. The height of the panels were approximately 8'-2". This allowed 9" erection tolerance between the boundary girders. If the wall plates within a single panel had to be spliced, full penetration butt welds were required.

Continuous erection tabs were welded in the shop, to the boundary columns and girders (see Figure 4). The stitch bolting size and spacing determination was left up to the Contractor. Once the wall framing was completely welded at each level, the plated wall panels were welded in place. It was required that the welded plate connections, usually to the erection tabs, must develop the ultimate strength of the plate itself.

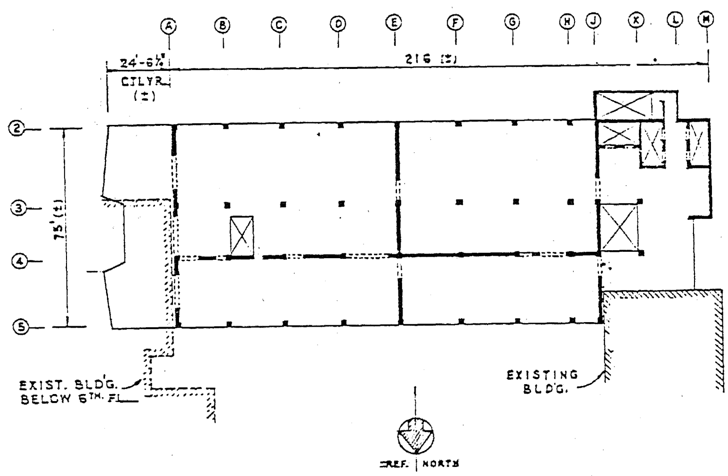
The Contractor elected to use shotcrete to provide the concrete cover on each side of the plates.

The use of structural steel plated shear walls on this project allowed the construction of this fifteen-story hospital, built to unusually stringent lateral force criteria, to be accomplished with a minimized thickness of shear wall. The amount of materials (steel weights averaged 28 pounds per square foot of building area) and labor was comparable to other structural steel, rigid framed structures of comparable size and designed for the same force level. The erection time was also not much different. The significant disadvantage of this type of construction is the required coordination between the design disciplines, to predetermine where all wall penetrations were to be. Although there were parameters given to allow the field coring of small holes (less than 8" Ø), all major openings had to be set. This meant that mechanical and electrical requirements for passage through the wall had to be determined during the design phase as opposed to the normal determination during construction. Needless to say, there were some post-construction openings that had to be made.



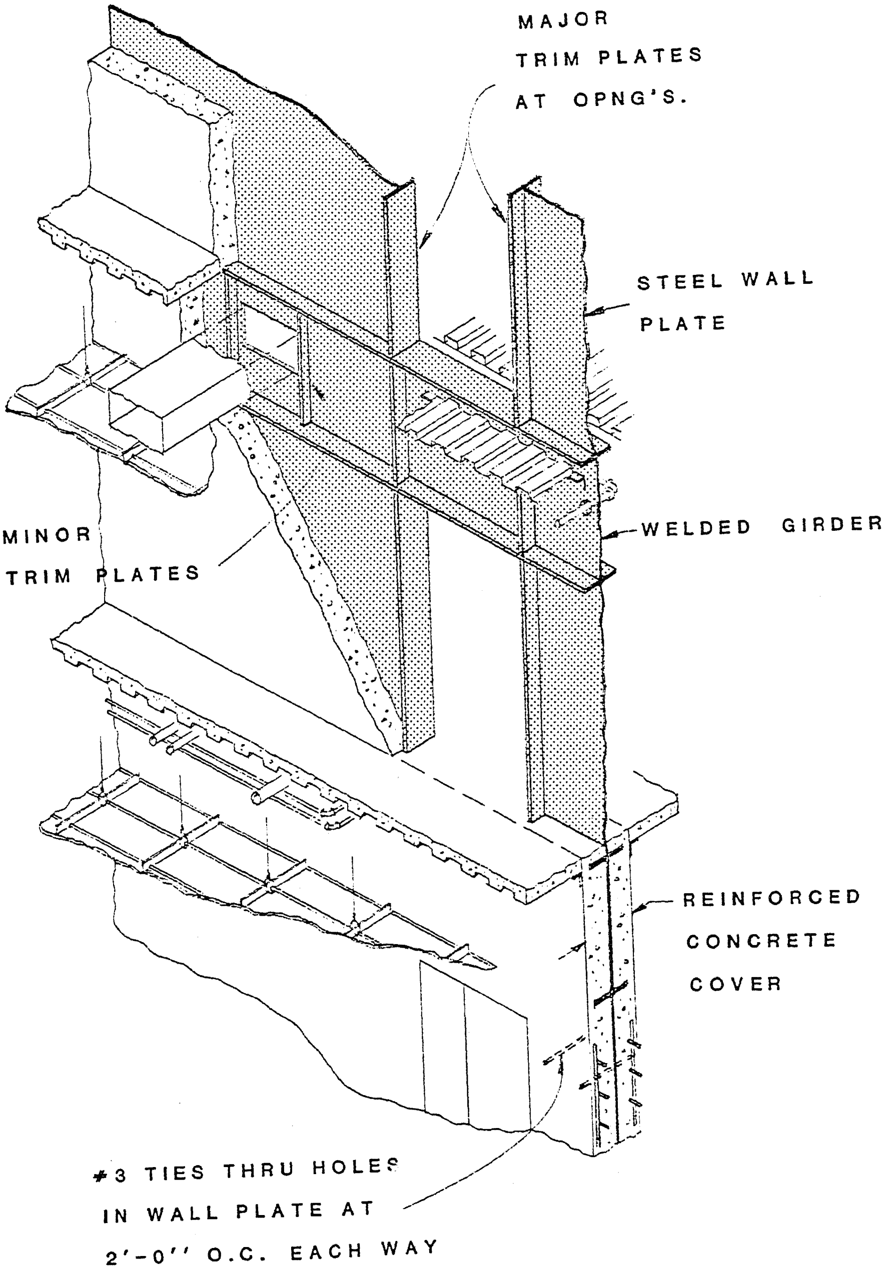
GROUND FLOOR PLAN

(FIGURE 1)

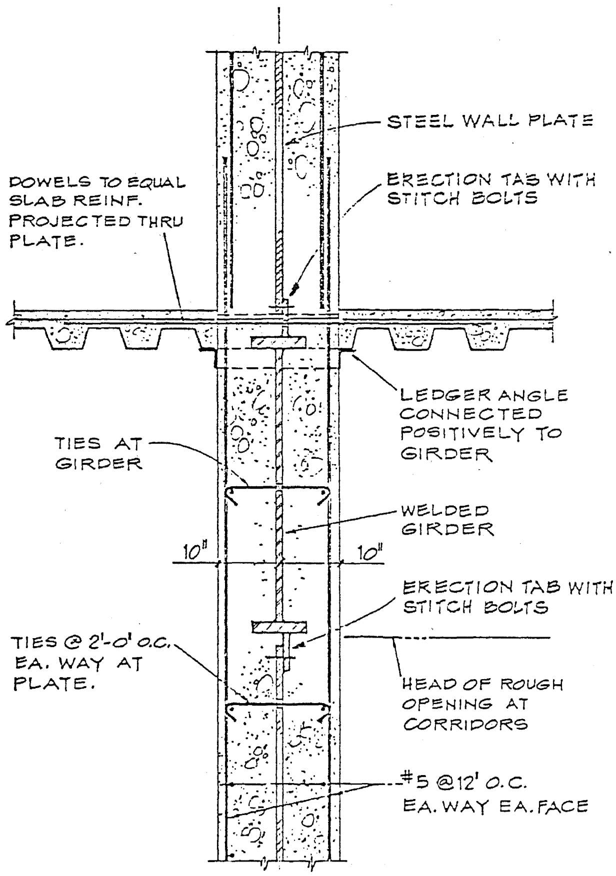


FLOOR PLAN - EIGHTH THROUGH FIFTEENTH FLOORS

(FIGURE 2)


MAJOR
TRIM PLATES
AT OPNG'S.
STEEL WALL
PLATE
MINOR
TRIM PLATES
WELDED GIRDER
REINFORCED
CONCRETE
COVER
#3 TIES THRU HOLES
IN WALL PLATE AT
2'-0'' O.C. EACH WAY

(FIGURE 3)



SECTION THROUGH PLATED WALL

(FIGURE 4)