Steel Shear Walls for Existing Buildings

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After the 1971 San Fernando earthquake in which two Veterans Administration hospitals collapsed, killing 46 people, the VA initiated an extensive program for evaluating and, if necessary, strengthening all VA hospital buildings in earthquake-troubled areas. One of the facilities that required strengthening was the VA Medical Center in Charleston, South Carolina. Because of the complexity of the project, URS/John A. Blume & Associates, Engineers, San Francisco, was selected to serve as structural consultant to Lafaye Associates, Architects, Columbia, South Carolina.

Built in 1963, the center is a complex made up of several buildings having from one to five stories. The three main buildings are separated from one another by 2-in. expansion joints. The total floor area of the five-story complex is 350,000 sq ft.

Each building uses reinforced concrete flat slab construction with columns at 20 ft on center each way. The structures were originally designed to resist lateral loads caused by wind only. The lateral forces were carried by isolated stair walls and frame action. Whereas the design base shear caused by wind forces was approximately 3% of the total dead load, the new VA earthquake code requires the strengthened structure to handle a base shear of 15%. The existing reinforced columns would be severely overstressed with a base shear of 15%; thus, the building had to be strengthened.

SHEAR WALLS

One proven method for strengthening existing buildings for lateral loads is the addition of shear walls. Shear walls added in either the transverse or longitudinal direction aid in strengthening a building by providing additional resisting elements and also by reducing the floor or roof diaphragm spans, both of which reduce diaphragm shears.

In the past, if shear walls were to be used to strengthen an existing building, they would, in almost all cases, be reinforced concrete, especially if the existing building was a reinforced concrete structure. For the Charleston VA hospital, however, many of the new shear walls were made of steel.

WHY STEEL?

From the first, the VA stressed that, whatever strengthening scheme was used, it must minimize the disruption of service in the hospital as well as respect the comfort and security of the patients. The use of hospital floor space for construction must be kept to a minimum because the associated cost could exceed the cost of building new walls.

An early analysis, as well as those that followed, proved that the only way to strengthen the central wing in the transverse direction would be to add shear walls at four parallel locations approximately 100 ft apart. This meant that, for a five-story building, 60 shear walls of either concrete or steel would be needed (4 frames × 3 walls per floor × 5 floors). To use concrete for these walls, however, would require that the rooms on either side of a new wall and the corridors leading to the rooms be removed from service during construction.

Another important consideration in rehabilitating hospital buildings is that rooms continually undergo changes in their function: for example, from a ward to a laboratory. These changes are generally accompanied by relocation of ducts and pipes. Therefore, the new shear walls had to be adaptable enough to allow for future penetrations.

Steel shear panels for interior walls were seen as a way both to minimize hospital disruption and to allow for future expansion. Such panels, made up of a steel web plate with both vertical and horizontal stiffeners, could be built in place or prefabricated and connected in place. The panels could be installed in one room against an existing partition, while, on the other side of that partition, the normal operations of the hospital could continue uninterrupted. Also, the corridor leading to the room would not be congested with the materials and equipment needed for concrete construction.

It was estimated that, although steel shear panels would cost $230,000 to construct and pneumatically placed concrete walls $170,000 (including the cost of equipment rental), the savings from the uninterrupted use of hospital

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floor space would be between \$100,000 and \$200,000. The total cost for the entire renovation project was estimated to be \$25/sq ft.

Perhaps the most important benefit was that steel shear walls could be constructed to permit future relocation of pipes and ducts without extensive modification of the existing wall. The panels were designed with continuous 4-in. slots in every other panel approximately 2 ft from the soffit of the slab above. The slots were to be reinforced with bridging plates 4 in. wide located at 8 in. o.c., so that the 4-in. slot would have a 4-in. × 4-in. opening every 8 in. for new and existing piping.

If future ducts were required, a whole subpanel (approximately 36 in. × 44 in.) could be removed and the horizontal stiffeners forming both horizontal edges could be modified without much trouble. If the wall were concrete, relocation of pipes and ducts would require core drilling and partial demolition of the wall.

DESIGN CONSIDERATIONS

The steel shear panels were designed and detailed so that they could be built in place or prefabricated in units (see Fig. 1). Their details also permitted all welded connections to be made from one side only. The shear forces, axial forces, and moments used in the panel design were developed from a dynamic analysis.

The first step in the analysis of an individual shear panel is to determine the stiffness of the shear elements (concrete shear walls and steel shear panels) that is required to restrict interstory displacement and to ensure that corresponding columns and diaphragms are not overstressed and remain within elastic limits. Once the required stiffness is determined, the next step is to develop a mathematical model of the existing building with the new shear elements. A model was developed for both the central wing and the east wing of the hospital complex. Using the computer program TABS 4,[1] the dynamic modal analysis of each building determined the shears, axial forces, and moments for each of the new shear walls, as well as for the existing columns and slabs.

VA handbook H-08-8[2] outlines the appropriate peak ground acceleration and response spectra for each of its major hospitals and requires that the shears, axial forces, and moments for each member be based on the larger of the two modal response values determined by: (*1*) the square root of the sum of the squares of the 10 highest modal responses modified by the participation factors, and (*2*) the sum of the absolute values of any 2 modes modified by the participation factors.

With this requirement, the location and number of new shear walls were determined by trial and error. During each iteration, the interstory displacement was determined to be within limits developed previously. The final strengthening scheme consisted of exterior reinforced concrete walls between exterior columns and interior steel shear panels, each of which was made up of several subpanels.

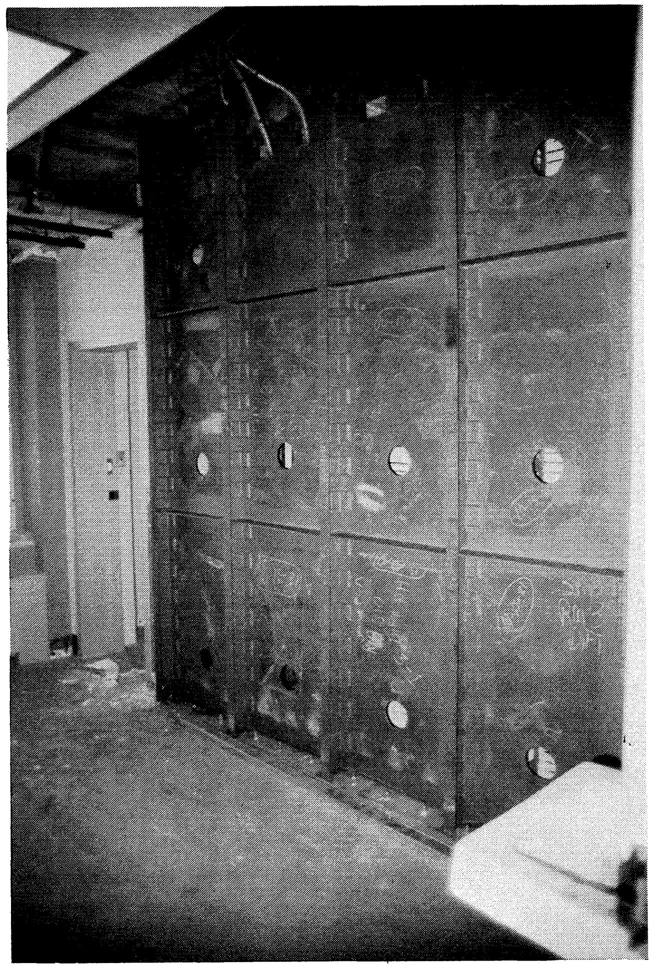

Fig. 1. Finished shear wall before plastering

It was necessary to ensure that each steel shear panel perform throughout the cyclic earthquake loading without plate buckling or loss of panel stiffness. This design feature was guaranteed by controlling the size of the web plate and the subpanel area delineated by both horizontal and vertical stiffeners. After considering the number of one-sided welds that could result in a large out-of-flatness factor for each subpanel and the uncertainty involved in this type of field installation, a factor of safety of 4 against plate buckling was selected.

DESIGN EXAMPLE

Consider, as an example, the fourth-story shear panel between columns **C** and **D** on building line **13** (Figs. 2, 3, and 4). The bending and shear stresses in both the stiffeners and the web plate were as high or higher than those in other panels. From the dynamic analysis, the story forces, which for this case were developed from the square root of the sum

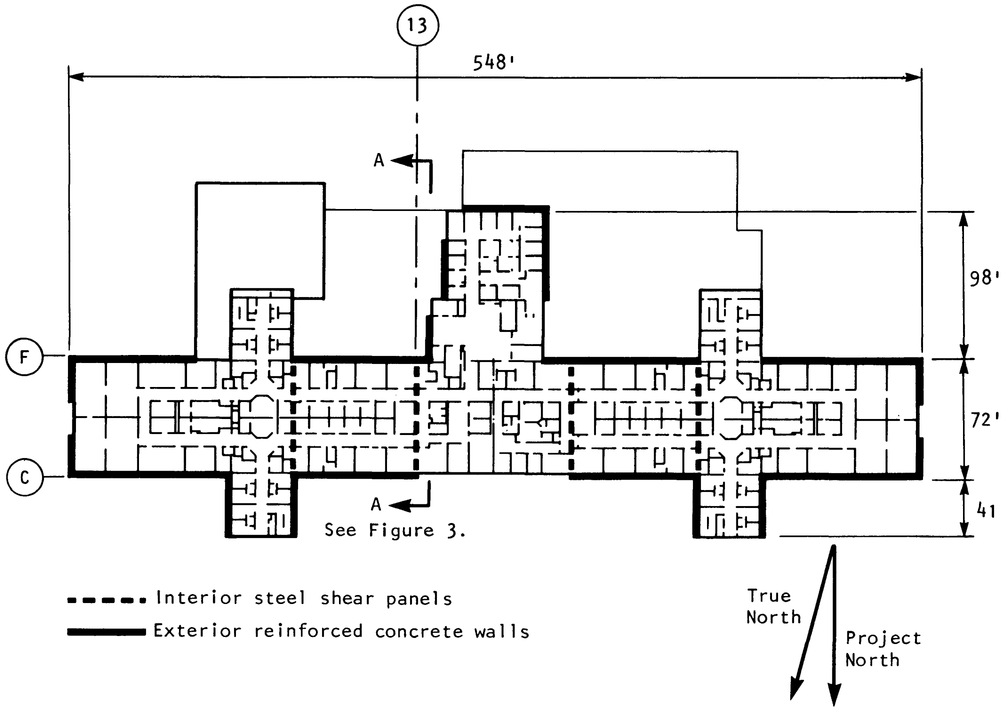

Fig. 2. Fourth-floor plan

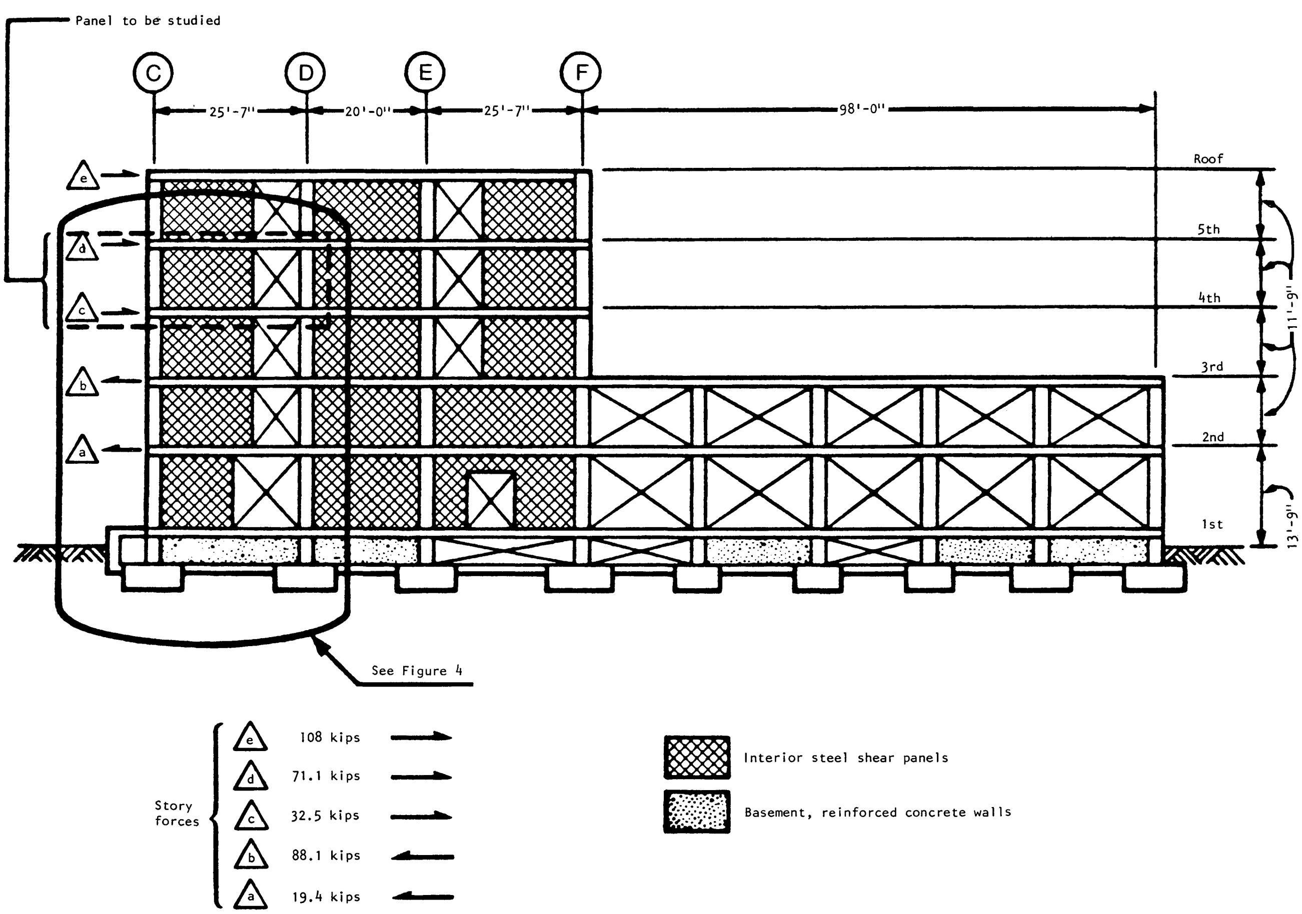

Fig. 3. Section A-A (at building line 13)

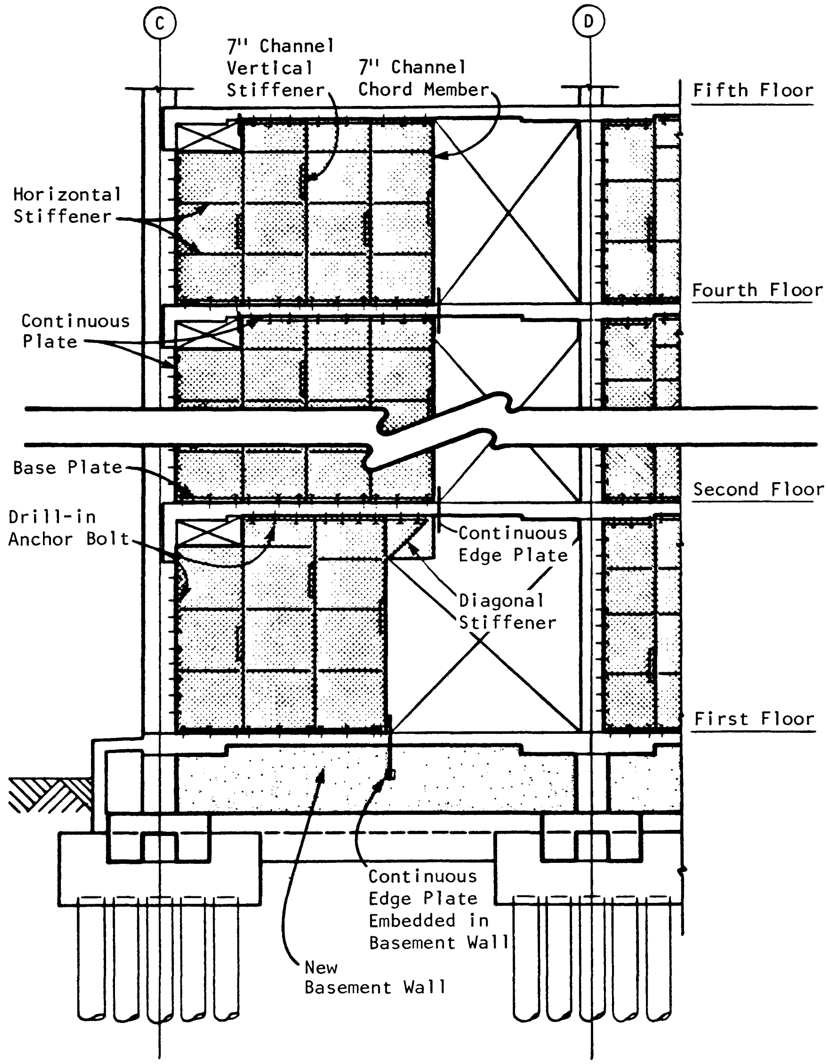

Fig. 4. Elevation of steel shear panels on building line 13

of the squares, are shown in Fig. 3. The corresponding interstory displacement at the fourth floor was 0.04 in. If all the shear panels were removed and the same story forces applied, the corresponding fourth-floor interstory displacement would be 0.20 in.

At the beginning of the seismic analysis, we determined that the relative stiffness of each of the frames on building lines 9, 13, 18, and 21 with new shear elements should be at least 4 times the relative stiffness of the existing frames without shear elements. With this requirement in effect, the column shears would be reduced by at least 50% for earthquake forces in the building's transverse direction. The relative stiffness of the fourth floor of the frame on building line 13 with shear elements is 0.20 in./0.04 in., which is equal to 5 times the relative stiffness without shear elements.

For selected shear panels, the shear, axial forces, and moments, derived from the dynamic analysis, are given in Fig. 5, and connection details are shown in Fig. 6.

Once these forces were known, the next step was to set up a simple finite-element model. Because this model treated the steel shear panel as independent of the concrete frame, the only forces on the panel were the shear forces at the top edge. In the in-plane direction, the two edges and the top of the panel were free, and the bottom edge was pinned. For the out-of-plane direction, all edges were pinned. With this simplified model, the calculated membrane stresses on each of the subpanels were slightly greater than they would have been if the model had coupled the shear panel to the concrete frame.

The membrane stresses for each of the 15 subpanels are given in Fig. 7.

WEB PLATE DESIGN

To limit plate buckling of the web plate caused by shear forces, a combination of vertical and horizontal stiffeners

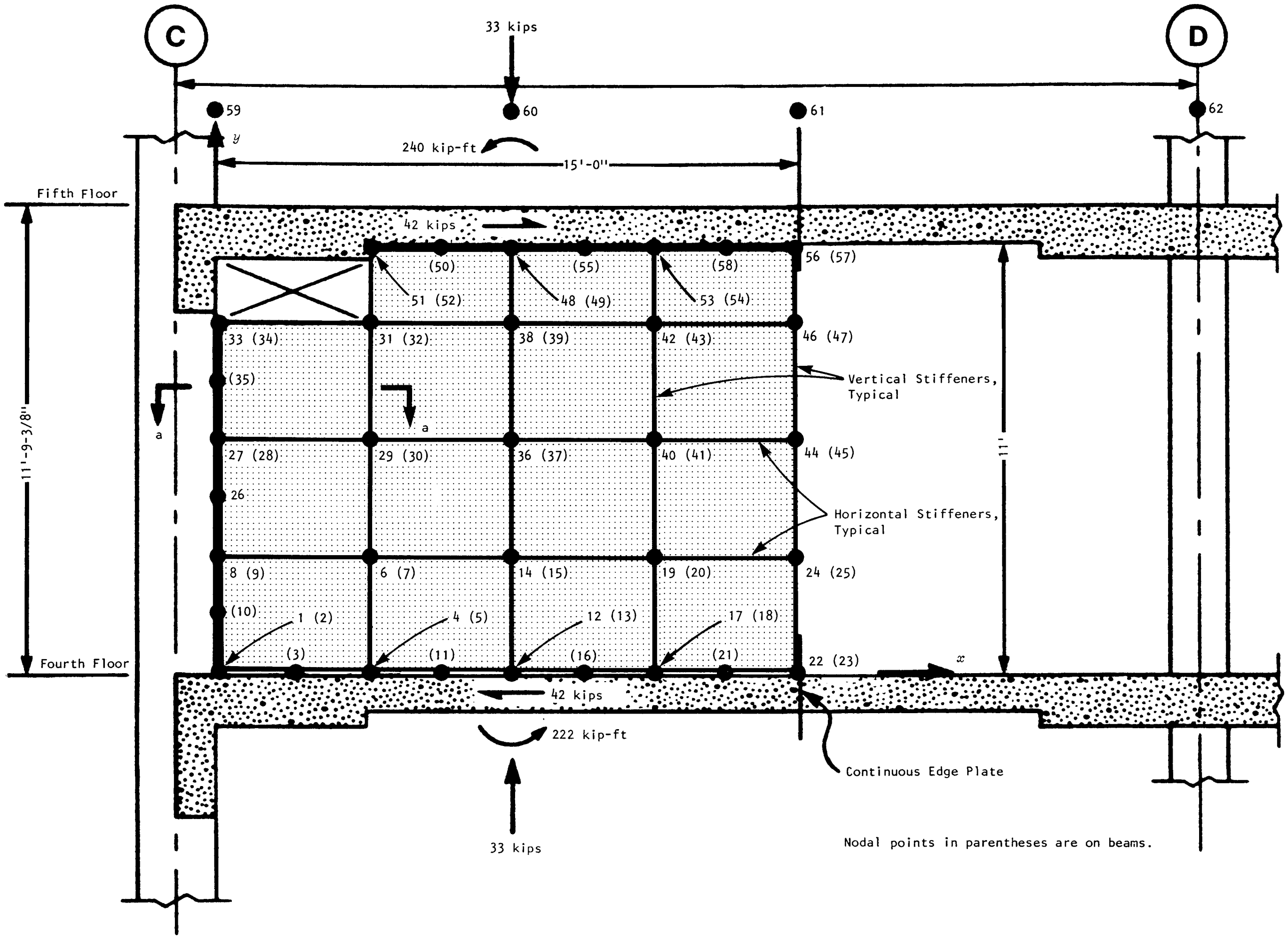

Fig. 5. Finite-element model of frame on building line 13

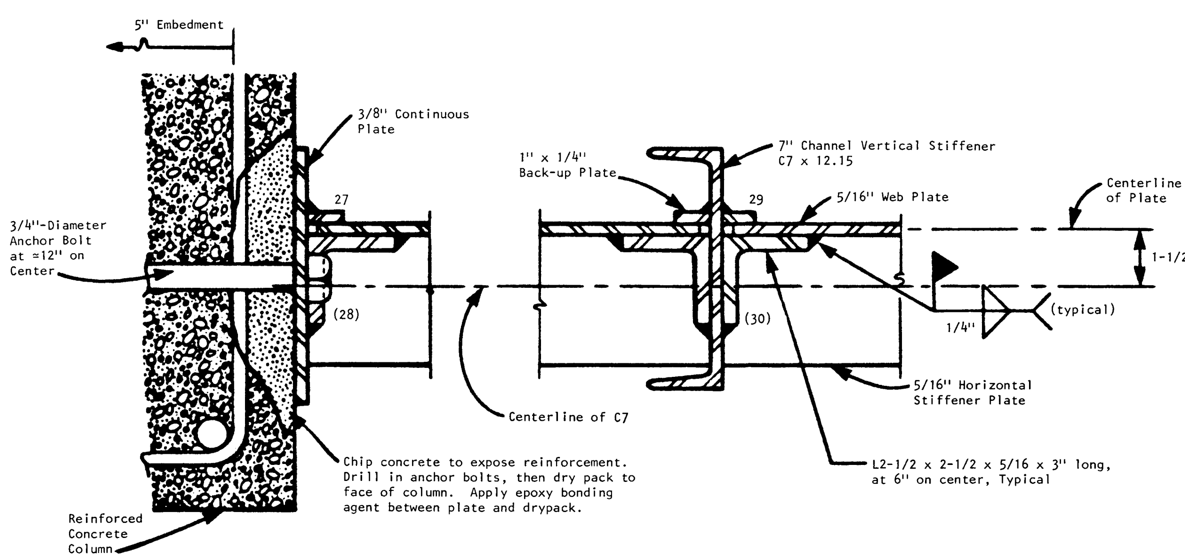

Fig. 6. Section a-a (Fig. 5)

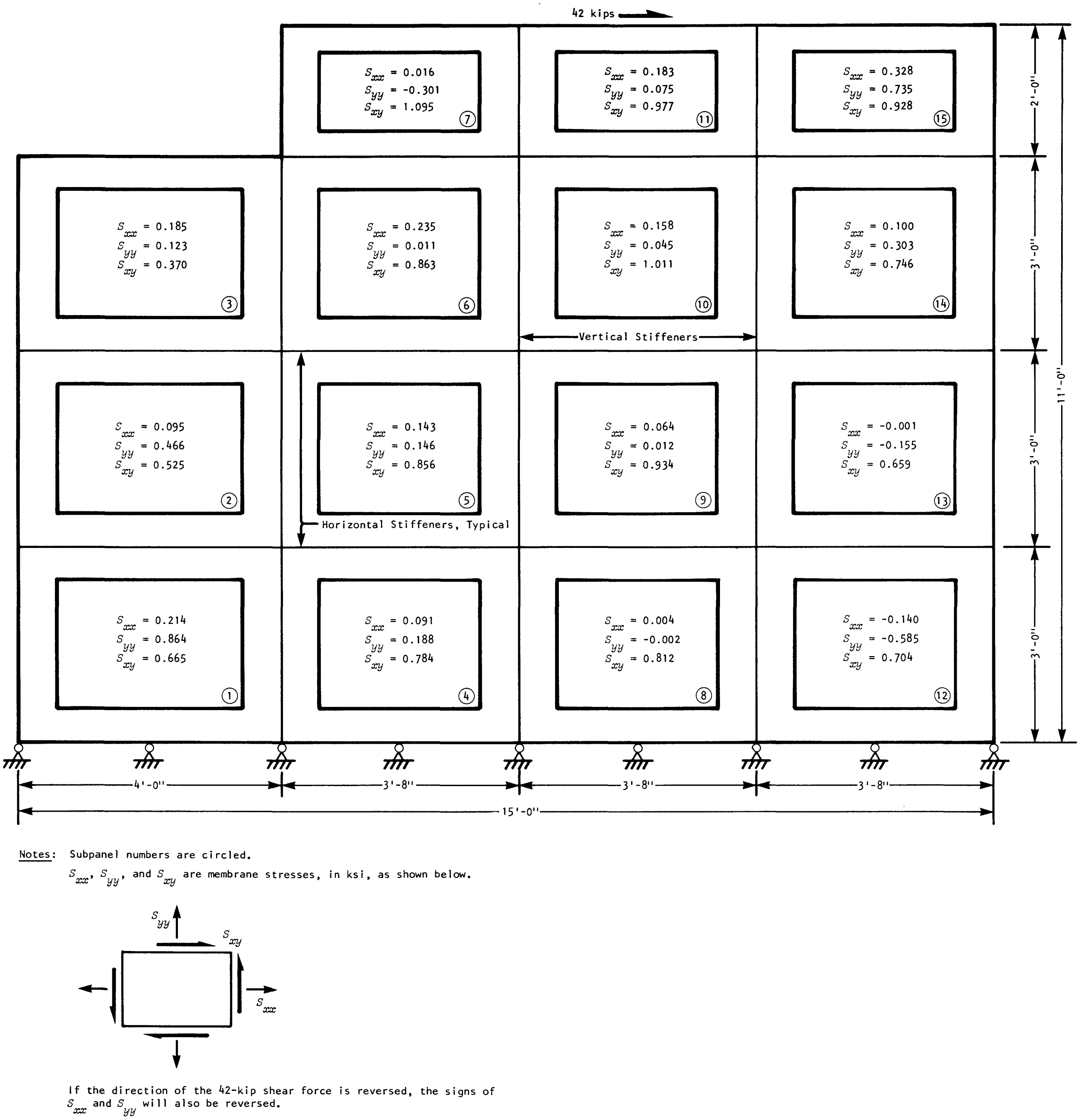

Fig. 7. Finite-element model

was used. Controlling the size and spacing of both allows the shear panel to be analyzed as a ribbed plate composed of subpanels, each approximately 3 ft × 4 ft. The web plates are 5/16-in. thick and weigh 150 lbs each. The critical buckling shear stress (τ_{cr}) and the critical buckling compressive stresses (δ_{cr}) were obtained from Ref. 3.

Two panels were checked for plate buckling: Panel 9 (Condition 1) has the highest in-plane shear stress, and Panel 1 (Condition 2) has both in-plane shear and compressive stresses. A factor of safety of 4 was used for both conditions.

Condition 1, Plate Buckling for Shear Only—The membrane stresses S_{xx} and S_{yy} are tension stresses that would increase the buckling capacity of the panel and may conservatively be ignored. See Fig. 8.

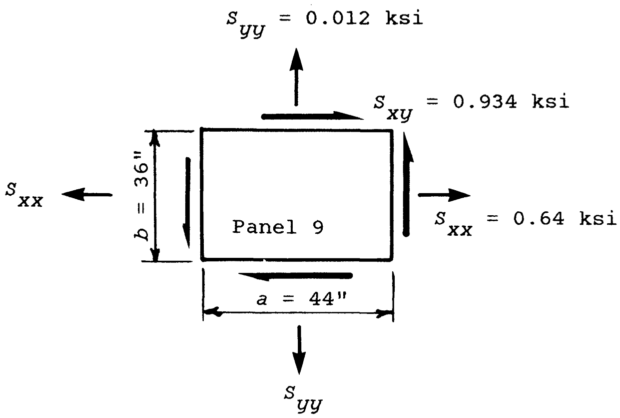

Fig. 8. Condition 1 (Panel 9)

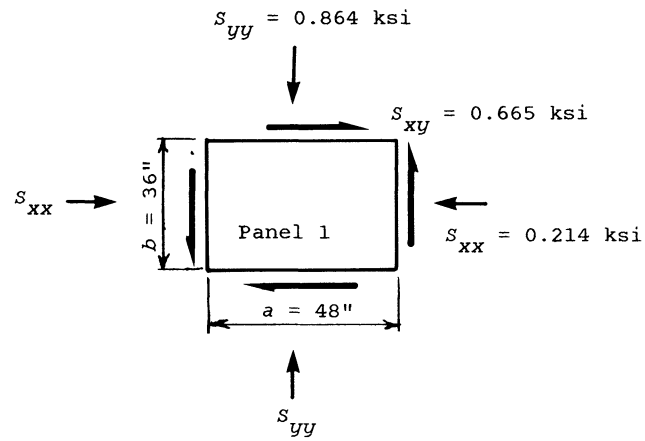

Fig. 9. Condition 2 (Panel 1)

$$\tau_{cr} = \frac{k\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2$$

where

- E = Modulus of elasticity in compression (30,000 ksi for steel)
- t = thickness of plate ($^5/_{16}$-in.)
- b = width of plate (36 in.)
- a = length of plate (44 in.)
- α = aspect ratio $= a/b = 44/36 = 1.22$
- ν = Poisson's ratio (0.3)
- k = buckling coefficient $= 5.34 + (4/\alpha^2) = 8.02$

$$\tau_{cr} = \frac{8.02(3.14)^2(30{,}000)}{12[1-(0.30)^2]}\left(\frac{5}{16(36)}\right)^2 = 16.38 \text{ ksi}$$

For a factor of safety of 4, τ/τ_{cr} must be less than $^1/_4$, where τ is the value of S_{xy} determined for the dynamic analysis. For this subpanel:

$$\frac{\tau}{\tau_{cr}} = \frac{0.934}{16.38} = 0.06 < {}^1/_4 \text{ (or 0.25)}$$

Condition 2, Plate Buckling Due to Both In-Plane Shear Forces and Biaxial Compressive Forces—For Condition 2 (see Fig. 9), the above formula is expanded to its more general form:

$$\frac{\delta_y}{\delta_{cr_y}} + \frac{\delta_w}{\delta_{cr_x}} + \frac{\tau}{\tau_{cr}} \le {}^1/_4$$

where

$$\delta_{cr} = \frac{k\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{h}\right)^2$$

For this subpanel:

$$h = b = 36 \text{ in.}$$
$$a = 48 \text{ in.}$$
$$\alpha_x = 48/36 = 1.33$$
$$k_x = 4.0 \text{ when } \alpha \ge 1.0$$
$$\delta_{cr_x} = \frac{(4)(3.14)^2(30{,}000)}{12[1-(0.3)^2]}\left(\frac{5}{16(36)}\right)^2 = 8.18 \text{ ksi}$$
$$\delta_x = S_{xx} = 0.214 \text{ ksi}$$
$$\alpha_y = 36/48 = 0.75$$
$$k_y = \left(\alpha + \frac{1}{\alpha}\right)^2 \text{ when } \alpha < 1.0$$
$$= \left(0.75 + \frac{1}{0.75}\right)^2 = 4.34$$
$$\delta_{cr_y} = \frac{(4.34)(3.14)^2(30{,}000)}{12[1-(0.3)^2]}\left(\frac{5}{16(48)}\right)^2 = 4.99 \text{ ksi}$$
$$\delta_y = S_{yy} = 0.864 \text{ ksi}$$
$$\tau_{cr} = \frac{k\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2 = \frac{7.59(3.14)^2(30{,}000)}{12[1-(0.37)^2]}\left(\frac{5}{16(36)}\right)^2 = 15.51 \text{ ksi}$$
$$\tau = 0.665 \text{ ksi}$$

Therefore,

$$\left(\frac{\delta_y}{\delta_{cr_y}}\right) + \left(\frac{\delta_x}{\delta_{cr_x}}\right) + \left(\frac{\tau}{\tau_{cr}}\right) = \left(\frac{0.864}{4.99}\right) + \left(\frac{0.214}{8.18}\right) + \left(\frac{0.665}{15.51}\right)$$
$$= 0.17 + 0.03 + 0.04 = 0.24 < 0.25$$

VERTICAL AND HORIZONTAL STIFFENER DESIGN

Both the vertical and horizontal stiffeners were designed as columns, similar to the way a bearing stiffener is designed. The vertical stiffeners are 7-in. channels spaced about 4 ft on center, and the horizontal stiffeners are 4-in. plates, $5/16$-in. thick, spaced at about 3 ft on center. A $3/8 \times 7 1/2$-in. continuous steel plate around the perimeter of the shear panel connects it to the existing reinforced concrete structure by means of drilled-in anchors spaced 6 to 18 in. on center and secured in place with an epoxy bonding agent. The center line of the $5/16$-in. web plate is set $1 1/2$ in. from the center line of the 7-in. channels and the center line of the $3/8$-in. perimeter plate to permit one-sided connections. A section through the panel is shown in Fig. 6.

At clear openings in the panel, the vertical plate, continuous through the floor slab, connects a 7-in. edge channel or chord member above to one below. The chord force would then be the sum of the top and bottom moments divided by the panel length. For the sample panel shown in Fig. 5, the chord force would be $(240 + 222)/15 = 30.8$ kips. The 7-in. edge channel would then be designed as a bearing stiffener as outlined in Section 1.10.5 of the AISC Specification.[4]

The continuous edge plate is designed to carry all the chord force in tension. The edge plate for the sample panel measures $4 1/2$ in. $\times$ $3/8$-in. $\times$ 18 in. long.

ANOTHER DESIGN APPROACH

The main design consideration for this analysis was to prevent each subpanel from buckling by restricting the thickness and area of the web plate to ensure that the design edge compression and shear stresses are less than one-fourth of the buckling stresses. It is also possible to design each shear panel as a truss where each subpanel acts as a tension strut and the vertical and horizontal stiffeners act as chords. Applying this concept of a tensile stress field in the manner of a stressed skin design could reduce the web plate thickness as well as the stiffener sizes. However, the relative panel stiffness requirements remain the same regardless of how the panel is designed, and these requirements limit the final size of the shear panel.

CONCLUSION

The use of steel panel construction for interior shear walls can be an economical means of strengthening an existing reinforced concrete building to resist earthquake forces.

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