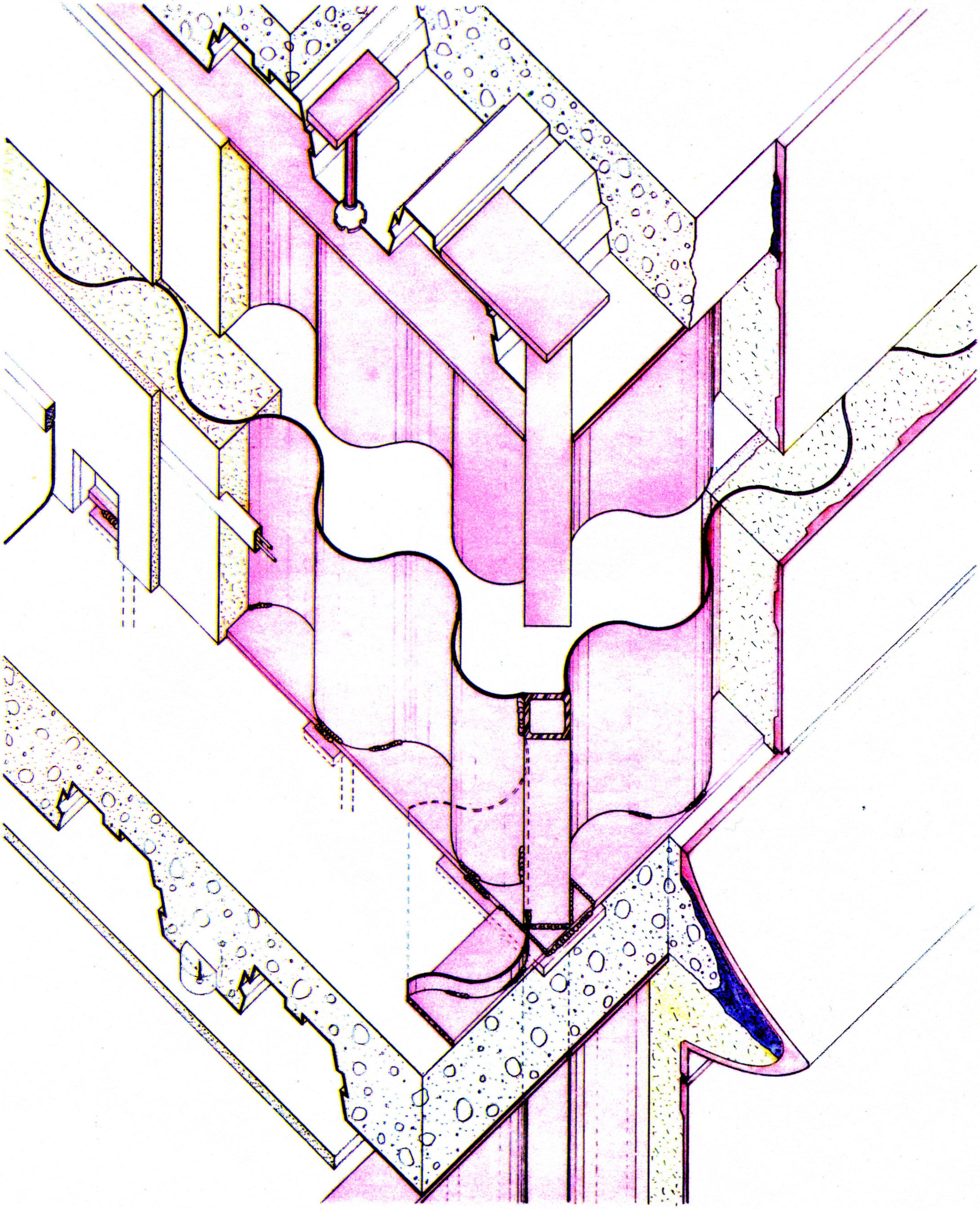

A NEW PREFABRICATION SYSTEM UTILIZING STEEL BEARING WALLS

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A NEW PREFABRICATION SYSTEM UTILIZING STEEL BEARING WALLS

As Applied to a Mid-rise Building With Offices and Elderly Housing

A Professional Report
by
Pete Retondo

Advisors:

Prof. Claude Stoller
Prof. Sim Van der Ryn

U.C. Berkeley
School of Environmental Design
Department of Architecture

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INTRODUCTION

The prefabrication of buildings has been an unrealized dream since the building of Joseph Paxton's Crystal Palace in 1851. The use of prefabricated elements has become more and more pervasive since that time, but the actual level of industrialization of the building process has not advanced significantly beyond Paxton's column, beam and skin system developed more than a century ago. The only significant advancements have been the development of the "mobile home" and the use of reinforced concrete wall elements. Neither of these has the capacity to break open a new frontier in the building process.

"Prefabrication" is a word which illuminates its own meaning: it presumes the division of the work of building into that which takes place during its erection, i.e., work in the field, and that which takes place in advance. The idea has always been to allocate as much work as possible to a factory setting, in order to take advantage of industrial efficiencies and large-scale processes. Work performed in the field is considered by most to be relatively inefficient, although the issue has been clouded by the fact that factory unions, and hence wages, have been historically less strong than those in the building trades. Nevertheless, the automobile industry, where the repair of perhaps 15% of a car can cost more than its production price, shows that mass production is inherently more efficient; indeed, consensus on the advantages of mass production has become so overwhelming that the prejudice in its favor, which really does call for a careful and critical scrutiny, has become a matter of "common sense."

The industrialization of the building processes has had a rocky history, due to more than one factor. The first is the difficulty of scale: buildings the size of a house trailer are the only ones to have been successfully built in factories. Second, there has been a cultural recalcitrance, initiated in the brilliant work of John Ruskin (1819-1900)[1] , who deplored the industrial revolution and argued that the preservation of traditional methods was necessary to the preservation of a healthy society. Despite the great work of people like Walter Gropius, this basic criticsm has remained unchallenged. Third, mass production implies standardization, as Le Corbusier was the first architect to understand completely[2], and the difficulty of adapting standardized elements to the variety of human needs, contexts and tastes has proved a deep problem. Finally, there is the complexity of the enterprise as a whole, which implies that the process of developing the methods and industrial bases must be one of evolution. There have been a lot of false starts--most notably, the failure to adapt the "quonset hut" idea to housing following WWII (witness also the failure of the "Lustron" building developed by U.S. Steel)--which, in the context of our

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[1] See, for example, Ruskin's <u>Seven Lamps of Architecture</u>, 1849.

[2] Le Corbusier, <u>Towards a New Architecture</u> (1923).

merciless economic system, has placed a chill on the idea.

Despite these difficulties and objections, the present state of the business cannot continue as it is. Recent history has shown us that industrial processes have swept away all competitors, and that the only vesitages of "craft" we are likely to retain in the the building industry are those defined precisely by William Morris' experiment of the 19th century: the realm of "high culture," art, and luxury consumption. The major share of building will inevitably shift to the most economic techniques; and there are a lot of signs of present inefficiency which presage major changes.

The jobsite on a substantial urban construction project is graphic evidence that building is the least industrialized of our major industries. Subcontractors and tradesmen compete for precious free space for the storage of materials, tools, and just to have a place to work. Repetitive movements, damage to materials, scheduling headaches, lack of coordination among the trades, are a common litany of the disadvantages we live with. The industry has adapted with increasing specialization and a variety of subcontracting arrangements, which tend to have the unfortunate side-effect of splintering job responsibility and creating a lot of paperwork. This bulky and redundant administrative structure only increases scheduling problems and confusion, sometimes deliberately cultivated, as to which responsibility is whose.

Suppose most of the work in a typical urban multi-story building were performed away from the site in a factory context. Then some of the major headaches of urban construction would be immediately be relieved. The job would have enough space; supply of materials could be centralized; tools would not have to dragged from site to site, and once there, through every room and floor of the building; production could be more easily coordinated and rationalized, and time-saving tools and practices could be instituted; large-scale machinery could be applied to processes which are presently done in bits and pieces by hand; and management structure could be arranged more optimally so that authority was neither too splintered nor too buklily centralized. These benefits alone would outweigh extra transportation costs, even without considering the possibilities of automation and standardization of production. Basically, the whole process could evolve out of the ordinary contractor's yard.

The major beneficiaries of such a rationalization might be the cities themselves. Because a large part of the work would be done in a satellite area, the city would be spared a large portion of the noise, pollution and inconvenience of construction. Delivery of materials would be more predictable, because the bulk of them would be coming from one source, and instead of being blocked by a protracted flow of bits and pieces, the streets would be used for a relatively short and highly choreographed period of time.

APPLICABILITY AND ADVANTAGES OF A BEARING WALL SYSTEM

Partial Prefabrication and Cellular Buildings

The insuperable obstacle to the complete factory fabrication of buildings of any size beyond the mobile home is their scale. The largest factory-built piece of a building would have to be on the order of 8 X 12 X 30 feet, which are the limits imposed by highway transportation. The foundation and site work would have to be done in the traditonal manner; moreover, the integration of the prefabricated components would have to be performed by methods more-or-less similar to those employed in ordinary construction.

This size limitation makes the use of prefabrication most optimal for cellular buildings, that is, buildings such as apartments, hotels, hospitals, where the partition walls are conceived of as being a permanent part of the structure. The application of industrial processes could be even more extensive in such buildings than it is in the typical factory or open-plan office building, where only the skeletal structure and cladding elements are today prefabricated in factories. In the construction of these cellular buildings, which are the most inefficiently built of all buildings, there is the opportunity to complete most of the partition, mechanical, interior finish, and built-in furniture work in the factory.

Bearing Walls and Organic Structure

The most important obstacle to the employment of prefabrication in cellular buildings is a conceptual one. The idea of a skeletal-frame and floor-diaphragm structure, with the rest of the building "floating" within the structure was developed for steel in the 19th century on behalf of the high-rise office building business, and for concrete in the early 20th century by Le Corbusier[3] for housing in Europe. Corbusier's idea has been raised into the realm of aesthetic dictum; nevertheless, to realize the potential of industrialization which he idolized, that structural concept must be set aside (Corbusier didn't understand building, anyway--he was a painter and a wealthy dilletante).

There is an inherent redundancy in the idea of skeletal structure filled out with non-bearing curtain walls and exterior cladding. The logical solution is one which seems more difficult, because in visualizing construction we carry around a great number of assumptions based upon site-restricted building methods. We can clearly see the skeleton rising, forming a work platform for all the other trades which rise through the building in succession. Postulate the performance of all those tasks in a factory removed from the city, however, with each operation and its tools remaining fixed in one place, with the building elements coming to the operation rather than <u>vice-versa</u>, and we must visualize the whole process differently. If the vertical

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[3] Le Corbusier, <u>op. cit.</u>, the "Maison Dom-ino."

load-bearing structure of the building is contained <u>within the partition walls</u>, the redundancy can be eliminated. The difficulties of integrating into one component the functions of structure, fire-protection, utility conveyance, noise containment and decoration can all be reasonably solved in an industrial fabrication context.

By thus integrating structure and amenity, a more organic form is realized. The knitting together of all the structural contributions made by elements currently considered "non-structural" can be empirically tested and taken into consideration. Ultimately, the way may be opened to develop an even more integrative approach than we can now imagine--a complete integration of plumbing, electrical conduits, ventilation space and structure into one matrix might be possible. The development of high-tensile strength ceramic materials may progess to the point where an entire building would be built on a matrix of ceramic load-bearing elements, with integral utility conduits. This organic approach is consistent with the shift in industrial production away from the exploded-diagram-bolted-together paradigm, towards the more sophisticated concept of the object/organism.

In order to realize this concept at the scale of a large building, care must be taken that the components are not simply stacked together; indeed, earthquake design prevents such an approach. But the problem remains of knitting the parts together in the field, and is a significant part of designing such a system. As discussed below, this is one of the difficulties with prefabricated concrete systems.

CONTEXT OF THIS PROJECT

440 Turk Street

The idea of factory-assembled bearing wall components has been developed here in conjunction with the design of a nine-story building for the San Francisco Housing Authority. The program for the design was a real one, and was borrowed from a project which is now nearing completion at 440 Turk Street in San Francisco. Because of the program for the 25,000 square feet of office space, which required the two lowest floors of the building, and which specified an elaborate and irregular division of the space, it was impossible to attempt to design the lower two floors with a cellular system in mind; it just was the wrong kind of animal. As a result, the lower two floors were designed as a concrete column and slab structure, and the development of the prefabricated bearing wall system was confined to the upper seven floors of elderly housing (90 units, total).

The requirements and restrictions of the program severely limited design freedom, so in some senses it was not the most advantageous context in which to demonstrate a new building system. Yet, in a way it was perhaps a good choice, because although some re-arrangement of a building is necessary to adapt to this technological method, it should not be required that the kind of space desired by a building's users, or the arrangements most amenable to the comfort and convenience of its occupants, be contorted beyond recognition by the technology which purports to serve them. One of the problems with technological design is the lack of an holistic approach, and attempting to integrate the designing of elements and whole systems is one way to counteract the tendency to work in a vacuum.

The design of the building is the result of a collaboration with my partner, Andy Bowen, who was the conscience of the project. Andy was much more concerned with the needs of the low-income elderly residents than he was with the rationale of the building system, and his perspective helped his partner avoid falling into the pitfalls of technological tunnel vision. As a result, the building system was designed to balance standardization of the components with an overall flexibility of form, necessary to the development of a good building, as opposed to the ultimately cheap building.

Lest the process be conceived of as an adversarial one, in which antagonistic values competed for a greater share of the compromise, it should be pointed out that it had a symbiotic side. The design of this elderly housing was subject to a specific program limiting the size and number of apartments, and prohibiting "luxuries" like bay windows. The expected savings resulting from the use of prefabrication provided a basis for using bay windows in almost all the apartments, providing a garden entrance off the street, a sunny lounge on every floor, a great deal of single-loaded corridor with ample natural light, a roof garden, and a number of amenities such as panel doors and high-quality windows. If it doesn't raise the quality of living in ways like these, the detriments caused by industrialization would hardly be worth it.

Technological limitations

The system has been designed without requiring the use of unobtanium. The general idea has been to design a system which might be developable by the year 2000. Limitations accepted here have been the use of materials already in existence, the proposal of industrial systems which could conceivably be developed using current technology, and an economic feasibility based on mass production. The automobile industry and the glass industry have provided models for the factory processes imagined. The only area in which a technological breakthrough might be required to implement this system is in the fabrication of large thin ceramic elements; advances in ceramics technology, however, are astounding. New ceramics are the basis of the semi-conductor industry; they are responsible for the new super-conductors; the development of the ceramic internal combustion engine is around the corner. With the declining use of brick, due to labor costs and structural considerations, ceramics have lost much of their traditional place in construction, but it may not be long before new ceramic materials and processes will revolutionize the construction industry. Ceramics are based upon some of the most abundant materials on earth, and the technology has been greatly refined in its 5000 year history.

THE SYSTEM

Elements and Materials

The building system is based upon steel as a structural element. The scheme is summarized in Figure 1, which shows a cutaway view of all the essential components in their final assembled form. The reasons for choosing corrugated steel and its characteristics are discussed later in the report in detail. Surrounding the steel is a layer of chopped mineral fiber in a cementitious binder. This material provides a lightweight (15 pcf) fire-protection and sound transmission barrier. "Cementitious foams" sprayed on structural steel have densities varying from 2pcf to 30 pcf, and can provide 1 hour protection in thicknesses of 5/8 inch (one manufacturer's material provides 2 hour protection with 1/2 inch thickness).[4] A typical value for heat transmission is R = 3.45 per inch. These materials bond to steel with adhesion up to 200 psf, a value which could be much improved with the use of primers on the steel. They can be applied by spraying or casting, and a foaming process has been developed for inserting insulation into cavities in old buildings.

Interior wall surfaces are formed of 1/2 inch-thick gypsum plaster cast directly on the insulation material. The resulting assembly would have a fire rating well in excess of the required 2 hours for an interior bearing wall, and form an excellent

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[4] Sweets Catalog, 1984, 7.14.

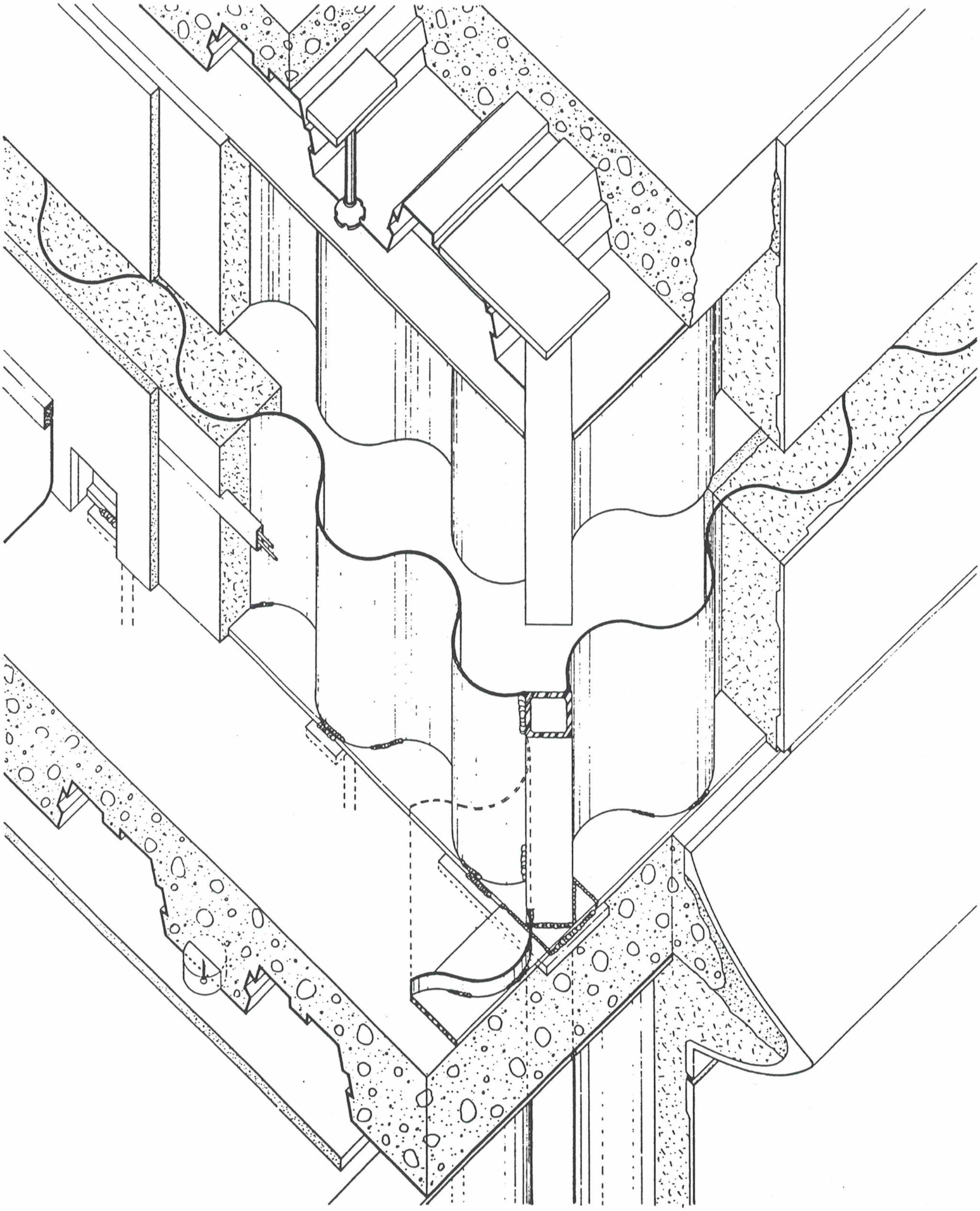

FIGURE 1

Bearing Wall System Assembly, as Installed in Building sound-transmission barrier (sound transmittance is inversely proportional to the mass of the wall, and is greatly enhanced by trapped air in the cementitious foam). A painted plaster surface is desireable for interior walls because of its ability to modulate changes in humidity.

Exterior surfaces are formed with thin ceramic panels. A grid of ribs on the inside surface of the large scale panels provides handling strength (thickness varies from 1/4 inch to 1/2 inch). Window openings would be sawed out of the panels, and cast window frames inserted (see Figure 2).

Industrial processes

The steel assembly line is the heart of the manufacturing system for prefabricated components. A plant would have to have three or four assembly lines, so that the different sections of a building could be produced simultaneously. A glance at Figure 5, below, shows that there are some major and some minor components involved; minor (i.e., compact) components could be produced for the whole job and stockpiled, while the different major components would be produced simultaneously and continuously, and shipped as a whole floor's quota was completed, thus reducing the need to stockpile an entire building before shipping for assembly.

The assembly lines would be run largely by robotic welding and steel handling machines. A cybernetic program would be developed for each building; some adjustment of the equipment would probably be necessary for each production run, but most

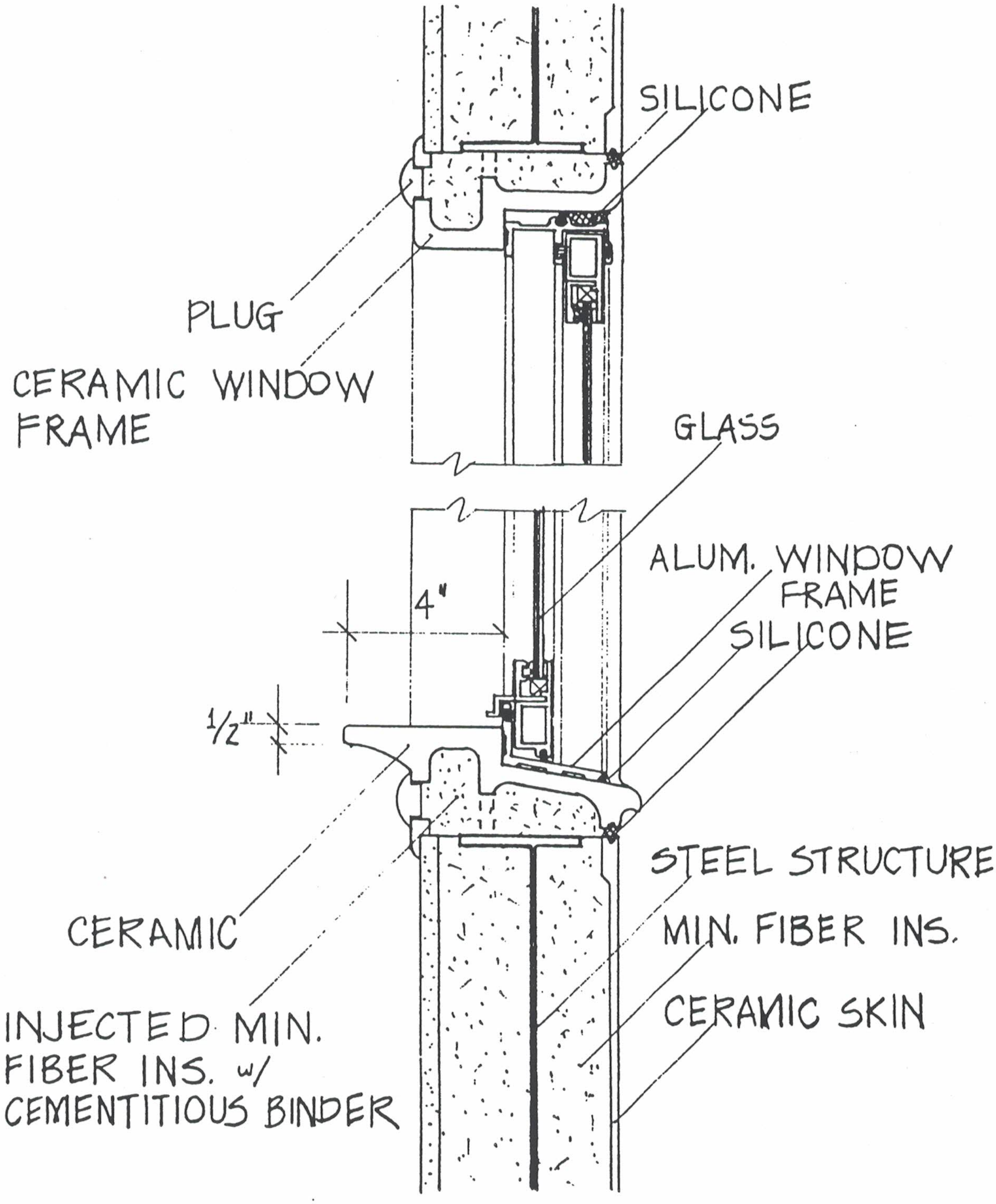

FIGURE 2

Section Through Window Sill and Head

details would be taken care of by the computer. The adaptability of each production line to jobs of differing sizes and specifications is extremely important. The scheme for production of a generalized building component might work like this (please refer to Figure 1, which illustrates most of the assemblies):

1) Long corrugated steel sheet (cold formed in a mill elsewhere) are cut to length, and continuously welded to form long wall panels;

2) Window openings, if any, are cut into the panel;

3) Top and bottom flanges and corner columns are welded in staggered patterns to the web of the wall (a corner columns is omitted in the case of those panels which are to be joined to another wall, which already has a column);

4) Batts of pre-cast insulating material, molded to the shape of the corrugations on one surface, and flat on the other, are cut to length and bonded to the wall in sections;

5) Openings for utilities are punched in the wall (i.e., for outlet boxes, ventilation ducts, plumbing);

6) The wall is laid flat, and passes through a plastering machine which lays down a 1/2 inch thick layer of gypsum, screeded to temporary metal screed attached to the top and bottom flanges, and smoothed with an industrial size squeegee;

7) If the wall is an simple interior wall, it is flipped over and the other side is plastered;

8) If the wall is part of a mechanical wall (cavity wall) assembly, the inside surface is left bare steel, and, after all

mechanical systems have been attached to the inside of one panel, another such panel is connected, leaving a 4 1/2 inch interior space (see Figure 3);

9) Completed wall panels are connected at corners by welding (see Figure 4 and Figure 4A), and access space for welding is filled with plaster;

10) A panel of exterior cladding is positioned parallel to the steel of the exterior wall, in an upright position, and the interstitial space is filled with insulating material, injected through tubes which are withdrawn as the casting proceeds (the cladding is bonded to the steel by the adhesive strength of the cementitious insulation);

11) Ceramic window frames are inserted into the wall from the inside, and bonded in a similar way by injection of insulating material through holes in the inside trim of the frame (see Figure 2);

12) Aluminum frame windows are placed in their ceramic frames from the outside, are held centered by neoprene wipers which are part of the perimeter of the aluminum frame (see Figure 2), and silicone caulking is injected into the space between aluminum and ceramic frames, bonding the window to the wall and forming a weather seal;

13) Joints in ceramic cladding are sealed with silicone, which forms an excellent bond to the glazed surfaces;

14) Various amenities, such as cabinets, are attached to the wall, and braced for transport.

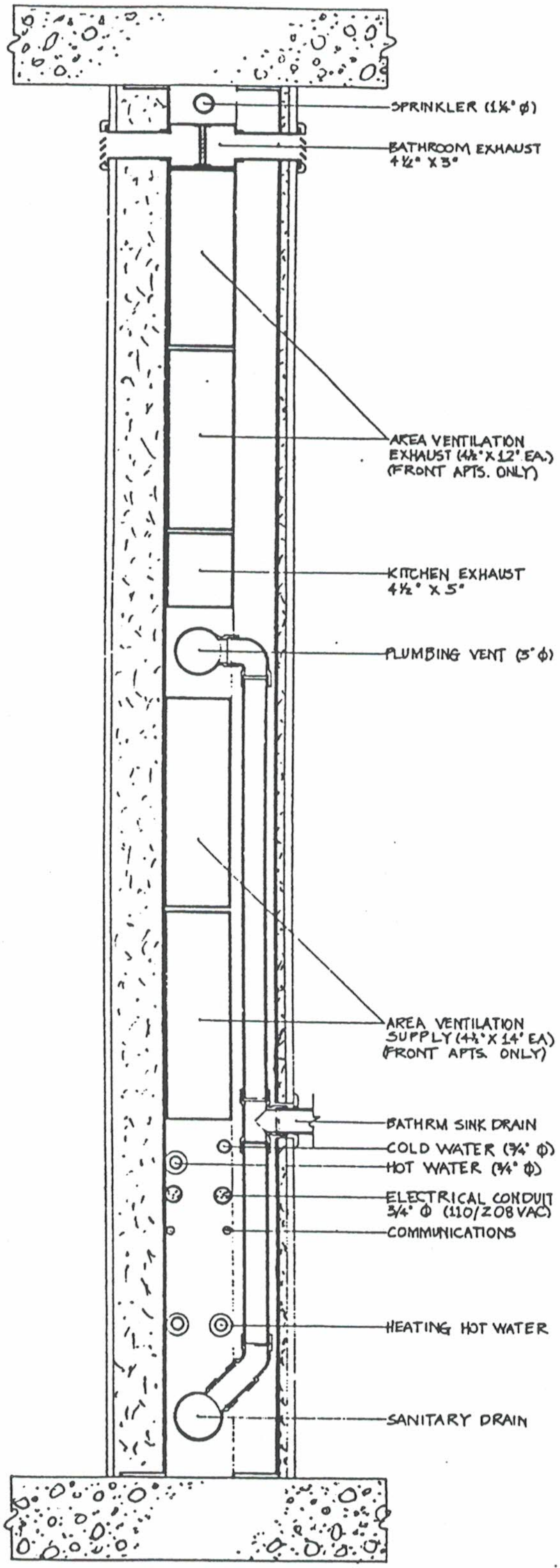

FIGURE 3

Section Through Mechanical Wall

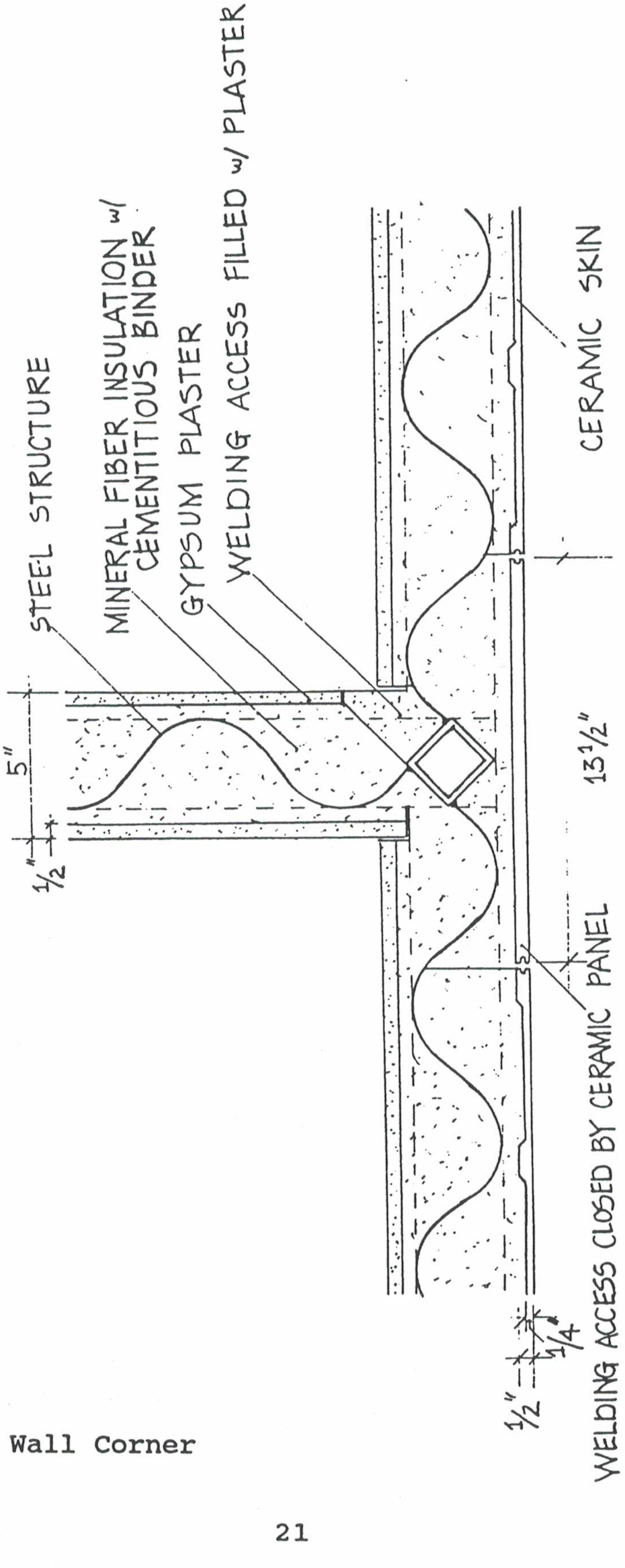

FIGURE 4

Section Through Wall Corner

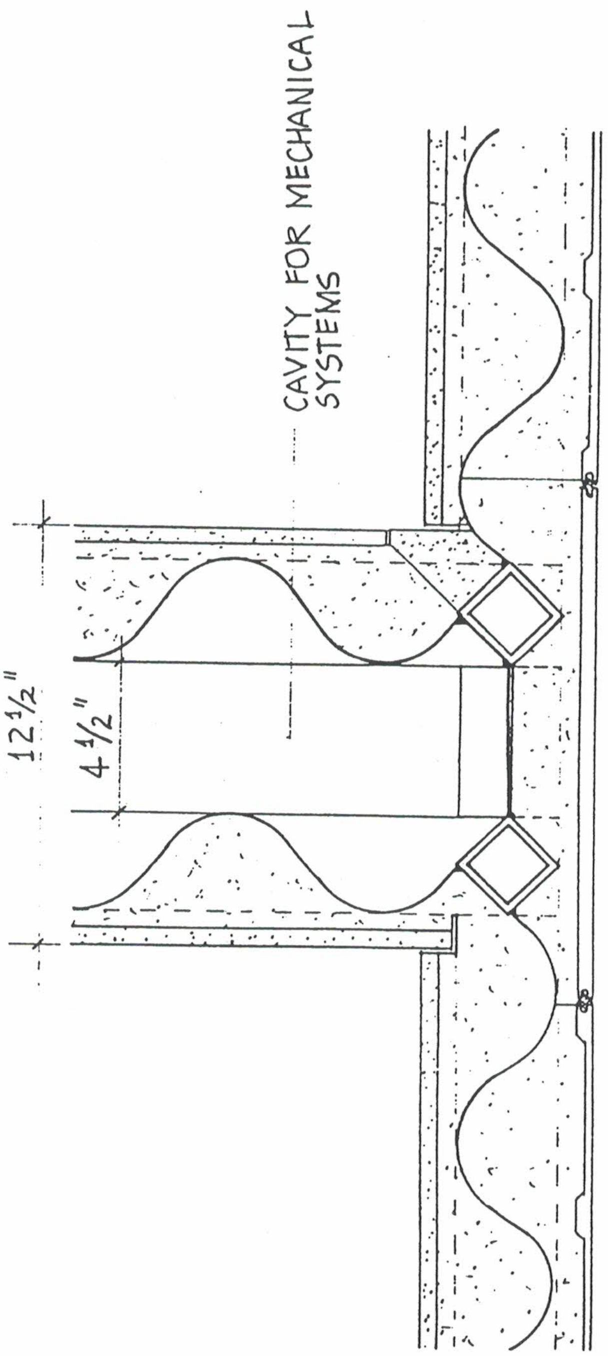

FIGURE 4A

Wall Corner (Mechanical Wall Variation)

The technology required to form the large ceramic panels (8 feet X 12 feet) is close to existing. Thin flat ceramic panels measuring two feet square and 1/4 inch thick have been successfully manufactured,[5] using an electrophoresis process instead of casting to form the green panels. A large scale operation would be able to mold panels using pressurized forming,[6] which reduces shrinkage during firing. Large kilns have been developed for the production of float glass, much larger than would be required to fire the panels; the major problem is evenness of heat to prevent warpage during firing. Special kilns with spot temperature controls are in use now.[7]

Construction Process

All finishes and detailing are applied to the components in the factory, so that few processes besides assembly are left to be done on the site. Following is a summary of work to be done in the field (please refer to Figure 5 for a schematic diagram of the central portion of the building assembly, and to Figure 1 for key construction details):

1) After being lifted into place, the components are welded to plates anchored in the slab (Figure 6) (all welding is performed with wire-feed inert gas welders);

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[5] Conversation with Brian and Edith Heath of Heath Ceramics in Sausalito.

[6] Baumgard, W., et. al., <u>Process Mineralogy of Ceramic Materials</u> (Elsevier: 1984), p. 14.

[7] Conversation with Brian Heath.

2) Components are welded to each other with connector plates (Figure 7), or using shear wall connection detail (Figure 7A) in the assembly of exterior shear wall elements;

3) Corrugated deck is installed and shear studs (see Figure 8) are welded through deck to top plates of walls (studs are made with an automatic "stop" to keep the welding plates at the critical distance from the bottom of the pan);

4) Wire mesh is laid and deck is poured with lightweight concrete, screeded flush with the top of the welding plates (building is now ready to receive the next floor's components);

5) One run of electrical conduit is made in recess of deck above to energize outlets in non-mechanical wall;

6) Connections are made to vertical utility runs in the mechanical chase at the hall (see Figure 9);

7) Ceiling is hung (Figure 10) by screwing into deck at locations where a plastic foam insert has been glued to metal deck prior to concrete placement (so we don't have to drive the screw into concrete);

8) Finish floor is laid;

9) Base molding covers interior welding access openings at floor;

10) Ceramic panels, bonded with cementitious foam, cover welding access openings in exterior of building;

11) Construction joints are caulked with silicone (refer to Figure 2).

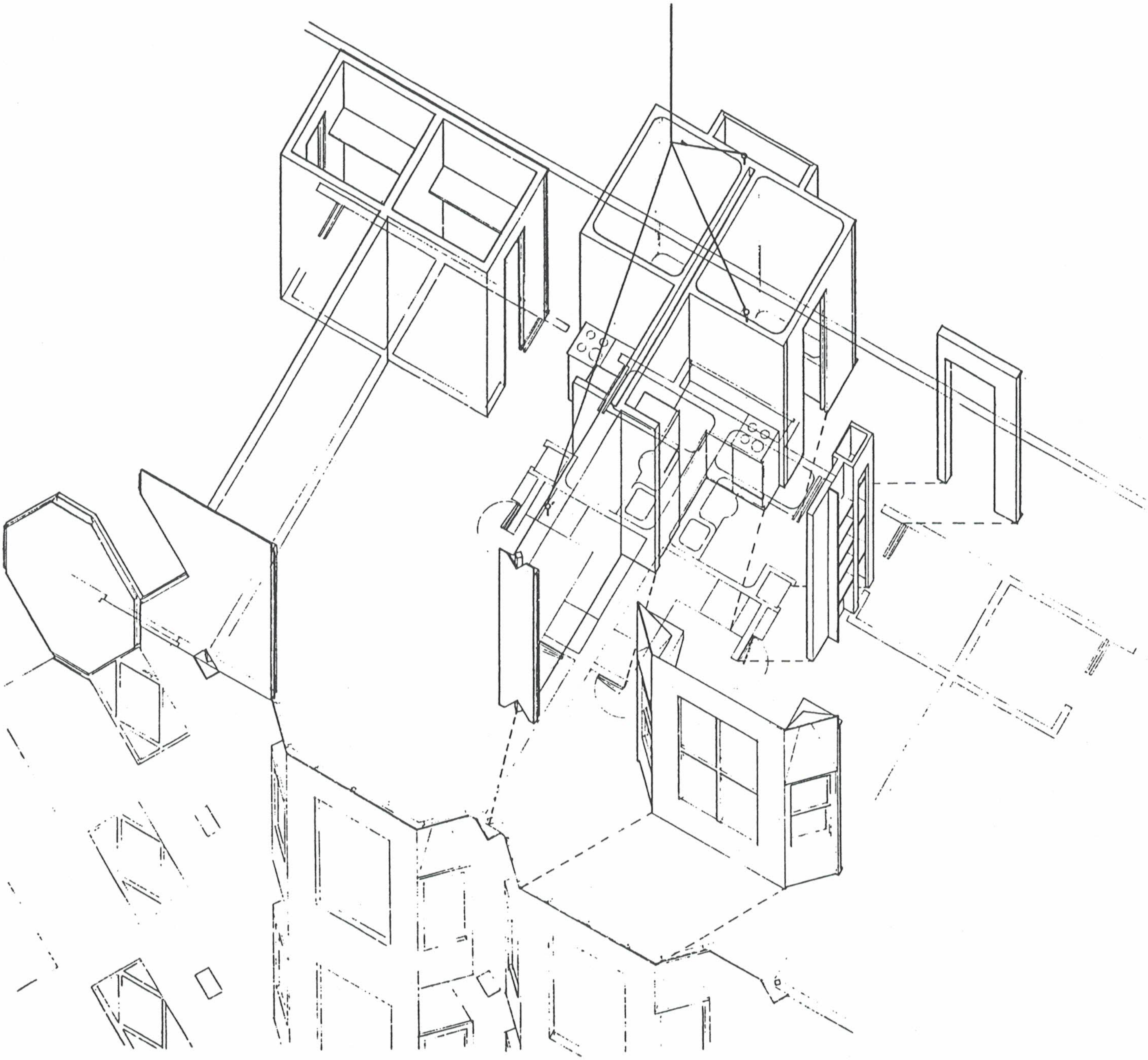

FIGURE 5

Exploded View of Building Components (440 Turk St.)

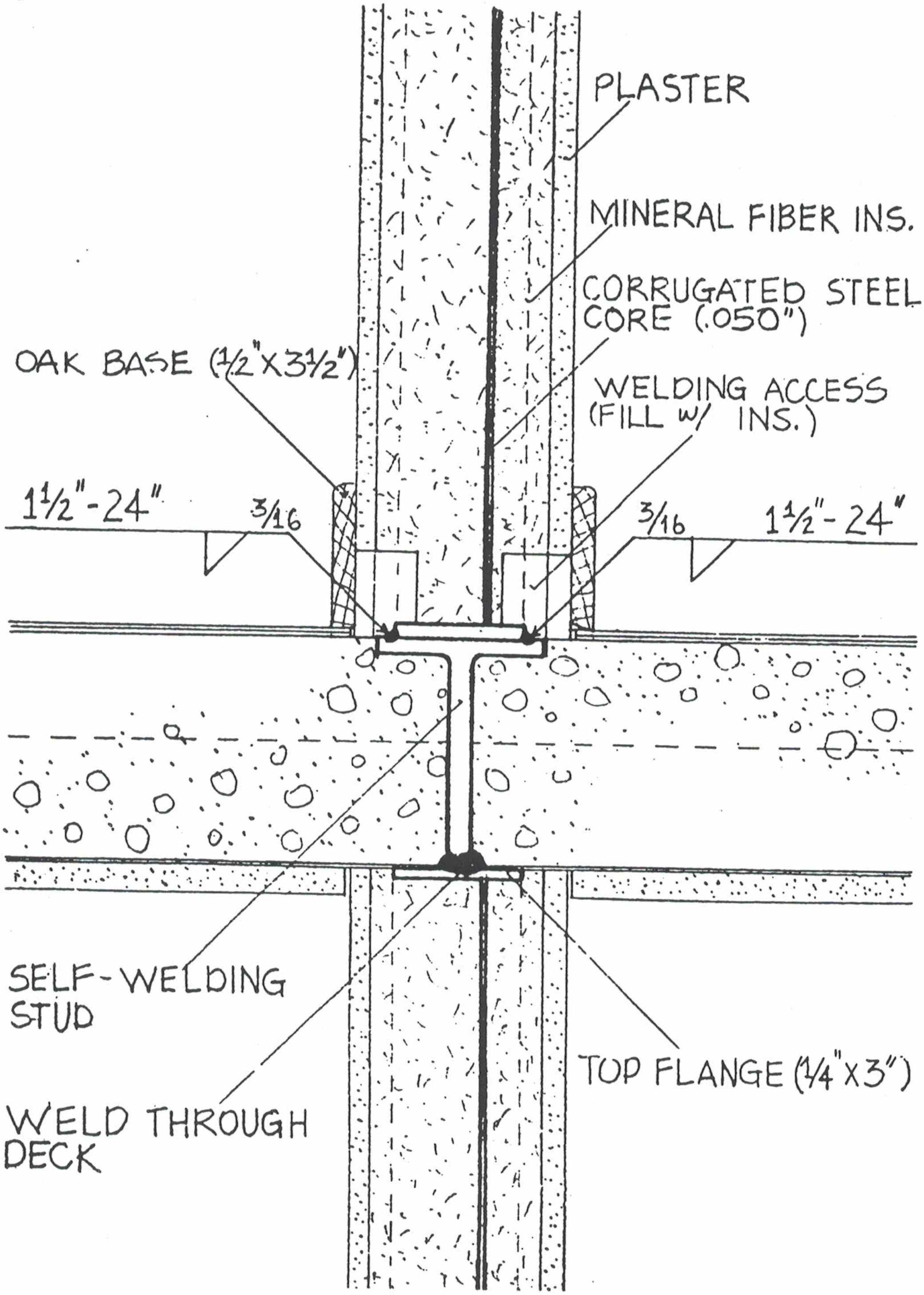

FIGURE 6

Through Slab Connections

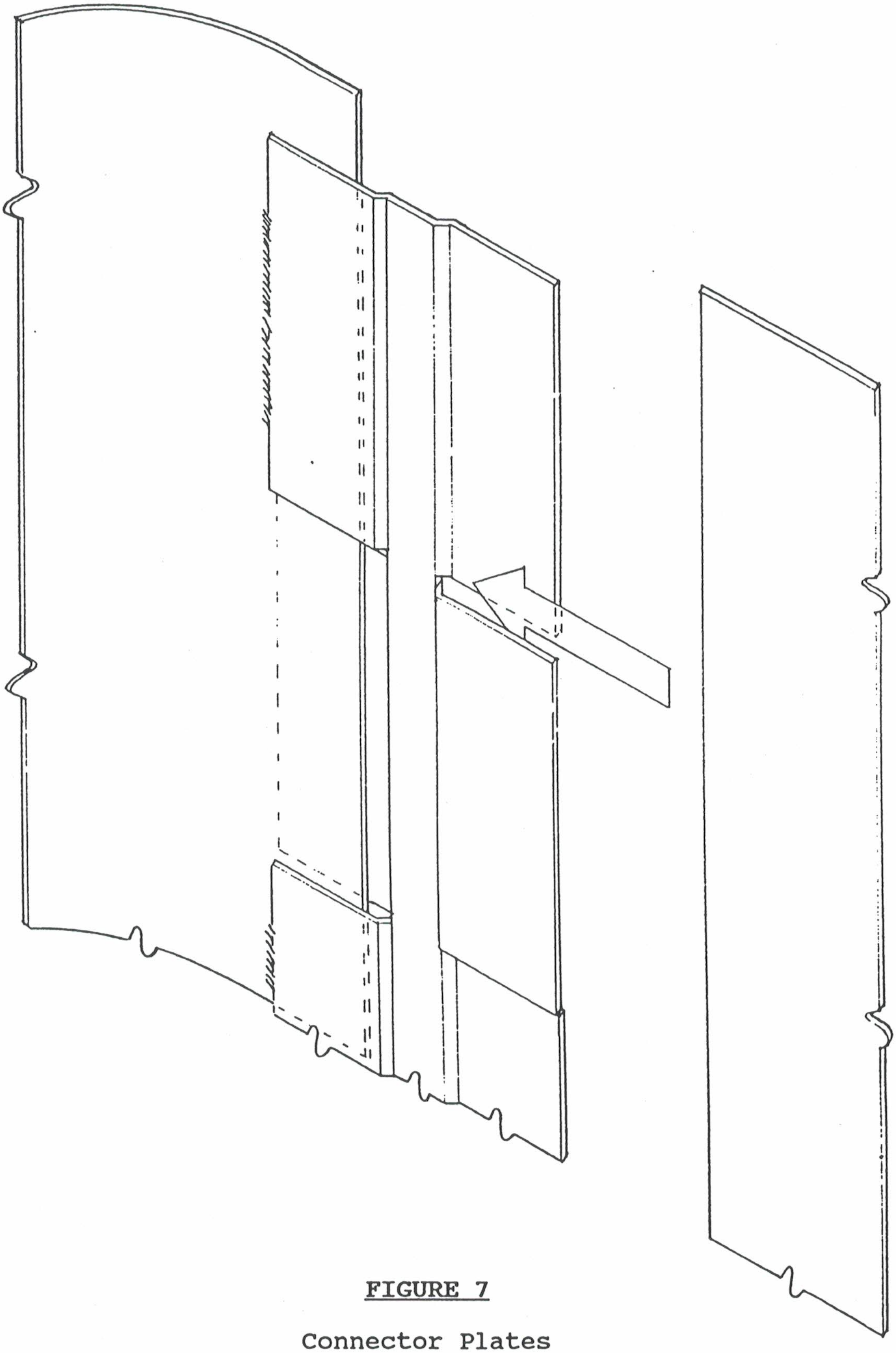

FIGURE 7

Connector Plates

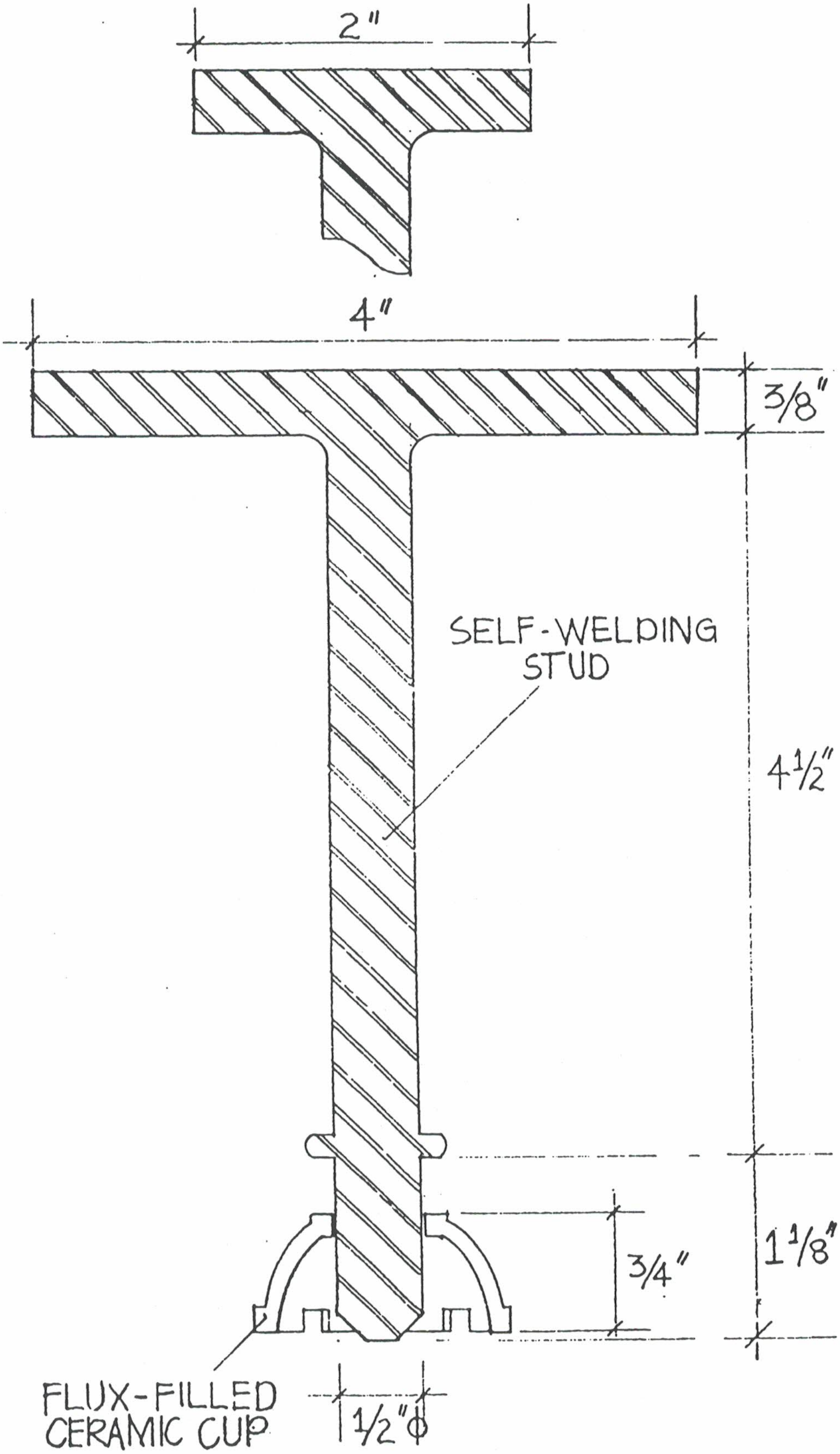

FIGURE 8

Shear Stud

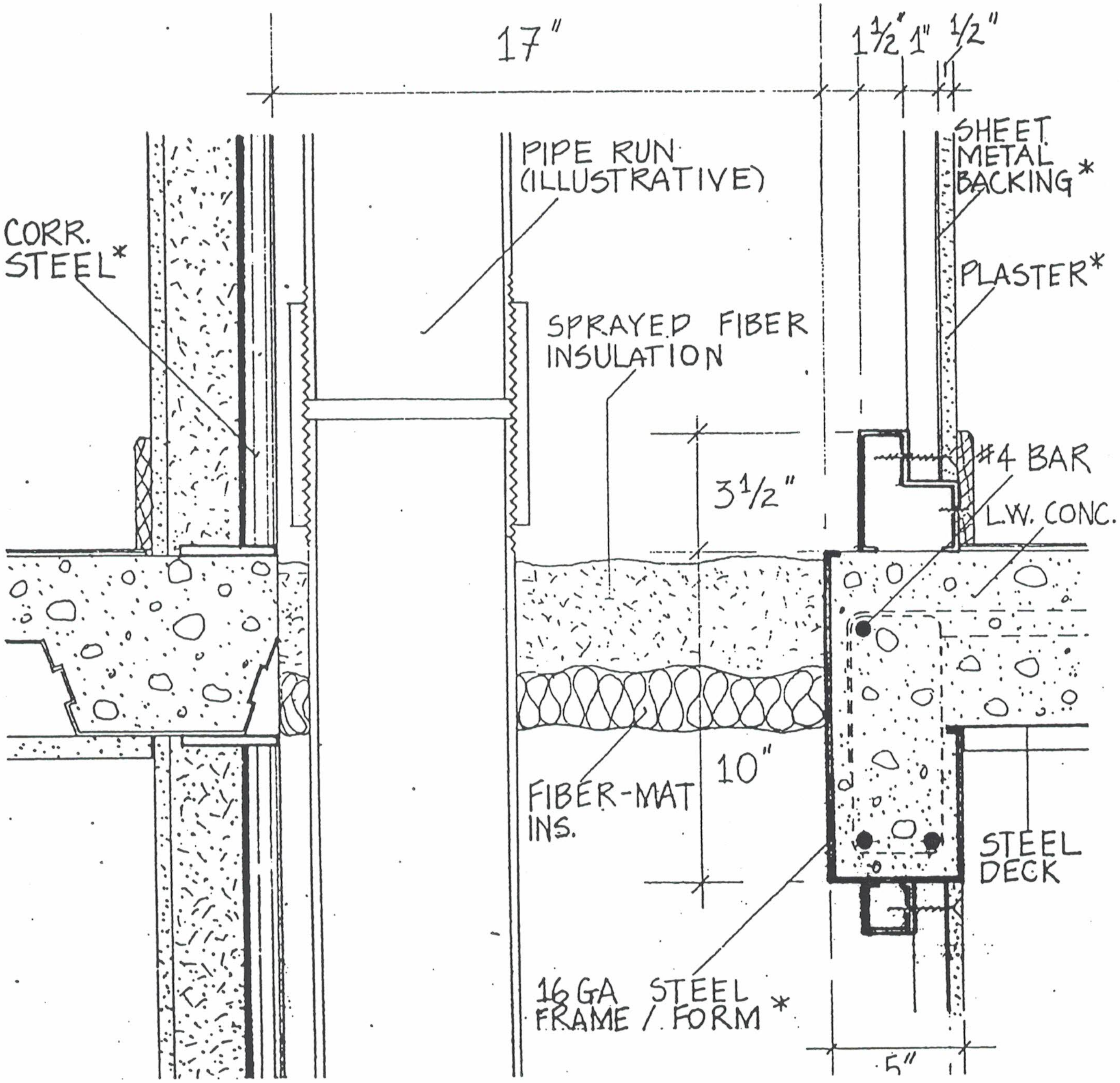

FIGURE 9

Mechanical Chase

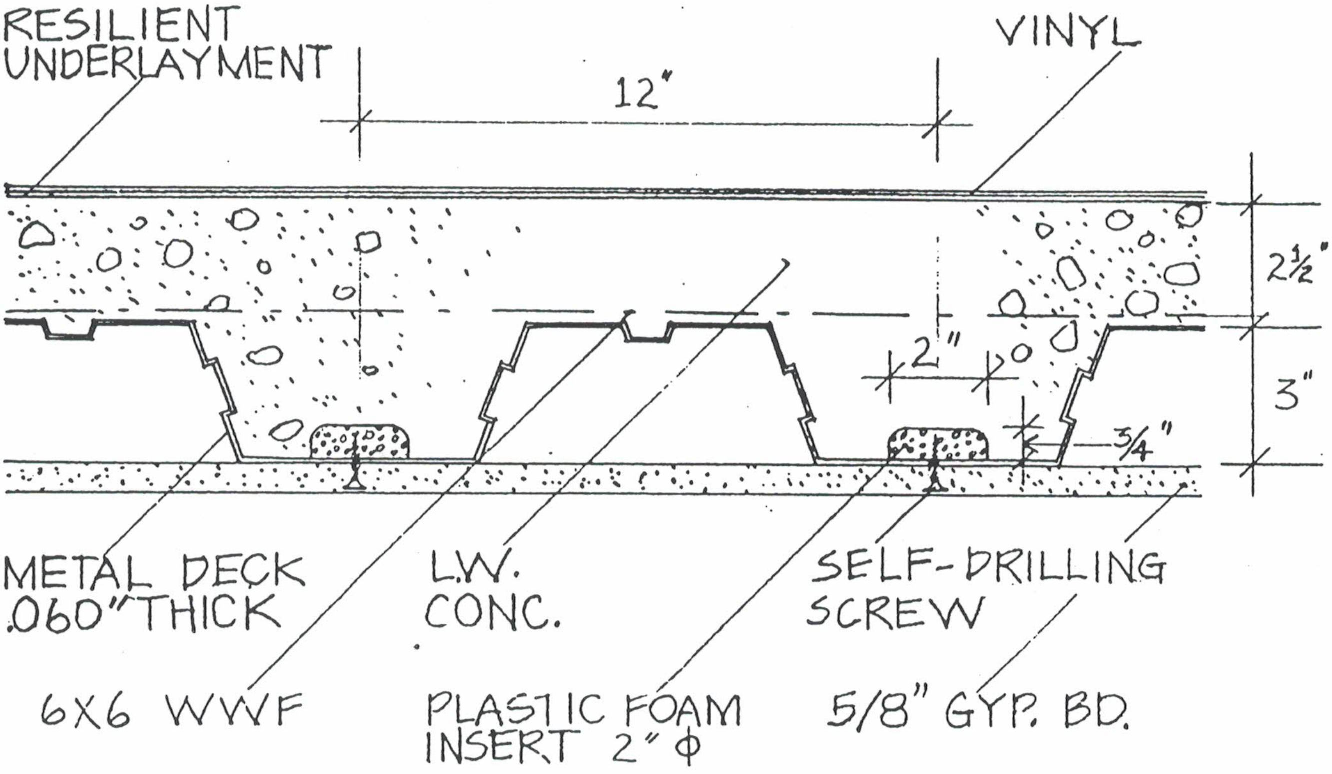

FIGURE 10

Section Through Deck Showing Ceiling

Installation Detail

Structural Characteristics

The structural performance of the building depends upon the nature of the corrugated steel wall panel, which is discussed in detail in the next section. Briefly, because the corrugated steel is flexible under lateral load, the building will as a whole be more flexible, and thus have a longer natural period of vibration than would be expected of a "box" system. This in turn will lead to smaller forces during an earthquake.

Welded joints are highly ductile, as they tend to fail in stages and have the capacity to form hinges, i.e., they get floppy before they break.

The weight of the structural system can be calculated from figures derived from Appendix A, Tables A5 and A6, which are based on the design for 440 Turk Street. In this case, the structural steel in the walls weighs 27.7 pounds per foot (pf); with 1215 feet of bearing wall per floor and a gross floor area of 8597 square feet, we have a little less than **four pounds per square foot** of steel for the vertical load bearing system. Furthermore, when we consider that if these walls were built as steel stud, non-bearing partition walls, they would have a steel weight of 16pf, we have a **net increase** of **1.7 pounds per square foot** of steel to make these walls carry the building!

Comparison With Concrete Systems

The only other system which is adaptable to prefabrication for cellular buildings is precast concrete. In all respects except for fire protection, the steel system compares favorably to concrete. First, concrete components require the construction of molds, which must be built and stored. By contrast, the steel system produces a variety of forms cybernetically. Steel reinforcing armatures for concrete require just as elaborate an assembly process as the corrugated steel panels, and are less adaptable to mass production techniques.

Precast concrete is joined in two ways:[8] by joining of steel plates anchored in the components, or by the casting of intermediate elements. The first method is less structurally integral than the welding of all-steel elements; the second requires that reinforcing be left sticking out of the precast element, and that a form be built on the site for the casting of the connecting elements. By contrast, the field connection system for the steel bearing wall system is simple and direct; it has been the standard technique for assembling ductile steel frames for decades.

The major drawback of concrete is weight. A four inch thick lightweight concrete wall of the same height as the walls for 440 Turk Street would weigh <u>284 pounds per foot</u> (pf), without gypsum surfaces. The assembled steel wall, with gypsum on both sides, weighs <u>95 pf</u>.[9] This means that for the largest component in the Turk Street system, the weight for concrete construction would be <u>ten and a half tons</u>, while the weight for the steel construction

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[8] ACI, <u>Design and Construction of Large Panel Concrete Structures</u>.

[9] See Appendix A, Table A5, below.

is only <u>four tons</u>.

CORRUGATED STEEL BEARING WALL ELEMENT

History

The key element in the structural system proposed is the bearing wall composed of a bounded, deformed metal sheet. Investigation of cold-formed steel elements is a relatively new area, having really gotten its start in the 1930's; corrugated sheet metal has been around for a much longer time--since 1784.[10] Most investigations have centered upon either column buckling, beam failure, local crushing, or horizontal diaphragm shear capacity. Although the last is related to shear in a bearing wall, the idea of using corrugated metal as a bearing wall does not appear to have been investigated, especially with boundary conditions in mind. One exception is the testing of a two-story stress-skin building, using "self framing" deep corrugated panels, at a nuclear weapons test site in 1955.[11] Another interesting related development is the Macomber PANLWEB girder[12] (Figure 11), which uses a trapezoidally corrugated sheet as a girder web. From the Macomber girder we can conclude that the corrugated sheet has significant shear capacity, although,

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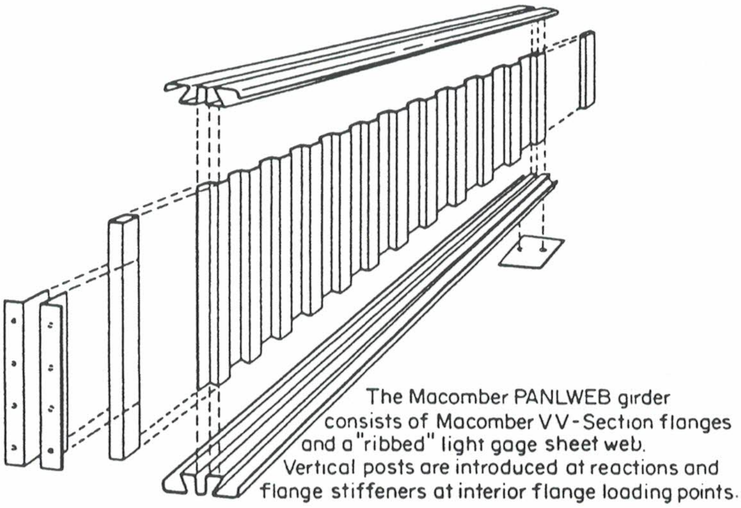
[10] Yu, Wei-Wen: Cold Formed Steel Design (John Wiley & Sons, 1985), p.439.

[11] *Ibid.*, p. 14.

[12] *Ibid.*, p. 441.



The Macomber PANLWEB girder
consists of Macomber VV-Section flanges
and a "ribbed" light gage sheet web.
Vertical posts are introduced at reactions and
flange stiffeners at interior flange loading points.

FIGURE 11

Macomber Girder (from Yu, Wei-Wen,

Cold Formed Steel Design (John Wiley & Sons: 1985), p. 441) according to Yu, the height of the panel has "considerable effect" on its shear characteristics.

Rationale

The choice of a corrugated panel, rather than some type of stress-skin sandwich panel, is based on an attempt to start with the simplest imaginable element, and to work from there. The corrugated panel obviously requires no welding or fabrication other than basic cold-forming, and if it can be found to be adequate, there would be a considerable savings in labor costs when compared to other, possibly more efficient, but more complex, assemblages.

Corrugated panels are made in two basic configurations: trapezoidal and continuously curved. Both deserve investigation, but this study will be limited to looking at the continuously curved section. Two arguments point towards the greater possibility of success with that choice: first, it seems intuitively obvious that a continuous section would perform better in shear than one with precipitous bends; second, the sine wave is the natural form for a cyclically-variable surface, and as such it seems to promise a minimized energy state (e.g., it may be the most energy-efficient form to produce). Continuously-curved corrugated panels are based on three different possible mathematical models: the sine curve, the parabolic curve, and the arc-and-tangent composite. All three are similar enough to be considered interchangeably for the purposes of this study; so, for the purposes of drawings, the arc and tangent model will be used, as it is the easiest to draw. Calculations, however, are based on the parabolic model, because the sectional properties of that form are mathematically more manageable.

The proposed prototype element (Figure 12) is a corrugated panel with a depth of 3", a full-cycle period of 9" and a thickness of 0.050", constructed from 36ksi steel. Based on the calculations attached, that section should be able to resist imposed vertical loads, plus axial loads due to seismic forces, at the bottom floor of a seven story structure of the type described elsewhere in the report. Its behavior under horizontal loads (i.e., in shear), is unknown. As Yu points out,

> "Because the structural performance of steel diaphragms usually depends on the sectional configuration of panels, the type and arrangement of connections, the strength and thickness of the material, span length, loading function . . . the mathematical analysis of shear diaphragms is complex."[13]

We can predict, however, that it will not perform as a rigid panel (as a sandwich panel, for example) might. It will have some flexibility ("The deeper profile is more flexible than are shallower sections"[14]), which has favorable implications for the period of the structure as a whole. The question of how deformations induced by horizontal loads would affect the capacity of the material to resist buckling under vertical loads is a big issue. Some areas in the panel would tend to flatten under the

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[13] Ibid., p. 385.

[14] Ibid., p. 387.

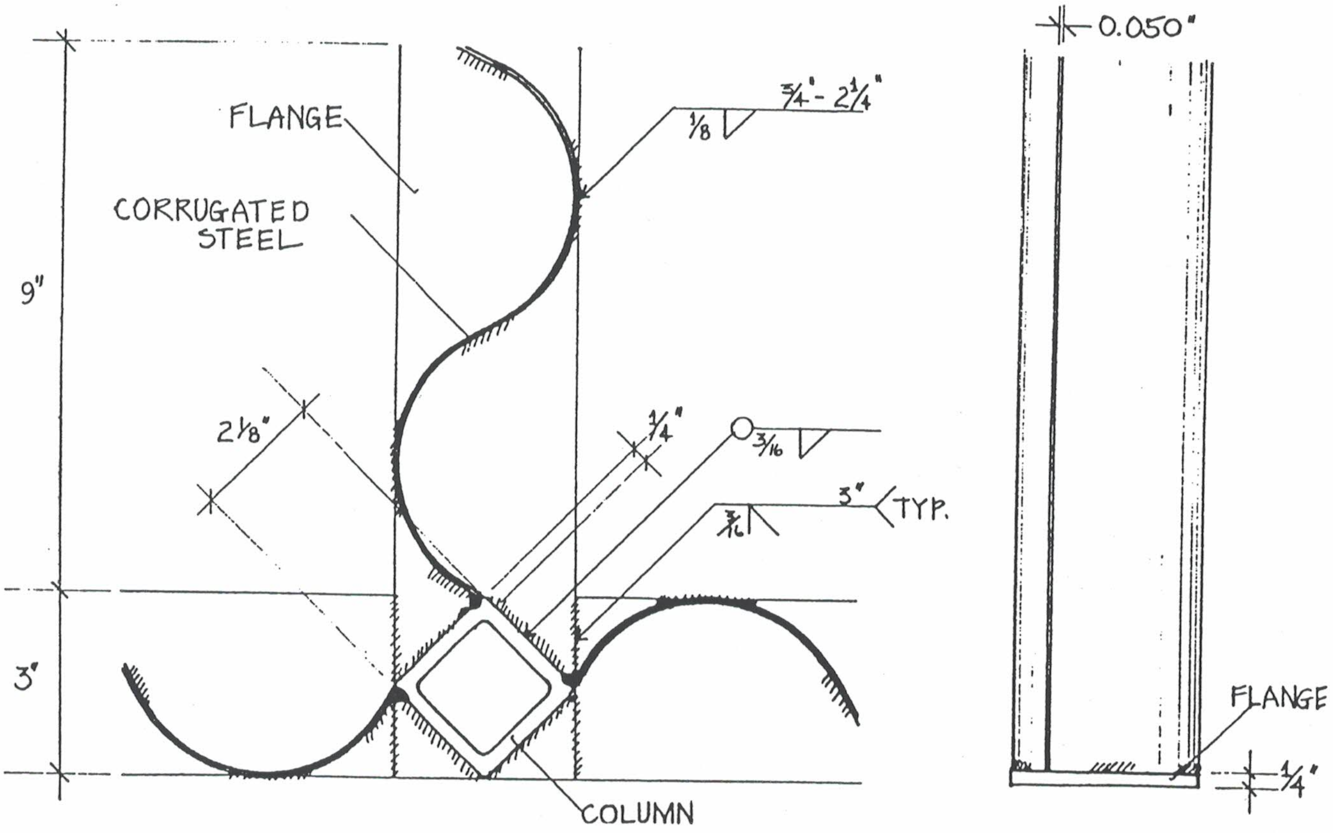


FIGURE 12

Corrugated Panel Cross-Section

influence of tensile forces, which would increase the local slenderness ratio; on the other hand, other areas, under the influence of compression, would increase their momement of inertia. The net effect might be a decreased capacity to bear vertical loads. It remains to be discovered whether simply thickening the guage of the steel, or increasing the depth of the corrugation would suffice to guard against this potential effect, or whether some form of applied transverse stiffening or diagonal bracing would be required. The most desireable correction would be one which would not remove the ability of the system to absorb some lateral load energy elastically.

Calculations

ALLOWABLE STRESS

From UBC section 2702(b)2, we are given a formula for allowable stress:

1) $$F_a = \frac{\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right] F_y}{F.S.}$$

where $$F.S. = 5/3 + 3/8\ \frac{(Kl/r)}{C_c} - 1/8\ \frac{(Kl/r)^3}{C_c^3}$$

and $$C_c = \sqrt{\frac{2\,(pi)^2 E}{F_y}}$$

(For $E = 2.9 \times 10^4$ ksi and $F_y = 36$ ksi, $C_c = 126$.)

SLENDERNESS RATIO

Allowable stress depends upon slenderness ratio (Eq. 1). If we choose $Kl/r = \underline{90}$, a conservative value, we get:

$$F_a = \frac{[1 - 0.5\ (90/126)^2]\ 36\text{ksi}}{5/3 + 3/8\ (90/126) - 1/8\ (90/126)^3}$$

$$F_a = \underline{14.2\ \text{ksi}},$$

a reasonable value.

REQUIRED MOMENT OF INERTIA

We have selected $Kl/r = 90$. Because we are conservatively modeling this element as a pin-connected column, $K = 1$.

We know that the floor-to-floor dimension is 8'-3", which gives us, after subtracting 6" for slab thickness, $l = 93"$. So,

$$r = \frac{93\ \text{in.}}{90} = \underline{1.03\ \text{in.}}$$

Since

$$r = \sqrt{\frac{I}{A}},$$

2) $$I_{req} = A_{req}\, r^2.$$

From an analysis of the building for which this system is being designed (Appendix A), maximum vertical load is <u>10.7 k/ft</u> (including axial load due to E.Q. horizontal forces = 1.2 k/ft.). So, we can find A_{req} from the known allowable stress:

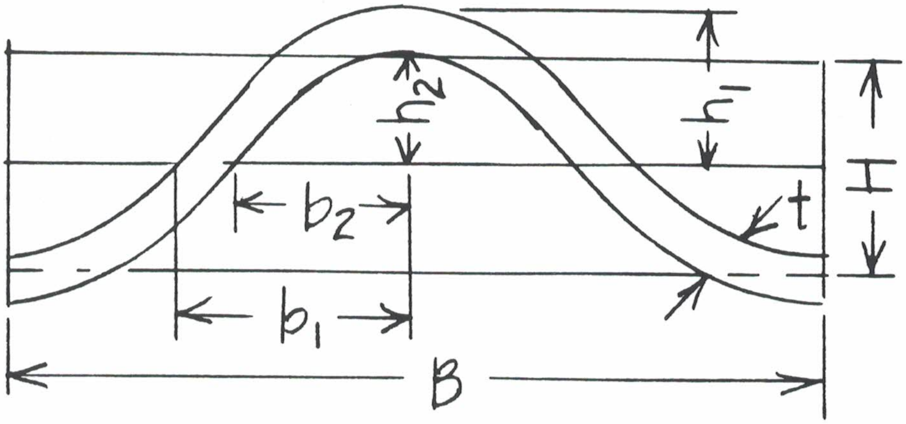
$$A_{req}/\text{ft} = \frac{10.7\ \text{k/ft}}{14.2\ \text{ksi}} = \underline{0.75\ \text{in}^2/\text{ft.}}$$

So, substituting in Eq. 2),

$$I_{req}/\text{ft} = 0.75\ \text{in}^2/\text{ft}\ (1.03\ \text{in})^2 = \underline{0.80\ \text{in}^4/\text{ft.}}$$

SECTION PROPERTIES

We will use a parabolic curve model for the corrugated panel cross-section, and seek values for the metal thickness (t), panel depth (H), and corrugation distance (B) (the distance from peak to peak along the cross-section).



We have the following formula for moment of inertia (per corrugation):[15]

3) $$I_{cor} = \frac{64}{105}(b_1h_1^3 - b_2h_2^3)$$

where

$$h_1 = 1/2\ (H + t),\ h_2 = 1/2\ (H - t),$$

$$b_1 = 1/4\ (B + 2.6t),\ \text{and}\ b_2 = 1/4\ (B - 2.6t).$$

In addition, we have[16]

4) $$A = \frac{t\ (2B + 5.2H)}{3}.$$

Therefore, since the length of one full corrugation is given by $L = A/t$,

5) $$L = \frac{2B + 5.2H}{3}$$

---

[15] Blake, Alexander, Handbook of Mechanics, Materials and Structures, p. 202.

[16] Ibid.

SECTION CHARACTERISTICS

Many corrugated sheet sections have a depth equal to 1/3 the distance of the corrugation. We therefore let

$$H = B/3.$$

From Eq. 4) above, we have

$$t = \frac{3A}{(2B + 5.2H)}$$

Since $A_{req} = 0.75\ in^2/ft = 0.0625\ in^2/in$, (substituting)

$$t_{req} = \frac{3\ (0.0625\ in^2/in)\ (B)}{2B + 5.2\ (B/3)}$$

$$t_{req} = \underline{.050\ in}.$$

(Note that t_{req} is independent of B because of the geometric characteristics of the section.)

We know that

$$I_{req} = 0.80\ in^4/ft. = 0.067\ in^4/in.$$

So, since $I_{cor} = I_{req}\ (B)$,

$$I_{req} = \frac{I_{cor}}{B} = \frac{64}{105(B)}\ \{\left[\frac{B + 2.6(.05in)}{4}\right]\ \left[\frac{B/3\ +\ .05in}{2}\right]^3$$

$$-\ \left[\frac{B\ -\ 2.6\ (.05in)}{4}\right]\ \left[\frac{B/3\ -\ .05in}{2}\right]^3$$

$$= \frac{2\ (3)}{105\ (B)}\ [(B/3 + 0.13/3\ in)(B/3 + .05in)^3$$

$$-\ (B/3 - 0.13/3\ in)(B/3 - .05in)^3]$$

We can express the terms in parentheses as

$$[(x+y)(x+z)^3 - (x-y)(x-z)^3]$$

where

$$x = B/3,\ y = 0.13/3\ in,\ \text{and}\ z = .05in.$$

Expanding,

$$(x+y)(x+z)^3 - (x-y)(x-z)^3$$

$$= (x+y)(x^3+3x^2z+3xz^2+z^3) - (x-y)(x^3-3x^2z+3xz^2-z^3)$$

$$= 6x^3z + 2xz^3 + 2yx^3 + 6yxz^2$$

$$= x^3(6z + 2y) + x(2z^3 + 6yz^2)$$

Resubstituting for x, y and z,

$$I_{req} = \frac{6}{105\ (B)} \{(B/3)^3\ [6(.05in) + 2(0.13/3\ in)]$$

$$+ (B/3)\ [2(.05in)^3 + 6(0.13/3\ in)(.05in)^2]\}$$

$$= (B/3)^2\ \{\frac{2}{105}\ [6(.05in) + 2(0.13/3\ in)]\}$$

$$+ \frac{2}{105}\ [2(.05in)^3 + 6(0.13/3in)(.05in)^2]$$

We know that: $I_{req} = .067\ in^4/in$; so,

$$B = 3\ \left\{\frac{.067\ in^4/in - 2/105\ [2(.05in)^3 + 6(0.13/3\ in)(.05in)^2]}{2/105\ [6(.05in) + 2(0.13/3\ in)]}\right\}^{1/2}$$

$$B = \underline{9.06\ in}.$$

Then, also,

$$H = B/3 = \underline{3.02\ in}.$$

This section, a panel 3" deep and about 1/20" in thickness, is highly desireable, since it is lightweight and allows us a total wall thickness of 5".

SHEAR REQUIREMENTS

Based upon an analysis of the 440 Turk St. building (see Appendix A, "Earthquake Load Calculations"), total shear due to earthquake loads (V) at the base of the seven stories of residential units is 633k.

From UBC Section 2702(b)2, we are given allowable shear stress (F_v):

$$F_v = 0.040\ F_y.$$

Since F_y = 36ksi, we have:

$$F_v = \underline{14.4\text{ksi}}.$$

For the whole structure, then, the area of steel required in the cross-section of the walls is:

$$A_{req} = \frac{V}{F_v} = \frac{633\text{k}}{14.4\text{ksi}} = 44.0\ \text{in}^2.$$

We make the assumption that for resisting shear, the effective area of the corrugated cross-section is the length of the curve projected onto the line of the lateral force (i.e., the simple linear length of the wall) times the thickness of the steel (0.050"). On the level in question, we have a total length of bearing wall of 447.5 feet which is oriented parallel to the lateral force. Therefore, we have an available area of steel of:

$$A = (447.5\ \text{ft})\ (12\ \text{in/ft})\ (0.050")$$
$$= \underline{268.5\ \text{in}^2} > \underline{44.0\ \text{in}^2}.$$

WELDING

The effective area of the fillet welds connecting the base flange of the walls to the slab is given by the product of the throat width (t) times the length of the weld (UBC Section 2702(e)). Using single-pass welding with 3/16" effective throat width, and asuming that allowable stress is 0.40 F_y[17] (= 14.4ksi) (UBC Table 27-B), we have:

$$L_{req} = \frac{633\text{k}}{14.4\text{ksi}\ (3/16\ \text{in})}$$

---

[17] Yu, op. cit., allows 0.75 F_y.

$$= \underline{234\ \text{in.}}$$

Distributed over 447.5 feet of wall, we find the number of inches of weld per foot of wall required:

$$\frac{234\ \text{in}}{447.5\ \text{ft}} = \underline{0.52\ \text{in/ft.}}$$

Since the design provides about <u>1 inch per foot</u> welding, we have a safety factor of 2.

SHEAR STUDS

Given 3.5ksi concrete, for 1/2" diameter studs the allowable load per stud is 5.5k (<u>UBC</u> Table 27-C). So the number of studs required (n) is given by:

$$n = \frac{V}{5.5k} = \frac{633k}{5.5k} = 115.$$

Distributed over 447.5 feet, we have a spacing (D) of:

$$D = \frac{447.5\text{ft}}{115} = \underline{3.9\ \text{ft}}.$$

So, our design spacing of 2 feet gives a safety factor of approximately 2.

Physical Test Description

The only way to determine the viability of the corrugated steel bearing wall is to physically test models of the element. This test proposal would constitute a preliminary investigation.

Several test models of the corrugated wall system would be built in 1/3 scale, assembled from available steel components by welding. The resulting pieces will measure 31" in height, approximately 36" in length (depending upon width of corrugated sheet available) and 1" in depth of corrugation. Three tests will be performed on the elements.

Buckling

The scale model of the element will be tested in a frame to determine its resistance to vertical loads. By "vertical loads" we mean uniform loads applied to the top flange of the assembly and directed parallel to the corrugations.

To determine the relationship between loads imposed on the test model, and the performance of the actual full-scale element, we need to find the theoretical proportion between the buckling strengths of the two. Buckling strength, in terms of allowable stress, is directly proportional to the square of the slenderness ratio:

$$F \sim (l/r)^2$$

In terms of imposed load, P,

$$P \sim FA \sim (l/r)^2 (A),$$

where A is the cross-sectional area of the element being tested. The slenderness ratio (l/r) is a dimensionless quantity and will remain constant when scaling down. So, since $A \sim s^2$ (where "s" is the linear scaling factor = 1/3), we have a quadratic relationship (equal to a factor of 1/9 in this case) between the load applied to the test element and the equivalent load applied to the actual element.

We should expect, then, that the element would fail at loadings greater than 1/9 the allowable load predicted for the full-scale element (= (10.7 k/ft)(1/9) = <u>1.2 k/ft</u>). For a length of 3 feet, the element would be expected to withstand a load of 3.6 kips, and, since F.S. = 1.9, to yield at a load of <u>6.8 kips.</u>

<u>Elastic deformation under horizontal load (shear)</u>

A 1/3 scale model of the element will be placed in a jig so that the lower flange is continuously pinned, as it would be when welded into place in actual construction, and so that the top flange is restrained from moving out of the plane of the element, but allowed to move laterally. Loads will be applied to one top corner of the assembly in a direction across the corrugations, as they would be applied inertially by the floor diaphragm in an earthquake situation. Under these conditions, it is expected that two types of deformation would occur. One is a form of local elastic buckling--in certain areas the cross-section of the corrugations would be modified, becoming deeper in some places and shallower in others. This deformation is expected to be complex, and should be observed and recorded photographically if that is feasible. It is not expected that any method for measuring these deformations will be useful.

The second form of deformation is linear--the movement of the corner to which the load is applied. This movement will be recorded at different levels of loading, and the return noted. Data will thus be developed to determine the level of loading at which the behavior of the assembly becomes inelastic, and, if possible, where and at which level of loading the assembly yields.

Combined axial and horizontal loads

This test is the most critical in establishing the viability of the bearing wall system. What should be tested is the effect that deformations due to horizontal loading have upon the buckling resistance of the element. This test would require the construction of several model elements, and the successive loading to failure of each of them under different degrees of strain induced by different levels of horizontal loading. The exact schedule of loading would be developed based upon the two experiments described above. From the data, a comparison to the system's performance under axial load alone and to its performance under combined load can be developed.

APPLICATION OF THE SYSTEM TO 440 TURK STREET

Building Description

Figures 13 through 20 illustrate the design of 440 Turk Street. Two floors of offices, designed on a semi-open plan, have their entry on the sidewalk towards the east side of the elevation. Basement parking is entered just to the east of the office doors. Seven floors, containing 89 elderly low-income apartments, the bulk of which are studios, are entered at the ground level through the entry court. The entry court is a secured garden off the street, but visually accessible to pedestrians and drivers on Turk Street. A portion of the ground floor is used as the sitting and meeting rooms for the elderly residents.

The entry court, as well as two smaller interior courts serving the offices, are an attempt to bring light and air to the users of the building (see Section A-A, Figure 15). By utilizing translucent partitions within the offices, there is no portion of the plan which is further than 30 feet from some source of natural light. The entry court is used visually by office workers, not only directly from the offices overlooking it, but as seen from the lunchroom balcony. Since the street elevation of the building faces south, the entry court will be sunlit during the middle of the day. It's major function is to provide a private park for the residents, and to form a quiet transition from the street to their homes.

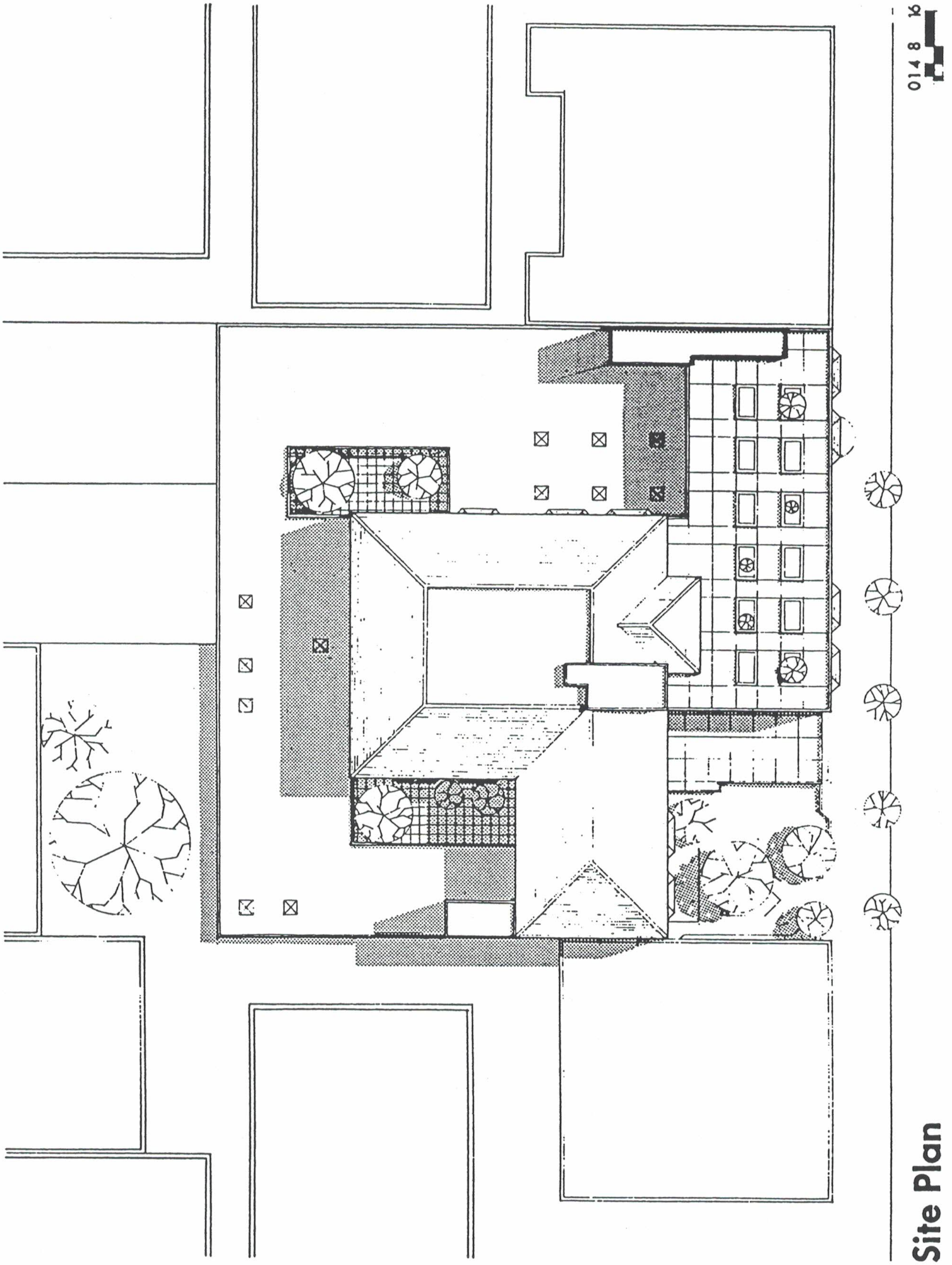
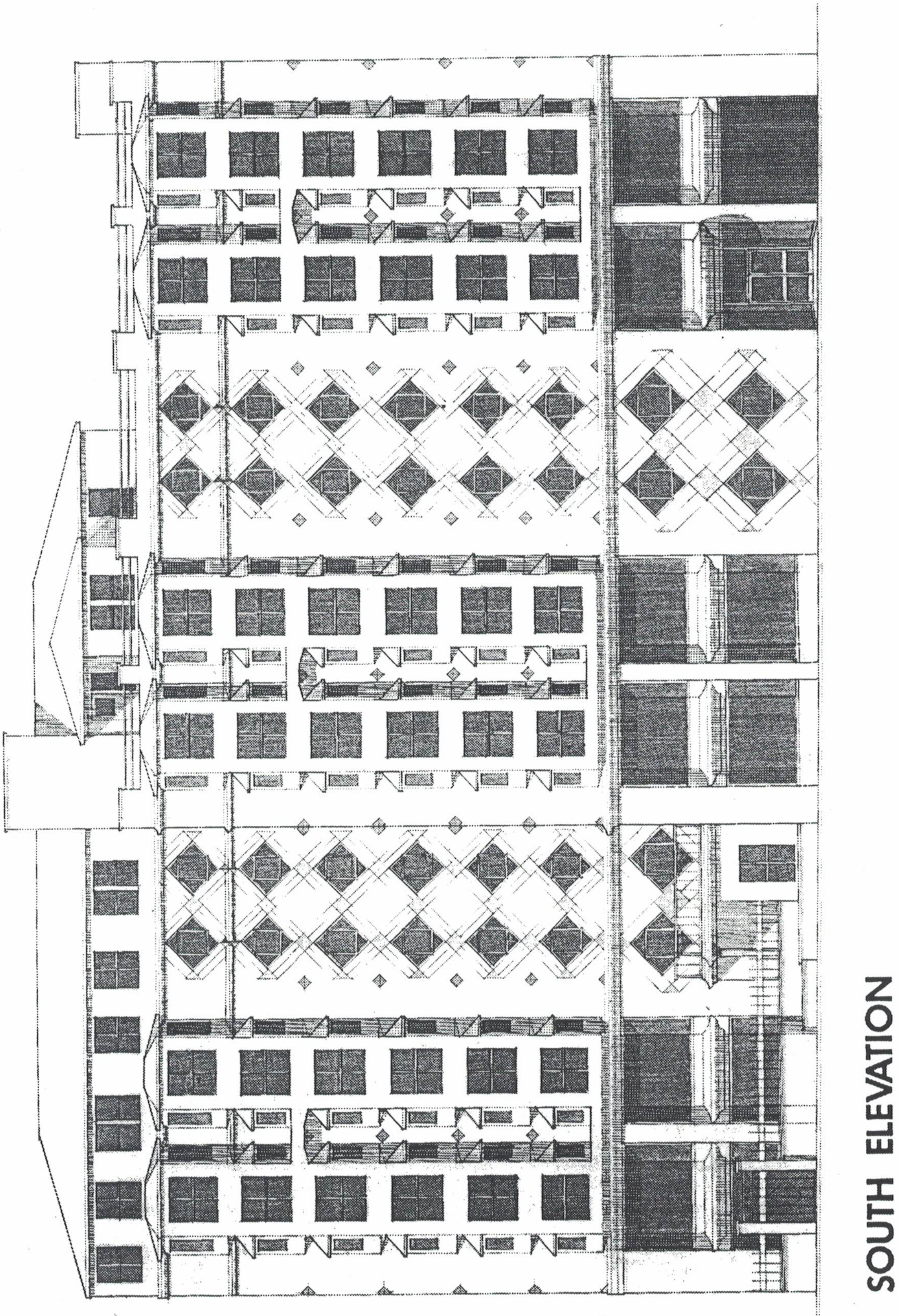


FIGURE 13
440 Turk Street, Site Plan



<u>FIGURE 14</u>
South Elevation

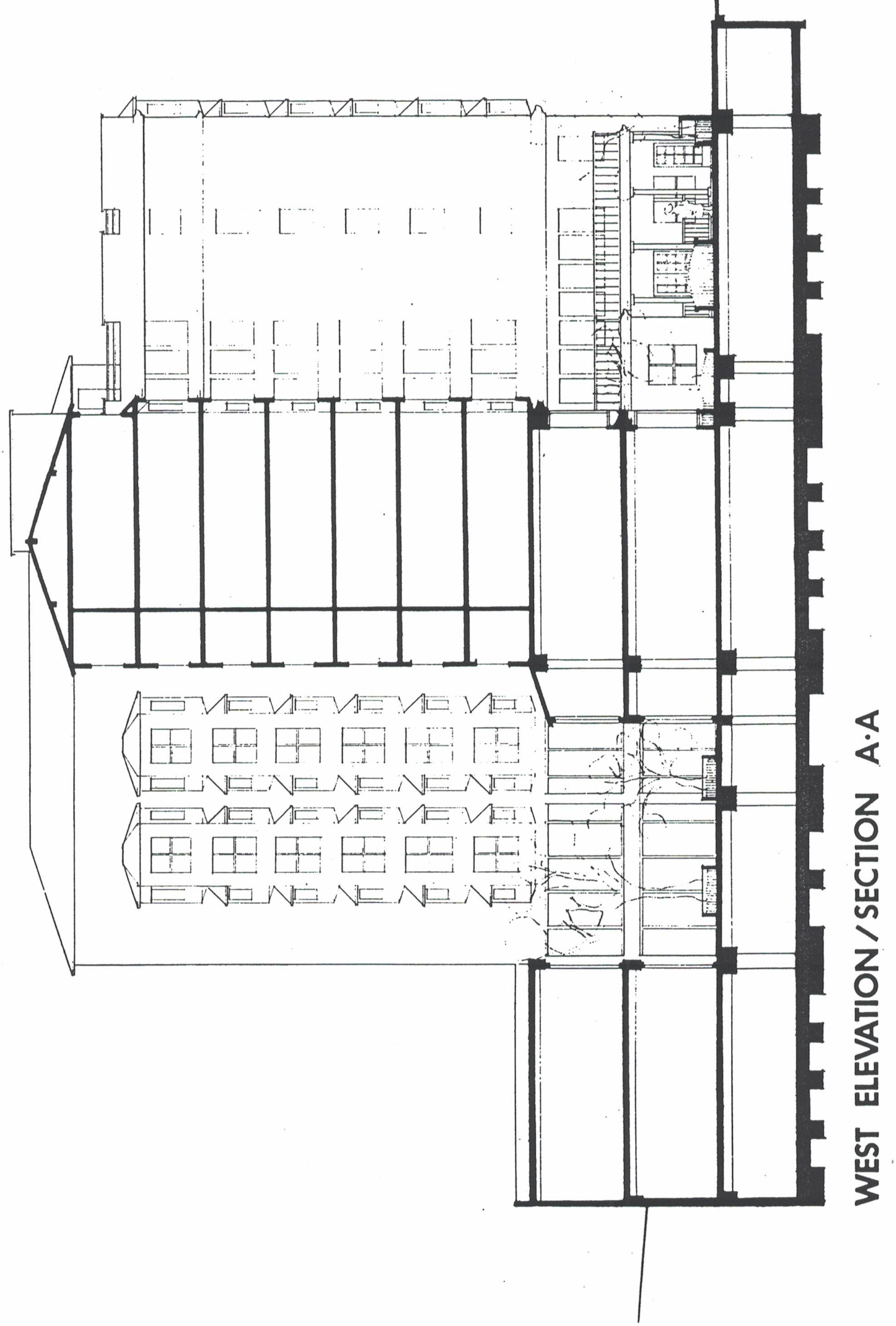


FIGURE 15
Section A-A

Entry Court Plan

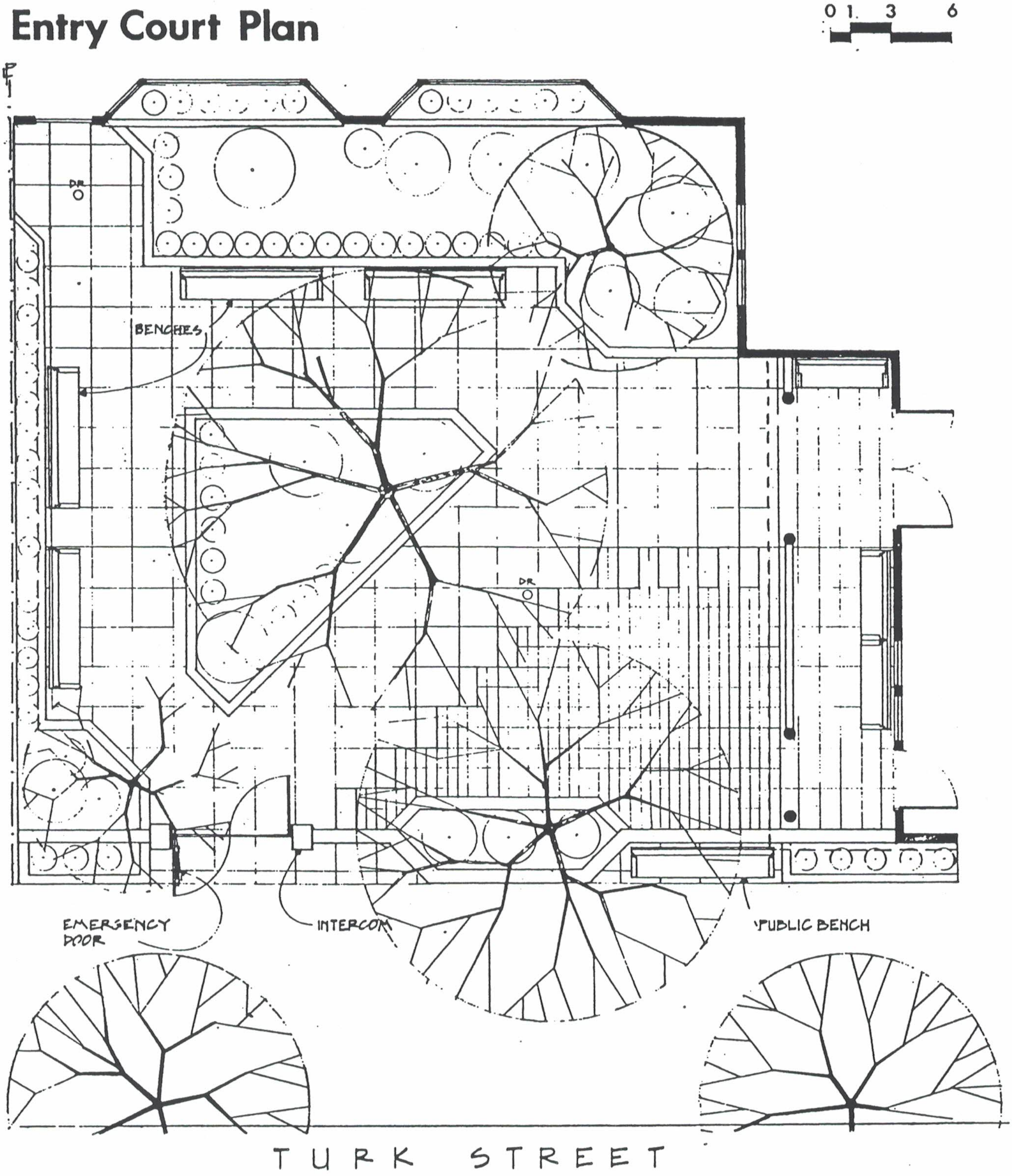
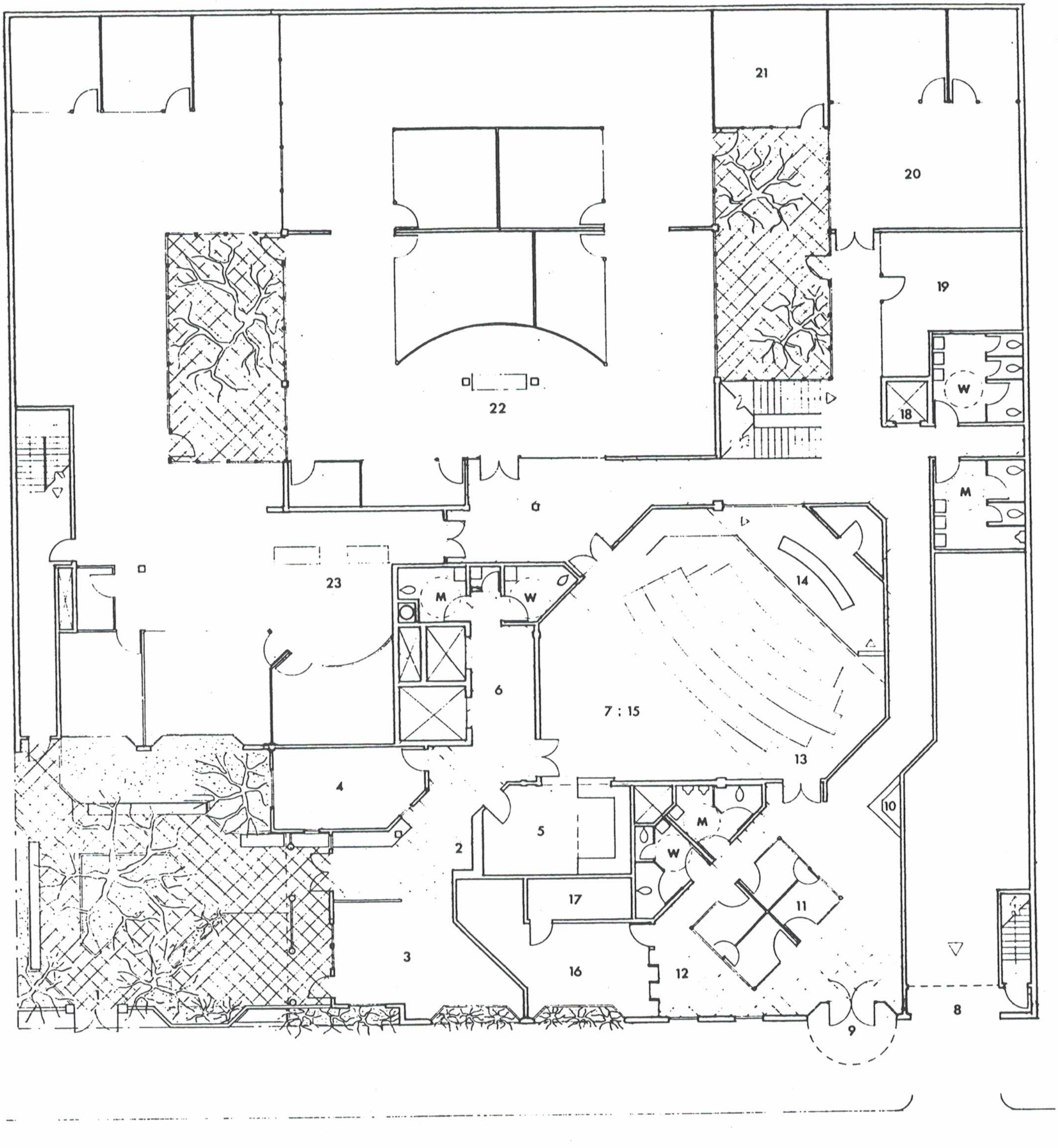


FIGURE 16
Entry Court Plan



GROUND FLOOR

FIGURE 17
Ground Floor Plan

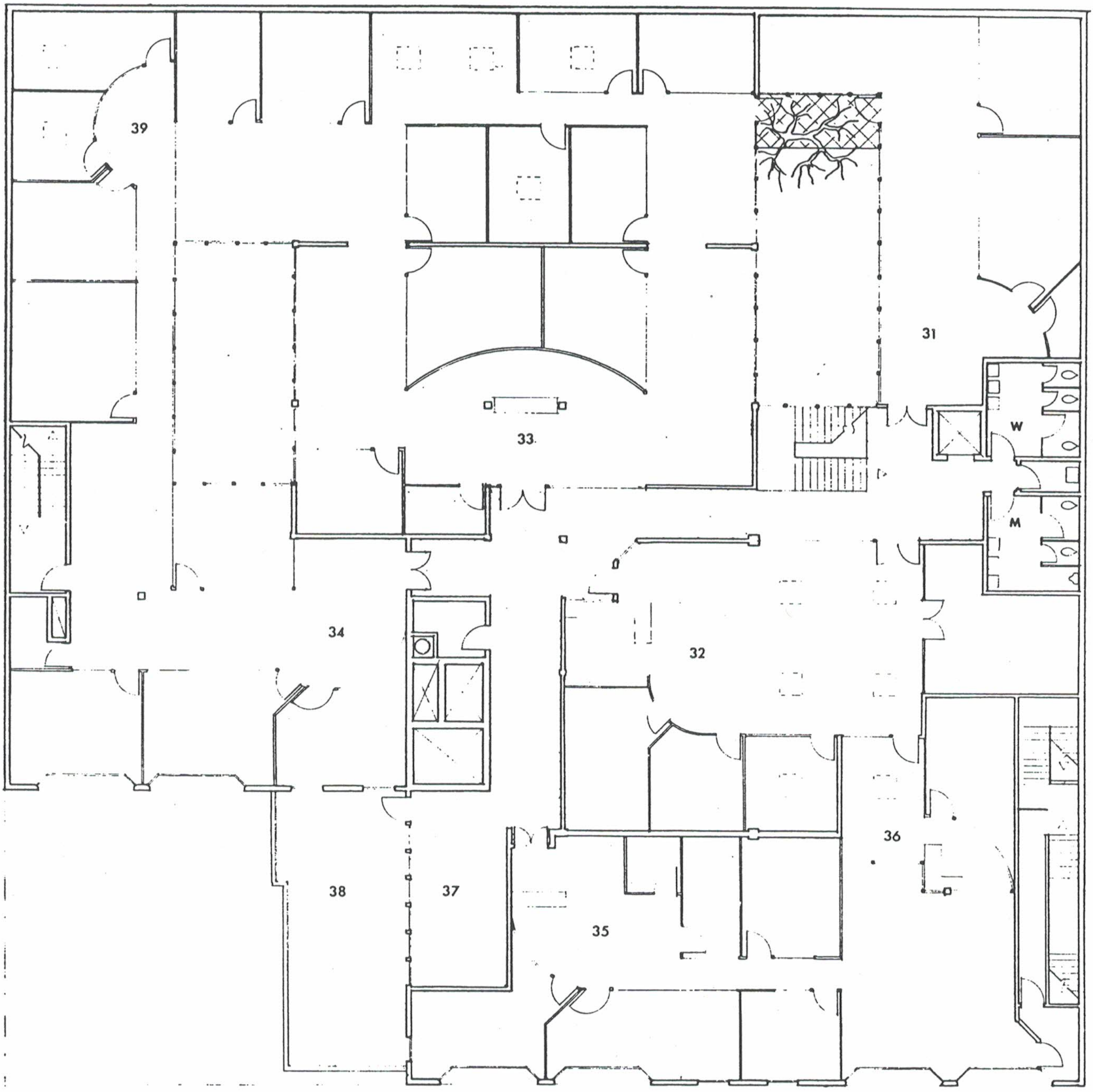
GROUND FLOOR

1	Gate to entry court
2	Mailboxes
3	Sitting room
4	Meeting room
5	Kitchen/Meeting room
6	Residential elevator lobby
7	Residents' large mtg. room
8	Garage entry
9	Housing office entry
10	Information/security
11	Interview rooms
12	Cashiers' windows
13	Commission room audience seating
14	Commission dais
15	Overflow seating
16	Office--Cashier
17	Vault
18	Office elevator
19	Copy/mail room
20	Office--Personnel
21	Conference room
22	Office--Leased housing
23	Office--Housing management

SECOND FLOOR

31	Office--Special Programs
32	Office--Data Processing
33	Office--Finance
34	Office--Executive
35	Office--Planning
36	Office--Family Services
37	Staff lunch room
38	Lunch balcony
39	Office--Legal

FIGURE 17A
Key to Ground and Second Floor Plans (Figs. 17 & 18)



SECOND FLOOR

FIGURE 18
Second Floor Plan

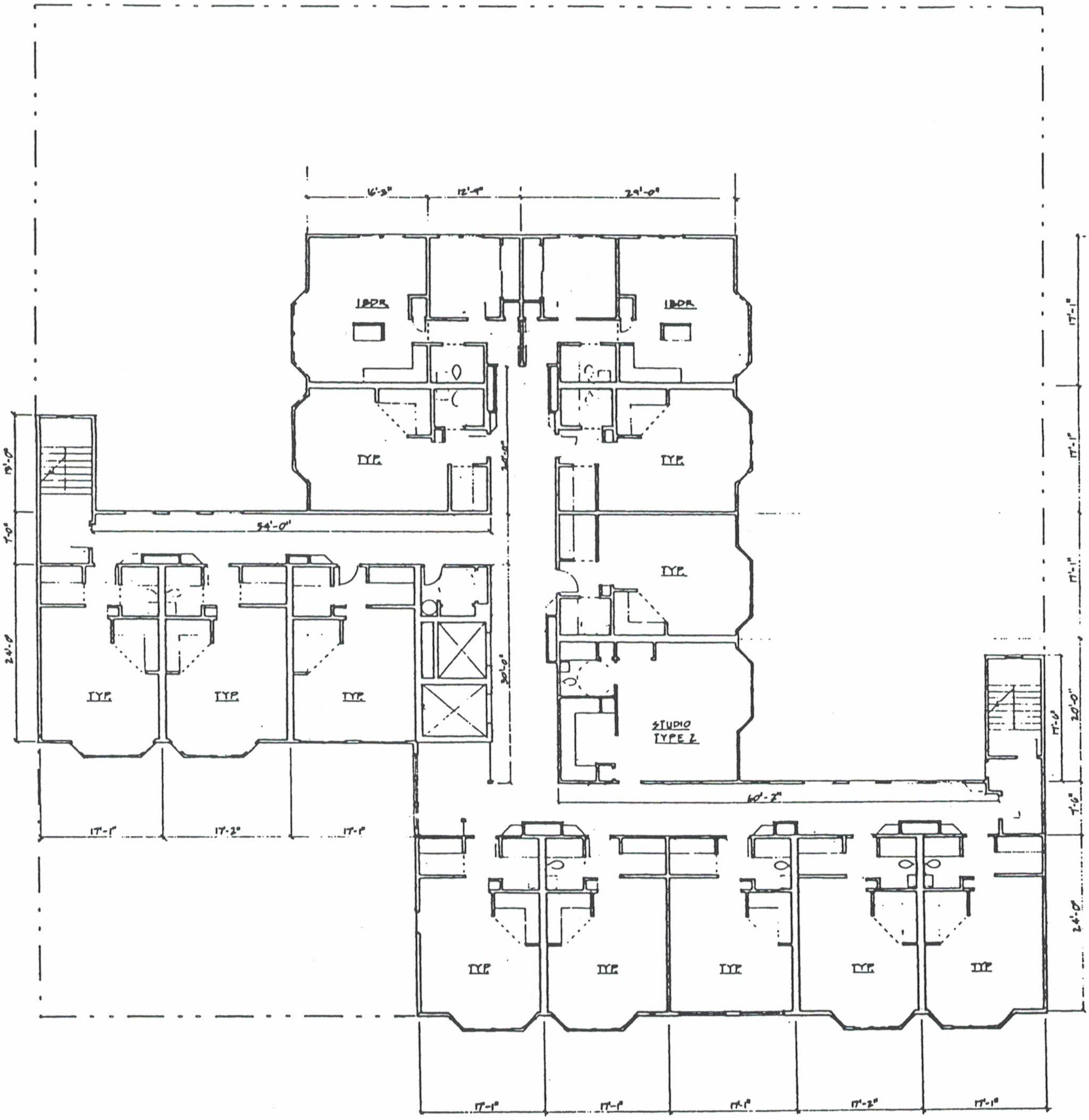


FIGURE 19
Third through Eighth Floor Plan

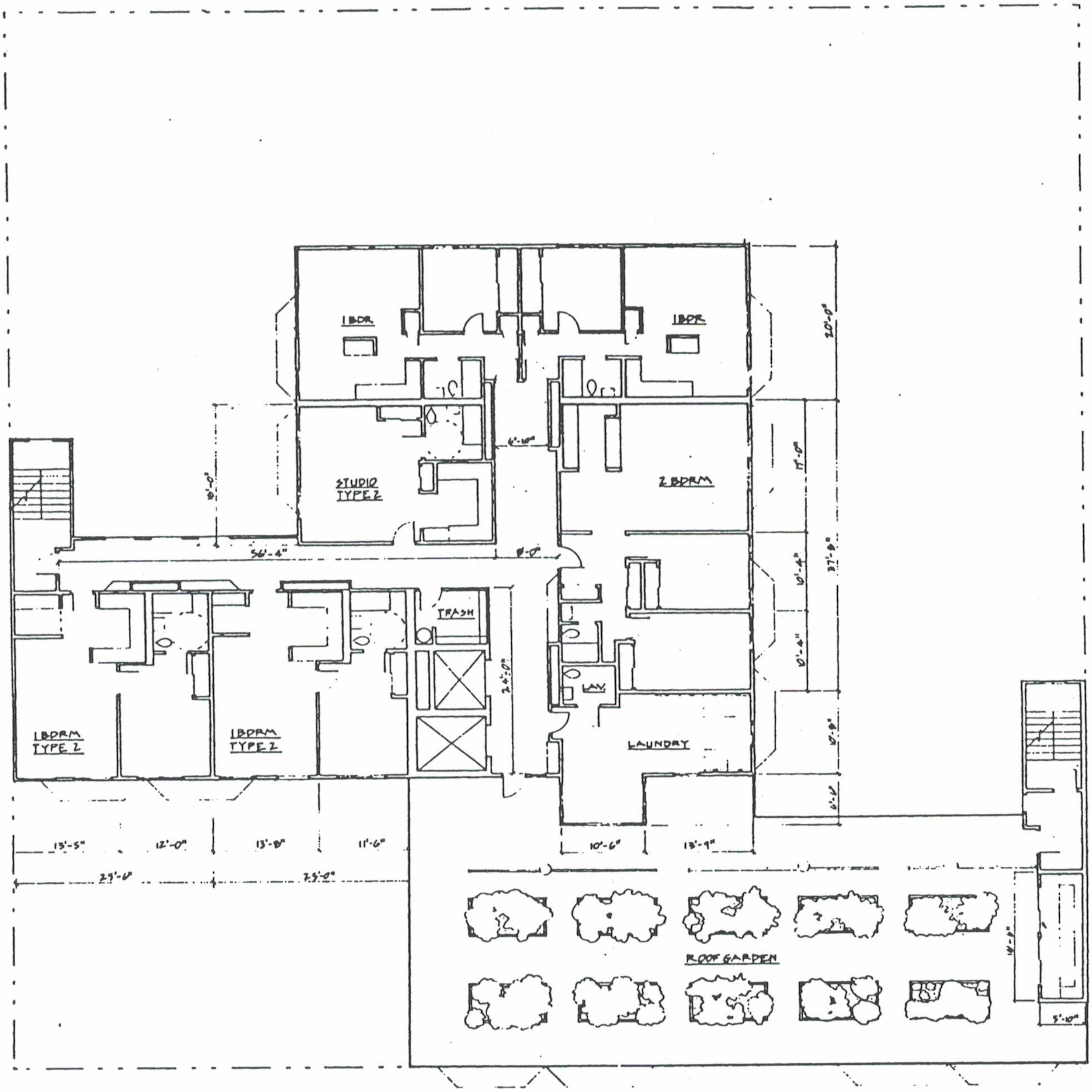


FIGURE 20
Ninth Floor Plan

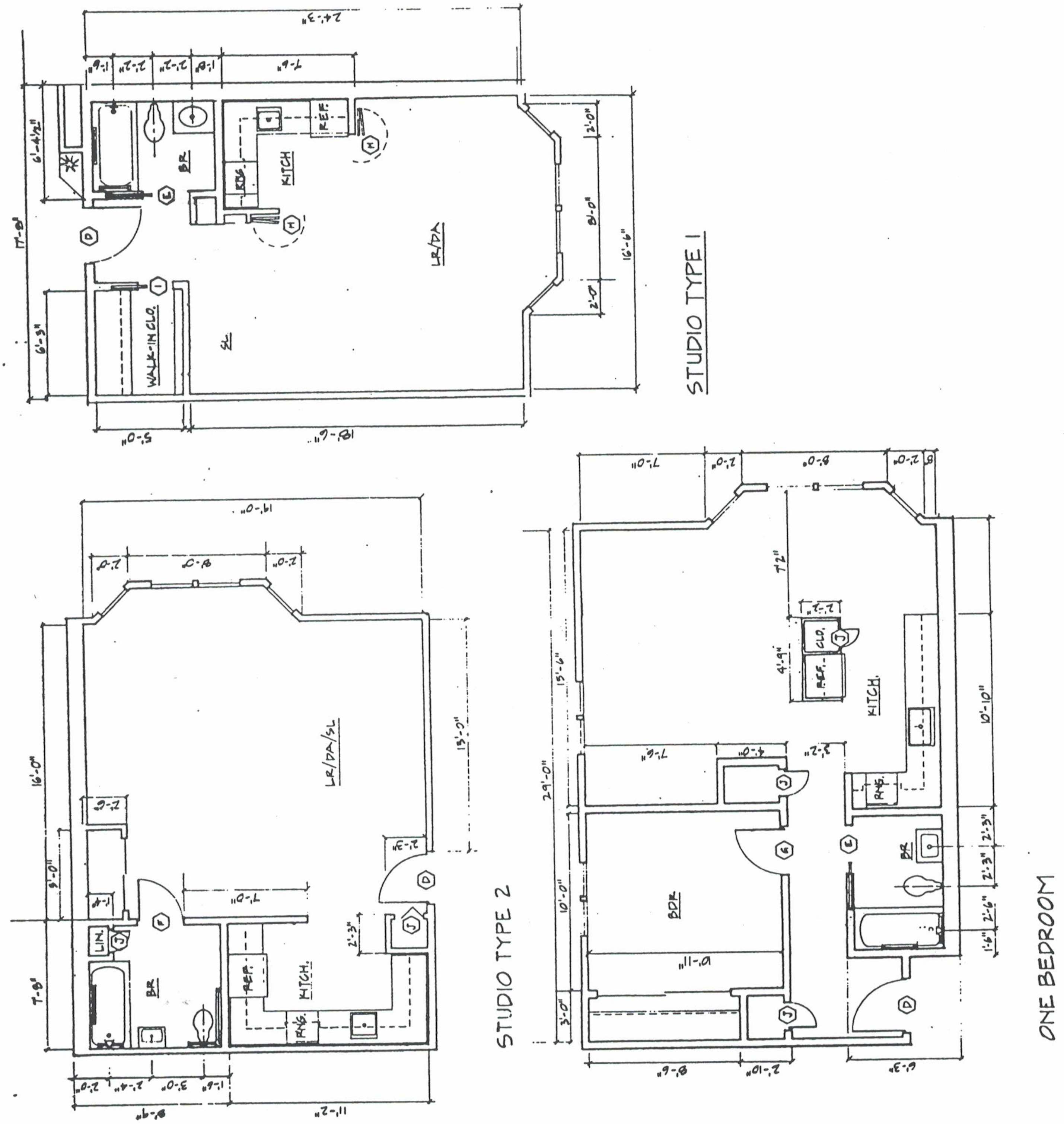


FIGURE 21
Typical Unit Plans

Transfer Slab Concept, and Other Structural Implications

Because, as has been mentioned, the semi-open office plan was not suitable for the cellular construction technique, the building was divided into two parts, divided by a thick slab with strategically located beams to carry vertical loads and lateral forces down into a column grid/shear wall system in the concrete portion of the building. Thus the office portion of the building forms a platform for the residential portion.

The structural plan for the concrete portion of the building was further complicated by the decision to optimize the design of the residential portion, imposing an irregular and difficult grid on the lower floors. In order to open the plan on the office floors, it was necessary to carry loads from the bearing walls above onto concrete beams, which stretch from the columns in the front of the building to columns or walls aligned under the outside wall of the corridor behind the apartments (see Section A-A, Figure 15). Because the corrugated steel bearing walls have the potential to act as beams themselves, the loads on these concrete beams would tend to concentrate at the ends, reducing their required depth.

Utilities Distribution

The prefabricated component system works most efficiently using the mechanical wall concept (see Fig. 3, above), which has the important benefit of allowing a natural reduction of the floor-to-floor dimension. The prefabricated mechanical wall works best with a vertical utility distribution, with a vertical utility chase for each column of mechanical walls (in this instance, serving 14 units). This means that, with the exception of minor electrical runs, there is no horizontal distribution of utilities at each floor level. Utilities are distributed horizontally in the ceiling below the platform slab. Obviously, this eliminates the necessity for false ceilings with ductwork and pipes. In this particular project, the desire to compress the building vertically was so great (due to a height limitation) that we were required to use the HUD minimum 7'-8" ceiling height. We were able to confine the slab thickness to 5 1/2" without resorting to an expensive post-tensioning system because the bearing wall system allowed slab spans to be short enough to use a composite metal deck/lightweight concrete system; and because there was no need to provide utilities space, we were able to have the required number of units without compromising the desire to have light and space amenities like the entry court, and a 30' minimum setback from the rear property line.

Effects of Standardization: Technological Poisoning?

Techniques like the one proposed in this report have had an enormous impact on our society. Although there is no doubt that people in general have embraced technology with the fervent grasp of efficient consumers, doubts about its ultimate worth remain unanswered. I will mention in passing the problems of pollution, social regimentation, and the disruption caused by inhumane factory work (exceeded only perhaps by the unemployment generated by automation), and concentrate on the effects which were created by the standardization necessary to make this building system workable as a mass-produced set of components.

By "standardization" I mean, not the sort of standardization which goes into making sure that one brand of bolts fits another brand of nuts, but the repetition of components necessary to reduce the number of kinds of parts which have to be produced. Probably the most rigid requirement in this regard was to maximize the use of the double kitchen/bathroom mechanical wall component (please refer to Figure 5). The use of this concept had two notable effects: one was that the kitchen was placed out in the room, rather than against the hallway as most effeciency units are designed. In my opinion, this configuration is a great benefit to the residents, who are mostly elderly, low-income women, whose kitchens have always been the centers of their homes. The position of the kitchen (see Figure 21) makes it not only potentially central to the social space of the apartment, it is also afforded a great deal more natural light than it would be against the hallway wall.

A second effect was to make a great many apartments the same, or mirror images of each other. Great pains were taken to make the system flexible enough in its industrial concept (the computerized steel assembly line is a key here) to allow the building as a whole to be asymmetrical and to respond to local conditions like sunlight. The circulation pattern on each floor is designed so that each turn is distinctive and memorable, rather than being uniform and thus confusing. Yet, it was impossible to extend that effort to make the individual apartments distinctive, without altogether destroying the possibility of mass production. In a way, this pattern can be compared to the form of a tree: each tree is morphologically similar, yet while the tree as a whole is distinct from all other trees, when we get to the level of the leaf, its parts are symmetrical, regular and more or less indistinguishable. This concept is a long way from the ideas of the early pro-industrial architectural movements of the twenties (e.g., the Futurists), which tried in blind adoration of industry to make their forms appear machine-like or machine-made even if crafted by hand. The notion that "the house is a machine for living" has been taken to absurd limits by some of the more alienated of our architect-gurus.

If there is any hope of humanizing industrial production, we must regard twentieth century technique as no different in essence from medieval technique, or from the techniques of nature. It is possible that the tailoring of the apartment components could be made individualized in minor ways by a highly sophisticated and thoughtful programming of the factory computers, so that there would be small distinguishing marks equivalent to the difference between one leaf and another on a tree.

In a way, the position I'm taking is an uncomfortable one because it is neither apologist nor positivist. That the proposed system will contribute to all of the global detriments mentioned at the beginning of this discussion on technology is not denied; what makes the question of its worth an open one, however, is the possibility that with more fervently applied health standards, a greater concern for humanizing jobs, and a genuine attempt to design not only goods but the system that produces them with a social conscience, the loss of traditional ways might be balanced by an increased ability to gracefully meet human needs for shelter, food and all the things which make for well-being. Can technique be blamed for all our ills? Or should we rather blame the imperfection of our ideals and consciences?

APPENDIX A

Load Calculations for Building

WIND LOAD CALCULATIONS

Calculations for wind load are based on UBC Section 2311, using Method 2 (areas projected in the horizontal and vertical plane). Since we are interested only in the upper seven storeys of the building, we will ignore the first two floors, and consider that the upper portion is resting on a platform. We will calculate the wind loads for winds from the south only.

PRESSURES

The formula for design wind pressures[1] is given as:

$$p = C_e \; C_q \; q_s \; I$$

Quantities I and q_s are constant for the building, with

$$I = 1 \text{ and } q_s = 13 \text{ psf (Table 23-F).}$$

So, we have,

$$p = 13 \text{ psf } (C_e \; C_q).$$

At this site we have Exposure B; so, for the portion of the building which is 24 to 40 feet high,

$$C_e = 0.8 \text{ (Table 23-G).}$$

For the portion 40 to 60 feet high, and 60 to 85 feet high,

$$C_e = 1.0 \text{ and } 1.1, \text{ respectively.}$$

From Table 23-H, we find that for the sides of the building,

$$C_q = 1.4,$$

and for the roof,

$$C_q = 0.7.$$

So, expressing pressures in the form of a table, we have,

---

[1] UBC, Section 2311(d).

TABLE A1: Wind Pressures by Zone

Element	Height[2]	Area	C_e	C_q	p
Walls	24 - 40 ft	2200 sf	0.8	1.4	**14.6 psf**
Walls	40 - 60 ft	2750 sf	1.0	1.4	**18.2 psf**
Walls (incl. proj. roof)	60 - 85 ft	3205 sf	1.1	1.4	**20.0 psf**
Roof	-	8943 sf	1.1	0.7	**10.0 psf**

WIND FORCES

F_h = (2200sf)(14.6psf) + (2750sf)(18.2psf) + (3205sf)(20.0psf)

= **146 k**.

F_v = (8943 sf)(10.0 psf)

= **89 k**.

OVERTURNING MOMENT DUE TO WIND

Overturning moment due to southerly winds is summarized in Figure 1 and Table A1:

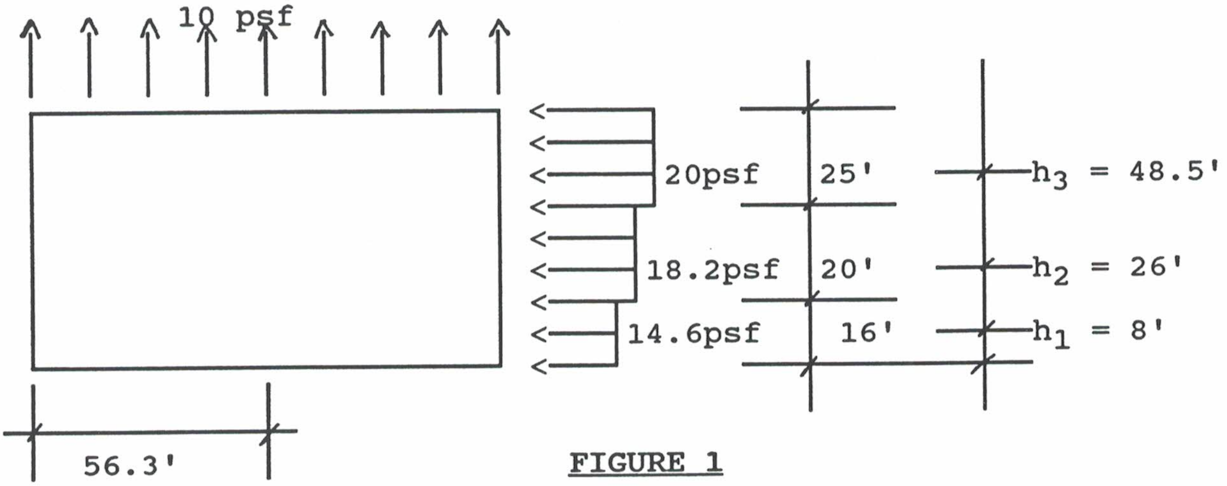


FIGURE 1

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[2] Cf. Table A4 below.

The location of the Neutral Axis of the roof is found by referring to Figure 2:

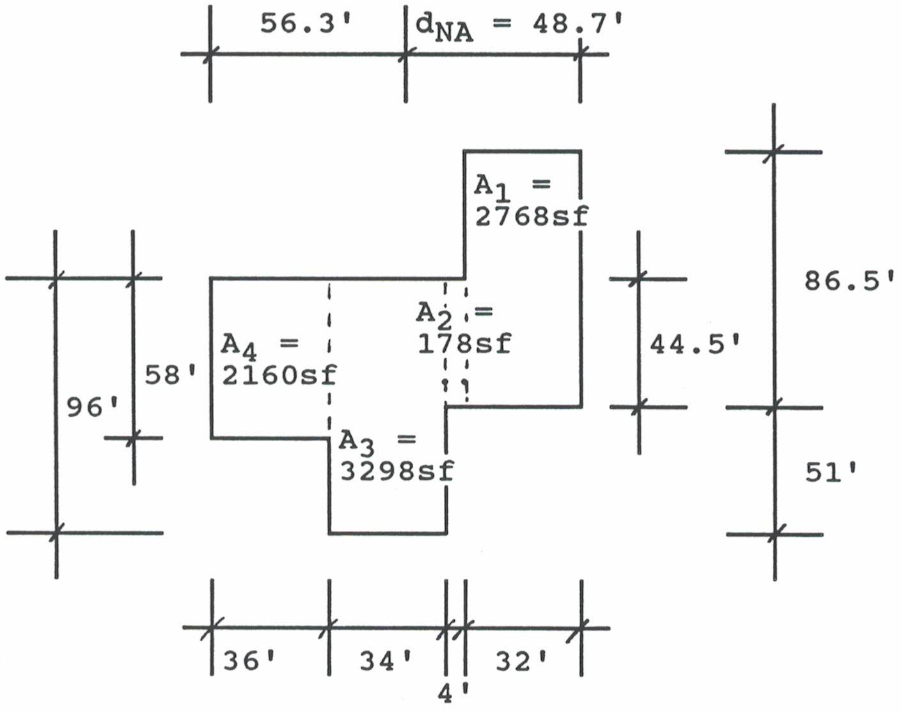

FIGURE 2

$$d_{NA} = \frac{A_1(16') + A_2(34.5') + A_3(51) + A_4(87.5')}{A_{tot}}$$

$$= \frac{[2768\ (16) + 178\ (34.5) + 3298\ (51) + 2160\ (87.5)]\ \text{ft}}{8404}$$

$$= \underline{48.7\ \text{ft}}.$$

TABLE A2: Moments Due to Wind

Element	Force	Moment Arm	Moment
Roof	80.9 k	56.3 ft	4555 ft k
Walls (60'-85')	64.1 k	48.5 ft	3109 ft k
Walls (40'-60')	50.0 k	26.0 ft	1300 ft k
Walls (24'-40')	32.1 k	8.0 ft	257 ft k
TOTAL			9221 ft k

EARTHQUAKE LOAD CALCULATIONS

ASSUMPTIONS

The building is nine stories, of which the bottom two stories are constructed as a concrete platform, and the top seven as the corrugated steel bearing wall cells. Since this paper is concerned with the characteristics of the steel bearing wall system, we will assume that the first two stories form a rigid base.

The building as designed has a complex plan, which does not lend itself to a straightforward earthquake analysis. Recognizing the limitations of this project, we will approximate forces by using formulas which would be applicable only if the building were a regular rectangular prism. In some instances numbers will be used which correspond to the actual plan of the building (e.g., we will use the actual base dimension of the building in calculating the period), but in other instances the model will be simplified by assuming we are dealing with a rectangular plan.

BASE SHEAR

From UBC Section 2312 (d), we are given a formula for base shear:

$$V = ZIKCSW,$$

where all values but W and C are a series of coefficients derived from tables. Thus,

$Z = 1$ (UBC Ch. 23, Fig. 1)

$I = 1$ (UBC Table 23-K)

$K = 1.33$ (value for a shear-wall building, UBC Table 23-I)

$S = 1.2$ (method B, UBC Section 2312 (d), deep stable sand).

The value for "C" is dependent upon the base of the building, which is 104 ft. in the NS direction and 137.5 ft. in the EW direction. So, based upon the formula given,

$$C = \frac{1}{15 \sqrt{T}} ,$$

where

$$T = \frac{.05\ h_n}{\sqrt{D}} ,$$

h_n is the distance from the third floor to the base of the roof (= 8.25 ft (7) = 57.75 ft.), and D is the length of the base in the direction considered in feet,

$$T_{NS} = \frac{.05\ (57.75)\ \text{sec}}{\sqrt{104}}$$

$$= 0.28 \text{ sec} ,$$

$$T_{EW} = \frac{.05\ (57.75)\ \text{sec}}{\sqrt{137.5}}$$

$$= 0.25 \text{ sec},$$

$$C_{NS} = \frac{1}{15 \sqrt{0.28}} = 0.13, \text{ and}$$

$$C_{EW} = \frac{1}{15 \sqrt{0.25}} = 0.13 .$$

But C need not exceed 0.12 (UBC Section 2312 (d)). So, for both axes,

$$C = 0.12$$

Then,

$$CS = 0.14$$

The value for "W" is calculated from the dead loads given in Table A3 below:

$$W = 6\ (372k + 116k + 34k) + (372k + 56k + 35k + 18k + 47k + 6k)$$

$$= \underline{3666\ kips}.$$

Finally, substituting all the values into the original equation,

$$V = (1)\ (1)\ (1.33)\ (0.14)\ (3666k)$$

$$= 0.19\ (3666k)$$

$$= \underline{683\ kips}.$$

FORCE DISTRIBUTION

UBC Section 2312 (e) prescribes a design distribution of forces at each floor level:

$$F_x = \frac{(V - F_i)\ w_x\ h_x}{\sum_{i=1}^{n} w_i\ h_i}$$

where

$$F_t = 0.07\ TV$$

$$= 0.07\ (.28)\ (683k)$$

$$= \underline{13.4\ kips}, \text{ and}$$

$$V - F_t = 633k - 13.4k = \underline{619.6\ kips}.$$

Force distribution is summarized in the following table:

TABLE A3: E.Q. Force Distribution

Level	w_x	h_x	w_xh_x (ft. kips)	F_x
roof	53k	57.75	3,061	19.7 + 13.4 = 33.1k
7	481k	59.5	28,620	184.2 kips
6	522k	41.25	21,533	138.6 kips
5	522k	33.0	17,226	111.0 kips
4	522k	24.75	12,920	83.1 kips
3	522k	16.5	8,613	55.4 kips
2	522k	8.25	4,306	27.7 kips
1	522k	0.0	0	0.0 kips
TOTAL	3666k		96,278	633.1 kips (okay)

OVERTURNING MOMENT DUE TO EARTHQUAKE

Figure 3 shows the building and lateral forces due to earthquake as a simple rectangular model:

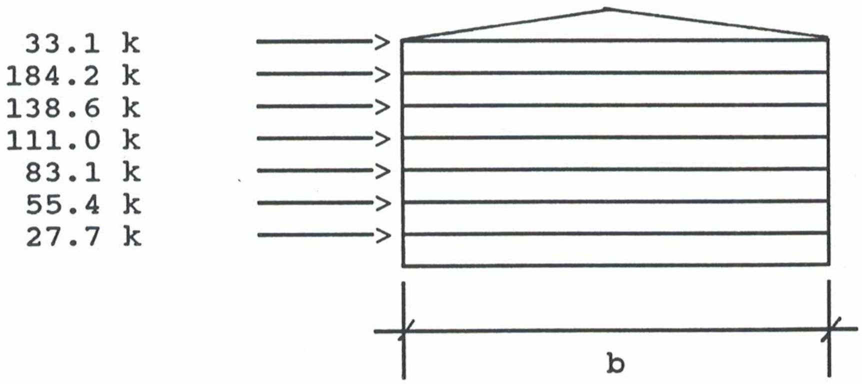

Figure 3

Total overturning moment is calculated in Table A4:

TABLE A4: Overturning Moment Due to E.Q.

Level	F_x	H_x	F_xH_x
Roof	33.1 k	57.75'	1912 ft k
7	184.2 k	49.5'	9118 ft k
6	138.6 k	41.25'	5717 ft k
5	111.0 k	33.0'	3663 ft k
4	83.1 k	24.75'	1057 ft k
3	55.4 k	16.5'	909 ft k
2	27.7 k	8.25'	229 ft k
TOTAL			29,605 ft k

RESISTANCE TO OVERTURNING

Resistance to overturning is calculated upon the dead load of the building. We need to find the location of the center of gravity of the typical floor, and accept that as the center of gravity of the building, neglecting the slight error due to the fact that the top floor is different from the others. For calculation of the center of gravity, refer to Figure 4:

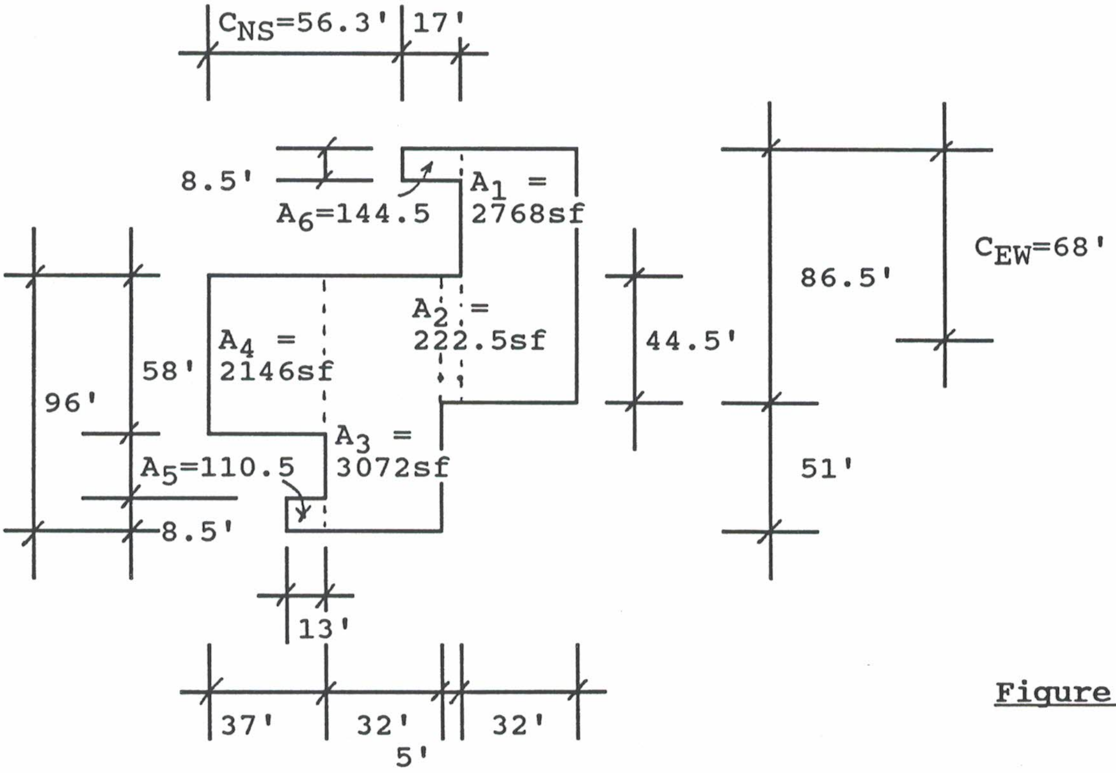

Figure 4

$$C_{NS} = \frac{A_1(16.5')+A_2(53')+A_3(72.5')+A_4(90')+A_5(30.5')+A_6(65.5)}{A_{tot}}$$

$$= 1/8463.5\ [2146(16.5) + 3072(53) + 222.5(72.5) + 2768(90)$$

$$+ 110.5(30.5) + 144.5(65.5)]\ \text{ft.}$$

$$= \underline{56.3\ \text{ft.}}$$

$$C_{EW} = A^{1}(70.5')+A_{2}(89.5')+A_{3}(64.25')+A_{4}(90')+A_{5}(133.25')+A_{6}(4.25)$$

$$= 1/8463.5\ [2146(70.50) + 3072(89.5) + 222.5(64.25)$$

$$+ 2768(43.25) + 110.5(113.25) + 144.5(4.25)]\ \text{ft}$$

$$= \underline{\text{68 ft.}}$$

We can now find the moment of resistance for overturning in the EW and NS axes:

$$M_{NS} = WC_{NS} = 3666\text{k}\ (56.3') = \underline{\text{206,396 ft kips}} \text{ and,}$$

$$M_{EW} = WC_{EW} = 3666\text{k}\ (68') = \underline{\text{249,288 ft kips}}.$$

It can be seen that the resistance to overturning far exceeds the overturning moment imposed by wind (9221 ft kips) and earthquake (23,605 ft kips) loads.

TABLE A5: Unit Loads

Component	Calculation	Weight
Bearing and Exterior Walls		
Steel	0.318pci [(.05")(93")(14.9"/ft) + (.25")(3")(24")] =	27.7pf
Insulation	15pcf[3] (.25')(7.75') =	29.0pf
Sheetrock[4]	2psf (2)(5/4)(7.75') =	38.7pf
Total		**95.4pf**
Curtain Walls	38.7pf + 16pf =	54.7pf
Floor Slabs		
Steel	(Wheeling Super-Bond 300)	4.1psf
Concrete	110pcf [(2.5/12)ft (1sf) + (3/12)ft (0.5sf)] =	36.7psf
Sheetrock (ceiling)	2psf (5/4) =	2.5psf
Total		**43.3psf**
Live Loads[5]		
Residential	R = .08 (.01)(415-150) = 0.21 LL = (1 - R)(40psf) =	31.6psf
Office, Computer, Assmby	(1 - R)(100psf)	varies

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[3] Estimated from density of "Mineral Fiber with resin binder" (ASHRAE 1985 Fundamentals Handbook, p. 23.7), and from the density of "Spraydon" insulation, 10.2 to 17.8 pcf (1984 Sweets Catalog, 7.14Am).

[4] Merritt, Frederick S.: Building Construction Handbook (McGraw-Hill, 1975), p 3-8.

[5] UBC, Table 23A and Sec. 2306

(TABLE A5: Unit Loads, cont.)

Component	Calculation	Weight
Roofing		
Framing[6]	4psf + 3psf/in (0.5in) =	5.5psf
Hot tar[4]		6.0psf
Cement shingles[4]		4.0psf

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[6] Merritt, op.cit., p. 3-8.

TABLE A6: Areas, Lengths and Weights Per Floor[7]

	Floors 3 - 8		Floor 9	
	Area/Length	Weight	Area/Length	Weight
Slab	8597 sf	372 k	8597 sf	372 k
Bearing Walls	1215 ft	116 k	583 ft	56 k
Curtain Walls (Steel stud)	630 ft	34 k	640 ft	35 k
Hot tar deck	-	-	3043 sf	18 k
Shingle roof	-	-	4921 sf	47 k
Hot tar roof	-	-	533 sf	6 k
Live Load	8597 sf	272 k	8597 sf	272 k

TABLE A7: Exterior Plane Areas

	Floors 1 and 2	Floors 3 - 9
South Walls	3300 sf	8155 sf
North Walls	2612 sf	8155 sf
East Walls	2995 sf	7220 sf
West Walls	2913 sf	7175 sf
Roof (projected horizontally)	7903 sf	8943 sf

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[7] Cf. Table A5, above.

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