PREFABRICATED CONCRETE MASONRY PANELS

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A Major Technical Report

in

The Faculty

of

Engineering

Presented in Partial Fulfillment of the Requirements
for the degree of Master of Engineering at
Concordia University
Montréal, Québec, Canada

July, 1979

CONCORDIA UNIVERSITY

ABSTRACT

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Prefabrication in the construction industry extends to concrete masonry wall panels built away from the building for speed, efficiency, and quality of construction. A remarkable development in this sector is the production of prefabricated wall panels using the Tomax Machine.

Prefabricated Tomax wall panels form the major subject matter of this report. The Tomax machine, its operation, production of Tomax panels and their advantages are described. Panel strength tests, panel connections and the application of the panels are discussed. This technical report also discusses the need for prefabricated concrete block panels and the properties of hollow concrete masonry with respect to fire resistance, noise control and energy conservation. It further covers the structural design of hollow concrete masonry walls.

ACKNOWLEDGEMENT

The writer wishes to express his gratitude and deep appreciation to Dr. Z. A. Zielinski, Professor of Civil Engineering, Concordia University, for his valuable guidance and corrections which helped in the presentation of this report.

Montreal, Quebec
July, 1979

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INTRODUCTION

1.1 NEED FOR PREFABRICATION

One of the main problems that we are facing today is the need for new lower-cost housing with a performance that is not below the desirable level in order to satisfy the needs of hundreds of millions of people around the world. This is becoming a major concern of all the nations and it is well noted in the activities of the United Nations. The provision of decent homes for every family of the world is an integral part of the human environment.

The shortage of housing, higher standard of living, shortage of skilled labour, high cost of construction materials, and weather conditions lead to the introduction of countless construction techniques ranging from conventional on site construction to completed prefabricated modules. This industrialization process has been proceeding at a rather rapid pace for the last twenty-five years or so. In Europe, industrialized building has achieved greater productivity — in terms of the value of building per man hour worked — and greater speed of erection than traditional building methods. In Canada, the on-site labour has been reduced by about 50%. This is mainly due to the following reasons:

1. It has been clearly demonstrated that mass production results in reduced costs.
2. There is a growing shortage of skilled tradesmen and as a consequence, greater reliance must be placed on machines and unskilled labour to handle the building process. This was the major reason put forward for Systems Building in Europe.
3. In-plant manufacturer of many of our building components has been shown to greatly reduce building time with consequent financial benefits.
4. The more work done in-plant the less serious will be the on-site problems with weather.
5. In-plant manufacture provides conditions for much better quality controls. Whether such better quality is actually achieved depends very much on the attitude of management, designers and supervisors.
6. Industrialization should make efficient use of our rapidly dwindling resources.
7. With better design and quality control, the achievement of durable and long-lasting structures is simplified.

Though labour costs, labour shortages and quality control are largely responsible for the growing trend away from on-site construction, prefabricated modular construction has been subjected to limitations. Experience with "Habitat 67" in Montreal, and "Operation Breakthrough" in the United States, has shown that concrete box-module systems have major difficulties in transportation, erection, and sustaining long-term success. For the above reasons, prefabrication systems using panels instead of modules appear to be preferable.

1.2 CONCRETE BLOCK PANELS

Major developments in prefabrication occurred in Europe. European systems have lots of experience to offer after costly experiments and mistakes in industrialized housing. Only a few economically viable systems survived out of all these efforts. The panelization system is the major contribution to the construction industry out of Europe. The panelized construction utilizing prefabricated panels represents a logical and practical compromise for many building systems due to the facts that:

1. Panels are easily fabricated. This applies to masonry, solid, cored and sandwich panels.
2. Panelization has relatively few system dictated constraints. Regardless of the layout of the structure, one can always almost successfully panelize it.
3. Panels are easy to handle and have no major problems in transportation and erection.
4. Panelization can lead to standardization and industrialization of building components to better building design and performance.

The use of concrete block for panels introduces the following advantages:

1. Concrete is the construction material that is available, inexpensive, and with desirable properties compared to other construction materials such as timber, steel and plastics.
2. Concrete block is the most efficient use of concrete.
3. Concrete block as building material introduces a number of specific qualities such as:

a. light weight
b. sound structural capabilities
c. very durable
d. fire resistance
e. sound isolation
f. easily insulated
g. hollow, thus economical and insulation properties
h. a textured surface, where required
i. esthetics
j. maintenance free
k. economy

About 10 years ago the US Housing and Urban Development (HUD) turned down the conrete masonry industry's proposals for "Operation Breakthrough" funding. Concrete masonry was considered as traditional type of construction, a type the HUD envisioned as to be replaced by the more esoteric systems construction and newer materials. But today, it appears that many of the new systems and materials that were accepted by HUD have proven to be more costly then conventional.

One of the main problems facing many building systems is market acceptability. Most of the systems can be classified as "closed" with limited freedom in planning. Prefabricated concrete masonry wall panels overcome these objections by permitting more design flexibility and providing a conventional type of wall construction that is familiar and readily accepted by all code authorities.

CHAPTER II

PROPERTIES OF CONCRETE MASONRY WALLS

2.1 FIRE RESISTANCE

Fire safety is a major consideration in building codes because fire is one of the major hazards to life and property in buildings. The governing philosophy behind various code requirements with respect to fire is: (1) the safety of occupants; (2) the safety of firemen; (3) and the reduction of property damage. The accomplishment of this philosophy depends mainly on the components of buildings and their resistance as barriers to firespread. The fire resistance of the enclosing elements of a compartment such as walls, partitions and floors are, therefore, major obstacles to the spread to a fully developed fire.

Concrete masonry walls have excellent fire resistant qualities. Because of these qualities concrete masonry walls are often used as an essential and key element in fire protection system of modern apartment buildings. They are also used to enclose steel columns and other less resistant materials in construction jobs.

2.1.1 Evaluating Fire Resistance

The fire resistance ratings of concrete masonry walls are based on fire tests made at Underwriters' Laboratories, Inc., the National Bureau of Standards, and other recognized fire testing laboratories. The procedure of the tests are described in ASTM E119, "Standard Method of Fire Tests of Building Construction and Materials." Standard test consists of exposing a large wall panel to a fire of controlled intensity for a time equal or greater than its rated fire resistance time. Immediately after firing, the hot face of the wall is subjected to fire hose stream. During the test, load bearing walls carry a load of 80 psi. The criteria applied during a standard fire test of a wall or partition is that (1) the wall must withstand the fire, (2) must not allow passage of flames or heated gases, or passage of water from the hose stream, and 3) heat transmission through the wall must be limited to less than 250^{o}F gain in temperature. Any one of the three criteria is decisive should it be violated first [2].

The fire resistance rating for concrete masonry walls is almost always determined by temperature rise on the unexposed side of the wall. Fire endurance can be calculated as a function of the aggregate type used in the block unit and the equivalent solid thickness of the wall. Equivalent thickness of hollow units is calculated from actual thickness and the percentage of solid materials. Both needed items of information are determined by ASTM C140 "Methods of Testing Concrete Masonry Units".

The concept of using "Equivalent Thickness" for determining fire resistance ratings has been adopted by the National Code of Canadian Supplement No. 2, Table 2.1, shows the equivalent thickness and fire endurance of concrete masonry walls [3].

Table 2.1 — Minimum Equivalent Thickness of Concrete Masonry Walls

Type of Concrete	1/2 hr.	3/4 hr.	1 hr.	1-1/2 hr.	2 hr.	3 hr.	4 hr.
	Fire-Resistance Rating						
S* or N**	1.7	2.3	2.9	3.7	4.4	5.6	6.6
$L_1$20S†	1.6	2.1	2.6	3.4	4.0	5.1	6.0
L_1††	1.6	2.1	2.5	3.2	3.8	4.8	5.6
$L_2$20S†	1.6	2.1	2.5	3.2	3.7	4.6	5.3
L_2††	1.6	2.1	2.5	3.1	3.6	4.4	5.1

Notes:

*Type S concrete is the type in which the coarse aggregate is granite, quartzite, siliceous gravel or other dense materials containing at least 30% quartz, chert or flint.

**Type N concrete is the type in which the coarse aggregate is cinders, broken brick, blast furnace slag, limestone, calcareous gravel, trap rock, sandstone or similar dense material containing not more than 30% of quartz, chert or flint.

†Type $L_1$20S and Type $L_2$20S concretes are the types in which the fine portion of the aggregate is sand and lightweight aggregate in which the sand does not exceed 20% of the total volume of all aggregates in the concrete.

††Type L_1 concrete is the type in which all the aggregate is expanded shale.

†††Type L_2 concrete is the type in which all the aggregate is expanded slag, expanded clay or pumice.

The ratings listed in the above table are almost the same as those contained in building codes in the United States.

2.1.2 Walls Covered with Plaster and Wallboard

The fire resistance value shown in Table 2.1 are for bare concrete masonry salls without plaster or wallboard finishes. The National building code of Canada contains detailed provisions for the contributions of plaster or wallboard finishes. Plaster or wallboard finishes do add to the fire resistance of the concrete masonry wall. The increase in fire endurance depend upon the type of finish, the type of block it is on, and whether the finish is on the exposed or non-exposed side of the wall. Plaster applied directly to the masonry surface behaves in a fire much the same as the masonry unit itself. Its contribution to increased fire resistance can be determined by adding the plaster thickness to the equivalent solid thickness of the masonry.

If plasters or wall finishes are applied to the unexposed side of the wall, multiplying or reduction factors are used to calculate the added thickness, the increase or decrease in added thickness depending upon the relative heat resistance of the concrete masonry and the finish material. Table 2.2 shows typical multiplying factors [3]. These factors can be used to correct the thickness of plaster applied directly to either or both sides of the wall, and for wallboard and plaster on lath on the unexposed side. The corrected thickness is then added to the equivalent thickness of the concrete masonry to determine the fire resistance.

With wallboards or plaster applied on the fire exposed side are treated in a different way. Their contribution to additional fire endurance is limited by the period of time they stay on the wall. Methods of fastening wallboard or lath to masonry walls are specified by the NBCC. Tables 2.3 and 2.4 show fire endurance time assigned for wallboard membranes and plaster membranes [3].

Table 2.2 — Multiplying Factors For Various Masonry Construction.

TYPE OF SURFACE PROTECTION	Type of Masonry: N or S	L 20S	L or L 20S	L
Portland Cement-Sahd Plaster	1	3/4	3/4	1/2
Gypsum-Sand Plaster or Gypsum Wallboard	1¼	1	1	1
Gypsum-Vermiculite or Gypsum-Perlite Plaster	1-3/4	1½	1¼	1¼

Table 2.3 — Time Assigned to Wallboard Membranes.

DESCRIPTION OF FINISH	Time min.
1/2 in. fiberboard	5
3/8 in. Douglas-fir plywood phenolic-bonded.	5
1/2 in. Douglas fir plywood phenolic-bonded.	10
5/8 in. Douglas fir plywood phenolic-bonded.	15
3/8 in. gypsum wallboard	10
1/2 in. gypsum wallboard	15
5/8 in. gypsum wallboard	30
Double 3/8 in. gypsum wallboard.	25
1/2 + 3/8 in. gypsum wallboard	35
Double 1/2 in. gypsum wallboard.	40
Double 1/2 in. gypsum wallboard.	50
3/16 in. asbestos-cement + 3/8 in. gypsum wallboard. . . .	40
3/16 in. asbestos-cement + 1/2 in. gypsum wallboard. . . .	50
Composite 1/8 in. asbestos-cement, 7/16 in. fiberboard. ..	20

Table 2.4 — Time Assigned to Plaster Membranes.

TYPE OF LATH	Plaster thicknesses, in.	Portland cement-sand or Portland cement-lime-sand	Portland cement-sand-asbestos fiber (3-lb./bag cement)	Gypsum-sand	Gypsum Wood Fiber	Gypsum & Perlite or Gypsum Vermiculite
Wood lath	1/2	5	10	20	20	-
1/2 in. fiberboard	1/2	-	-	20	20	-
3/8 in. gypsum lath	1/2	-	-	35	35	55
3/8 in. gypsum lath	5/8	-	-	40	40	65
3/8 in. gypsum lath	3/4	-	-	50	50	80*
Metal lath	3/4	20	35	50	50	80*
Metal lath	7/8	25	40	60	65	80*
Metal lath	1	30	50	80	80	80*

*The values shown for these membranes have been limited to 80 min. because the fire-resistance rating derived from this table must not exceed 1½ hr.

2.1.3 Effect of Filling Core Spaces

Standard fire tests have shown that filling the hollow core spaces in the concrete block, with insulation, grout, or with dry granular materials will reduce the rate of heat flow through the wall and results in an increase in fire endurance. For example, filling cores with perlite, vermiculite loose-fill insulation will increase the fire resistance of a 2-hr. wall to at least 4 hours. The same is true if cores are filled with expanded shale or slag. In fire tests conducted by the Portland Cement Association in 1934, it was indicated, Fig. 2.1, that a fairly wide range of granular material may be used for filling core spaces to increase fire resistance [4]. Filling the cores will alsso have the benefit of increasing the resistance of heat flow through the wall.

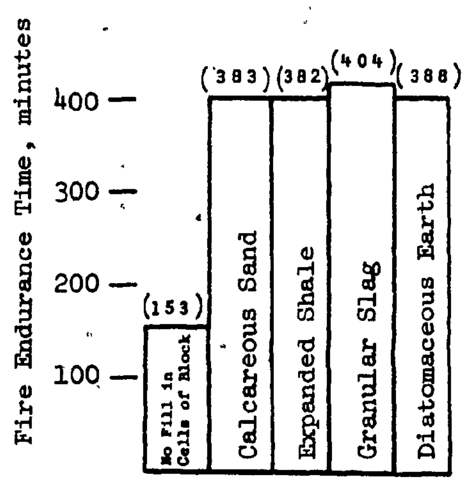


Fig. 2.1 — PCA Research in 1934 indicated that a fairly wide range of granular materials in the cells of the concrete block will greatly increase the fire endurance of the wall.

In reinforced hollow unit construction, where cells are filled with grout, the grout has the effect of adding to the equivalent thickness resulting in a corresponding increase in fire resistance. An 8-inch thick fully grouted wall, for instance, would have equivalent thickness of 7.6 inches which is the same as if the wall was composed of 100% solid units.

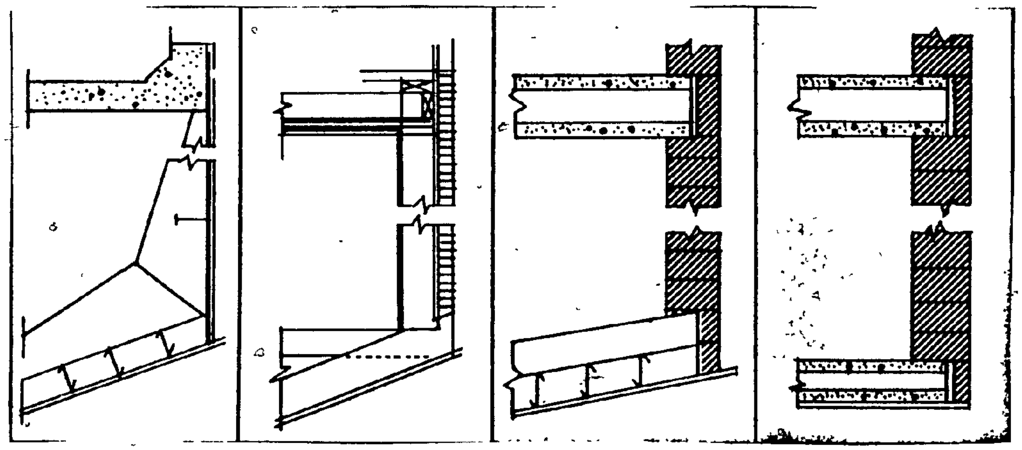
Both American and Canadian building codes allow filling core spaces in hollow masonry units; however, the fire enduracne time may not exceed that of a wall of solid units of the same concrete type.

2.1.4 Building Insurance Benefits

Insurance costs against fire loss for both building and content are determined mainly by the type of construction material used. Insurance rates are based upon actual experience, the actual risk that is involved. Concrete masonry buildings are considered superior to the non-combustible structures and the rates are lower for fire and extended coverage. Insurance costs continue for the life of the building and lower rates can produce significant savings. Table 2.5 lists average insurance rates per $100.00 for several construction types and occupancy classification [5]. The figures in the table were obtained by averaging values from information which include several protection classes in more than 50 cities. These figures are not exact and are listed merely to indicate relative differences between rates for various types of construction.

Insurance savings are not only to the owner, there is also savings to the tenant. Tenants in concrete masonry buildings who purchase insurance on their personal belongings, pay less annual insurance premium than those who live in other buildings. In addition to that there is a psychologic value. Concrete masonry walls convey a sense of security, privacy and permanence.

Table 2.5 — Typical Fire Insurance Rates for Various Building Types and Occupancies.*



Occupancy \ Construction	Concrete Masonry with concrete floors & roof	Concrete Masonry concrete floors wood roof	Wood Frame brick veneer	Pre-engineered metal building
Apartment	$ 0.149	$ 0.548	$ 1.097	-
Office	0.080	0.247	0.762	-
Storage Warehouse	0.116	0.497	1.399	$ 0.928

*Annual cost per $100.00 for Fire and Extended Coverage - 80% coinsurance - contents not included; figures represent average from more than 50 cities covering several Protection Classes.

2.2 Noise Control

Noise introduces one of the major threats to the quality of our lives. More and more noise is produced with each advancement in technology. In the home, interior noise comes mainly from the noise-making appliances such as television, radio, air-conditioning, vacuum cleaners, washing machines, and other devices. In the office, interior noice is caused by computers, typewriters and other noise-making equipment. Exterior noise may come from street traffic, jet aircrafts, high-speed trains, and construction equipment. People are becoming very conscious about noise and the need of quietness is becoming acute. Studies and surveys of occupants' desires show conclusively that people want residences which are soundproof. Since there is no hope that noise will vanish by a technical breakthrough, simply because there is no such a thing as a noiseless machine, it is one of the functions of a building to reduce all ordinary noise to a tolerable level. Two techniques are commonly used to reduce noise: sound absorption and limiting sound transmission.

2.2.1 Sound Absorption

Sound absorption is a method of reducing noise by diminishing the sound level within a room or a service area in a building by employing material which absorbs the sound instead of reflecting it back. Sound is absorbed by any surface that dissipates sound energy by converting it to heat.

The sound absorption coefficient is an indication of the sound absorbing efficiency of a surface. If the surfaces of a room were capable of absorbing all sound generated within a room, they would have a sound absorption coefficient of 1. If 45 percent of it was absorbed, the coefficient would be 0.45. A commonly used measure of sound absorption is the noise reduction coefficient (NRC). The NRC is determined by averaging the values of the sound absorption coefficients (SAC) at sound frequencies of 250, 500, 1000, and 2000 cycles per second (cps). The SAC is the amount of sound energy absorbed compared to perfectly absorptive surface such as an open window. Typical noise reduction coefficients for concrete blocks and some other materials are given in Table 2.6 [6].

Concrete blocks have NRC values between 0.26 and 0.50 while glass, plaster and smooth surfaces have NRC values less than 0.05. Painting concrete masonry decreases its NRC value.

2.2.2 Sound Transmission

The reduction of noise can be achieved by preventing sound waves from being transmitted from one space to another through the use of sound insulating material. The effectiveness of a wall as a means of sound isolation or noise insulation, is measured by a two-room test method in which sound is generated, measured in a specially designed room, passed through the test wall and measured in the adjacent test room. The measurements are made at sixteen different frequencies over a range of 125 to 4000 cycles per second (cps). The difference in intensity of the generated sound and the received sound indicates the transmission loss of the wall. The test method ASTM #90-70, provides a means of rating sound transmission by a single number called sound transmission class (STC). The higher the STC of a wall, the better its performance as a noise barrier. Recent sound transmission loss test data for various types of concrete masonry walls ranging from 4" to 12", and of various surface treatment are given in Tables 2.7 through 2.11 [7].

Table 2.6 — Approximate Noise Reduction Coefficients.

Material	Surface Texture or Finish	Approximate NRC
Brick, unpainted	All	0.05
Concrete floor, bare	All	.02
Wood floor	All	.03
Glass	All	.02
Plaster	Rough	.05
Plaster	Smooth	.04
Wood panel	All	.06
Acoustical tile	All	.55
Carpet on concrete	Heavy	.45
Lightweight aggregate block, unpainted	Coarse	.50
	Medium	.45
	Fine	.40
Normal weight aggregate block, unpainted	Coarse	.28
	Medium	.27
	Fine	.26

REDUCE BLOCK VALUES BY FOLLOWING AMOUNTS WHEN PAINTED:

Paint	Application Method	One Coat	Two Coats
All	Spray	10%	20%
Oil	Brush	20%	55%
Latex	Brush	30%	55%
Cement	Brush	60%	90%

Table 2.7 — STC Tests on 4-Inch Thick Concrete Masonry Walls.

Wall Description	Wall Weight lb/sf	STC
NO SURFACE TREATMENT		
hollow, lt. wt.	18	40
hollow, lt. wt.	22	29
hollow, norm. wt.	27	45
hollow, norm. wt.	33	45
hollow, norm. wt.	39	46
solid, norm. wt.	40	46
SURFACE SEALED WITH PAINT:		
both sides, hollow, lt. wt.	26	41
both sides, hollow, lt. wt.	22	43
both sides, hollow. lt. wt. hollow cells filled with sand	34	43
both sides, hollow, norm. wt.	32	44
both sides, hollow, norm. wt.	29	44
one side, solid norm. wt.	40	47
SURFACE SEALED WITH PLASTER:		
1/2", both sides, hollow, lt. wt.	35	44
1/2", both sides, hollow, lt. wt.	30	48
1/2", both sides, hollow, lt. wt.	32	49
1/2", both sides, hollow, lt. wt.	33	45
5/8", both sides, hollow, lt. wt.	42	50
5/8", both sides, hollow, lt. wt.	37	43
5/8", both sides, hollow, norm. wt.	43	48
5/8", both sides, hollow, norm. wt.	50	51
SURFACE SEALED WITH GYP. BOARD:		
1/2", both sides, hollow, lt. wt.	25	45
1/2", both sides, hollow, lt. wt.	27	40
1/2", both sides, hollow, lt. wt.	26	47
1/2", both sides, hollow, norm. wt.	32	48
1/2", + 1" blanket insul. & block sealer, one side, solid, norm. wt.	43	51
1/2", both sides + 1" insul. & block sealer, one side, solid, norm. wt.	44	52
1/2", both sides, block sealer, one side, solid, norm. wt.	44	44
1/2", one side, block sealer, one side, solid, norm. wt.	42	46
1/2", both sides, block sealer + 1/4" perlite board, one side, solid, norm. wt.	45	47
SURFACE BONDED WALLS:		
1/16", both sides, hollow, lt. wt. T&G	28	43

Table 2.8 — STC Tests on 6-Inch Thick Concrete Masonry Walls.

Wall Description	Wall Weight lb/sf	STC
NO SURFACE TREATMENT:		
hollow, lt. wt.	21	44
hollow, norm. wt.	43*	45
SURFACE SEALED WITH PAINT:		
both sides, hollow, lt. wt.	28	46
one side,hollow, lt. wt. (Soundblox)	35	49
one side, hollow, lt. wt. (Soundblox)	38	47
one side, hollow, lt. wt.	32	43
both sides, hollow, norm. wt.	39	48
SURFACE SEALED WITH PLASTER:		
1/2", both sides, hollow, lt. wt.	31	46
both sides, solid, norm. wt.	54	52
SURFACE SEALED WITH GYP BOARD:		
1/2", one side, hollow, norm. wt.	45*	45
1/2", one side, hollow, norm. wt.	45*	46
1/2", one side, hollow, norm. wt.	45*	49
1/2", both sides, hollow, norm. wt.	47*	44
1/2", both sides, hollow, norm. wt.	47*	45
1/2", both sides, hollow, norm. wt.	47*	47
5/8", both sides, hollow, lt. wt.	35	49
1/2", one side - hollow, lt. wt., paint, one side	27	53
1/2" + 1" insul., one side, hollow,	45*	50
norm. wt.	45*	51
5/8", one side - hollow, lt. wt. 1/2", one side	38	47
1/2", both sides - hollow, norm. wt. 1" insul., one side	48*	49
1/2" + 1" insul., both sides, hollow, norm. wt.	48*	46
1/2" + 1" insul., both sides, hollow, norm. wt.	48*	48
SURFACE BONDED WALLS:		
1/8", both sides, hollow, lt. wt.	30	43

*Estimated weight.

Table 2.9 — STC Tests on 8-Inch Single Wythe Concrete Masonry Walls

Wall Description	Wall Weight lb/sf	STC
NO SURFACE TREATMENT:		
hollow, lt. wt.	30	45
hollow, lt. wt.	39	49
hollow, lt. wt.	43	49
hollow, lt. wt., cells filled w/insul.	40	51
hollow, norm. wt.	53	52
hollow, norm. wt.	55	46
SURFACE SEALED WITH PAINT:		
both sides, hollow, lt. wt.	30	46
both sides, hollow, lt. wt.	34	48
both sides, hollow, lt. wt.	35	44
both sides, hollow, lt. wt.	32	46
both sides, hollow, norm. wt.	55	46
SURFACE SEALED WITH PLASTER:		
one side, hollow, lt. wt.	38	52
both sides, solid, norm. wt.	67	56
SURFACE SEALED WITH GYP BOARD:		
1/2", both sides, hollow, norm. wt.	55	49
1/2", one side - hollow, norm. wt., paint, one side	43	50
1/2", one side, hollow, lt. wt.	40	56
1/2", one side - hollow, norm. wt., paint, one side	55	48
1/2" + 1" insul, both sides, hollow, norm. wt.	60	46
1/2", both sides - hollow, norm. wt., 1" insul., one side	60	49
SURFACE BONDED WALLS:		
1/8", both sides, hollow, lt. wt.	34	42
1/8", both sides, hollow, norm. wt.	47	47
1/8", both sides, hollow, norm. wt.	49	48
1/8", both sides, hollow, norm. wt., cells filled with insulation	48	48
1/8", both sides, hollow, norm. wt., cells filled with concrete grout	92	56
REINFORCED AND GROUTED WALLS:		
No surface treatment	73	48
Surface sealed with:		
paint, both sides	73	55
1/2" plaster, both sides	79	56
1/2" gyp. board, both sides	77	60

Table 2.10 — STC Tests on 10-inch Thick Single Wythe Concrete Masonry Walls

Wall Description	Wall Weight lb/sf	STC
NO SURFACE TREATMENT:		
hollow, lt. wt.	47	44
hollow, norm. wt.	57	47
SURFACE SEALED WITH PAINT:		
both sides, hollow, lt. wt.	47	47
both sides, hollow, norm. wt.	57	49
SURFACE SEALED WITH PLASTER:		
both sides, hollow, lt. wt.	49	55
both sides, solid, lt. wt.	81	58
SURFACE SEALED WITH GYP. BOARD:		
1/2", both sides, hollow, lt. wt.	51	50

Table 2.11 — STC Tests on 12-Inch Thick Single Wythe Concrete Masonry Walls

Wall Description	Wall Weight lb/sf	STC
NO SURFACE TREATMENT:		
hollow, lt. wt.	53	39
hollow, norm. wt.	69	49
solid, norm. wt.	121	55
SURFACE SEALED WITH PAINT:		
one side, hollow, lt. wt.	53	51
both sides, hollow, lt. wt.	53	50
one side, hollow, norm. wt.	69	50
SURFACE SEALED WITH PLASTER:		
5/8", both sides, hollow, lt. wt.	56	50
5/8", one side — hollow lt. wt., paint, one side	56	50
5/8", both sides, hollow, lt. wt.	59	50
1/2", one side — hollow, norm. wt. paint, one side	72	52
1/2", both side, hollow norm. wt.	75	49
SURFACE SEALED WITH GYP BOARD:		
5/8", one side, solid, norm. wt.	124	58
1/2", one side — hollow, lt. wt. paint, both sides	55	49
5/8" + 1 1/2" insul. one side, solid, norm. wt.	124	56
1/2" one side — hollow, lt. wt., paint, one side	55	57
1/2" P 1" insul, one side — hollow, lt. wt., paint, one side	55	50
SURFACE BONDED WALLS:		
1/8", both sides, hollow, lt. wt.	67	51

The values in the STC tables indicate that concrete masonry walls fall in the range STC = 40 to STC = 60, which is approximately proportional to the logarithm of the weight per square foot of the wall. Also it may be observed that the treatment of the wall surface by using paint, plaster or sypsum board increases the sound insulation benefit.

The values of sound transmission loss through a wall are meaningless if a wall is made ineffective through poor or improper construction. Sound leakage will occur through any opening in a wall. As an example an improperly fitted corridor door provides much sound leakage. An opening of only 1/16 inch around a typical door produces a total opening of about 12 square inches. This illustrates the importance of achieving good, tight construction where openings occur, and well compacted mortar joints in the masonry wall itself [7].

2.2.3 Standards of Sound Transmission

Structural adequacy of walls does not necessarily satisfy the sound isolation requirements. Minimum acceptable standards have been established by building codes and official organizations. These requirements depend on background noise. The higher the level of background noise, the more transmitted noise can be tolerated. The current guides from the Department of Housing and Urban Transportation are given in Table 2.12 [6]. As it is shown in the Table, the STC values range from a low of 40 to a high of 55. The partisions which the table shows to require the highest STC level are those next to public space and service areas.

Table 2.12 — Sound Transmission Class Limitations.

Location of Partition	Low Background Noise: Bedroom adjacent to partition	Low Background Noise: Other rooms adjacent to partition	High Background Noise: Bedroom adjacent to partition	High Background Noise: Other Rooms adjacent to partition
Living Unit to Living Unit	50	45	45	40
Living Unit to Corridor	45	40	40	40
Living Unit to Public Space (Average Noise)	50	50	45	45
Living Unit to Public Space & Service Areas (High Noise)	55	55	50	50
Bedrooms to other rooms within same living unit	45	NA	40	NA

NA - not applicable

The Canadian Building codes adopt the American test methods for sound transmission class ratings and for minimum acceptable standards. A minimum acceptable standard between dwelling units in the same building as well as between dwelling units and space are not to be less than 45 STC [8].

In the previous section it was shown that the STC values of concrete masonry walls fall in the range STC = 40 to STC = 60, depending on the weight per square foot and surface treatment. The type of masonry and its thickness should be chosen to meet the STC requirements for the various kinds of walls and partitions. Concrete masonry walls have good reliability in acting as effective noise barrier. They are not sensitive to workmanship as some other specially designed sound barrier partitions which end up having low STC value because of improper installation.

2.3 ENERGY CONSERVATION

The need for conservation and efficient utilization of energy is a vital necessity. We can severely reduce the demand for energy through conservation and improved efficiency measures. It is known that almost 25% of the primary energy used in Canada is consumed for space heating. The conservation for energy in buildings depends as much upon operation as it does upon design. In the recent past, the use of energy in buildings was not a great concern of designers. Energy was abundant and cheap. A great number of the constructed buildings have low retention capabilities. This energy waste in our buildings can be stopped if the buildings we are building now are designed so that they are capable of being operated in an energy efficient way.

Measures for energy conservation have been developed in Canada. The Standing Committee on Energy Conservation issued a document, "Measures for Energy Conservation in New Buildings", in which the following is stated: [9]

1. Upon the application for a permit, the owner shall submit all plans, specifications and calculations to show in sufficient detail all relevant data and features of the building.
2. Calculations for the design of heating and cooking systems, and the thermal resistance of building assemblies shall be made in conformance with great engineering practice.

The committee considers the procedures described in the American Society of Heating, Refrigerating and Air-Conditioning Engineers, Inc. (ASHRAE) Standard 90-75, Energy Conservation in New Building Design, the

Digest of Heating, Refrigerating and Air Conditioning (HRA) of Canada, and the Hydronics Institute (HI) manuals as good engineering practice.

Concrete masonry buildings have the tendency to remain warmer in winter and cooler in summer. This is because of significant heat capacity available in the relatively thick and dense masonry walls. The mass of the wall absorbs heat and delays its flow either in or out. Also, air spaces in concrete masonry units act as insulators by resisting the transfer of heat by conductance. The flow of heat through a concrete masonry wall, as well as other kind of walls and building components, depend mainly on the U-factor, the overall heat transmission coefficient; the amount of heat, expressed in BTU (British Thermal Unit), transmitted in one hour through one square foot of a building section (wall, floor or ceiling) for each degree F. of temperature difference between air on the warm side and air on the cold side of the building section.

2.3.1 Estimating U-Factors

The value of U-factor includes the effect of air films, materials of construction, and air spaces. The ASHRAE Guide published by the American Society of Heating, Refrigerating and Air-Conditioning Engineers is the most common source for the methods used in computing the values of the U-factor for various types of construction. Tables 2.13 through 2,15 give estimated U-values for concrete masonry walls as affected by various details of construction [10]. These estimated values are not as accurate as those that would be obtained by a laboratory test. It is recommended that where laboratory test values are available they should be used.

Table 2.13 — CONSTRUCTION: 6" Hollow Concrete Block.

Density of concrete used in blocks, lbs/cu ft	60	80	100	120	140
Approximate weight of masonry, lbs/sq ft	20	26	33	40	46
WALL DETAILS	"U" VALUE				
(1) No insulation	.32	.37	.42	.47	.59
(2) No insulation, 1/2" gypsum board on furring strips	.22	.24	.26	.28	.32
(3) No insulation, 1/2" foil backed gypsum board on furring strips	.15	.16	.17	.18	.19
(4) Loose-fill insulation in cores	.16	.18	.22	.26	.41
(5) Loose-fill insulation in cores, 1/2" gypsum board on furring strips	.13	.15	.17	.19	.26
(6) Loose-fill insulation in cores, 1/2" foil backed gypsum board on furring strips	.10	.11	.12	.14	.17
(7) 1" rigid glass fiber insulation & 1/2" gypsum board applied direct to wall surface	.13	.14	.15	.15	.16
(8) 1" polystyrene insulation & 1/2" gypsum board applied direct to wall surface	.12	.12	.13	.13	.14
(9) 1" polyurethane insulation & 1/2" gypsum board applied direct to wall surface	.10	.11	.11	.11	.12
(10) Same as (7) plus cores filled with loose-fill insulation	.09	.10	.11	.12	.15
(11) Same as (8) plus cores filled with loose-fill insulation	.08	.09	.10	.11	.13
(12) Same as (9) plus cores filled with loose-fill insulation	.08	.08	.09	.10	.11
(13) R-7 blanket insulation, 1/2" gypsum board	.09	.10	.10	.10	.11

Table 2.14 — *CONSTRUCTION: 8" Hollow Concrete Block.*

Density of concrete used in blocks, lbs/cu ft	60	80	100	120	140
Approximate weight of masonry, lbs/sq ft	24	32	40	47	55
WALL DETAILS	"U" VALUE				
(1) No insulation	.32	.34	.38	.43	.55
(2) No insulation, 1/2" gypsum board on furring strips	.21	.23	.25	.27	.31
(3) No insulation, 1/2" foil backed gypsum board on furring strips	.15	.15	.16	.17	.19
(4) Loose-fill insulation in cores	.12	.14	.18	.21	.35
(5) Loose-fill insulation in cores, 1/2" gypsum board on furring strips	.10	.12	.14	.17	.24
(6) Loose-fill insulation in cores, 1/2" foil backed gypsum board on furring strips	.08	.10	.11	.12	.16
(7) 1" rigid glass fiber insulation & 1/2" gypsum board applied direct to wall surface	.14	.14	.15	.15	.17
(8) 1" polystyrene insulation & 1/2" gypsum board applied direct to wall surface	.12	.12	.12	.13	.14
(9) 1" polyurethane insulation & 1/2" gypsum board applied direct to wall surface	.10	.10	.11	.11	.12
(10) Same as (7) plus cores filled with loose-fill insulation	.08	.09	.10	.11	.14
(11) Same as (8) plus cores filled with loose-fill insulation	.07	.08	.09	.10	.12
(12) Same as (9) plus cores filled with loose-fill insulation	.07	.07	.08	.09	.11
(13) R-7 blanket insulation, 1/2" gypsum board	.09	.10	.10	.10	.11

Table 2.15 — CONSTRUCTION: 12" Hollow Concrete Block.

Density of concrete used in blocks, lbs/cu ft	60	80	100	120	140
Approximate weight of masonry, lbs/sq ft	34	45	55	67	78
WALL DETAILS	"U" VALUE				
(1) No insulation	.24	.29	.34	.38	.50
(2) No insulation, 1/2" gypsum board on furring strips	.18	.20	.23	.25	.29
(3) No insulation, 1/2" foil backed gypsum board on furring strips	.13	.14	.15	.16	.18
(4) Loose-fill insulation in cores	.08	.10	.13	.17	.29
(5) Loose-fill insulation in cores, 1/2" gypsum board on furring strips	.08	.09	.11	.13	.21
(6) Loose-fill insulation in cores, 1/2" foil backed gypsum board on furring strips	.07	.08	.09	.10	.14
(7) 1" rigid glass fiber insulation & 1/2" gypsum board applied direct to wall surface	.12	.13	.13	.14	.15
(8) 1" polystyrene insulation & 1/2" gypsum board applied direct to wall surface	.10	.11	.12	.12	.13
(9) 1" polyurethane insulation & 1/2" gypsum board applied direct to wall surface	.09	.10	.10	.11	.11
(10) Same as (7) plus cores filled with loose-fill insulation	.06	.07	.08	.10	.13
(11) Same as (8) plus cores filled with loose-fill insulation	.06	.07	.08	.09	.11
(12) Same as (9) plus cores filled with loose-fill insulation	.05	.06	.07	.08	.10
(13) R-7 blanket insulation, 1/2" gypsum board	.09	.09	.10	.10	.11

2.3.2 Dynamic Analysis For Estimating Energy Requirements

There are two methods of analysis for estimating the energy requirements for heating and cooling buildings: 1) The "steady state" method, which is based on a steady state or steady periodic heat flow concept and where heating and cooling requirements are calculated for a specific outdoor temperature for winter and summer; and 2) "dynamic analysis", where heating and cooling requirements are determined according to the response of a building to hourly changes in weather conditions. The 'steady state' method has been accepted for many years because it is not complicated. The 'dynamic analysis method' was not used due to the complexity and expense of the calculations even though it is more accurate because it takes into account heat storage capacity of building elements. This leads to the fact that a distinction must be made between masonry and non-masonry construction because of the difference in heat storage capacity [11].

In a part of a research program to improve performance test procedures and criteria for computing heating and cooling loads, the National Bureau of Standards (NBS) in the United States conducted a series of tests on a full-scale building [11,12]. The concrete masonry building shown in Figs. 2.2 and 2.3, was built in the environmental laboratories of the NBS where weather conditions could be controlled over the range of -50^{0}F to 150^{0}F and relative humidity from 15 to 85%. Ten tests were conducted with the variable studies falling into three categories: insulation, windows and internal mass. The results of the tests showed that:

1. The effect of internal mass on the thermal behaviour of the building was small. It was suggested that an internal mass may influence the thermal behaviour of a lighter weight structure to a greater degree

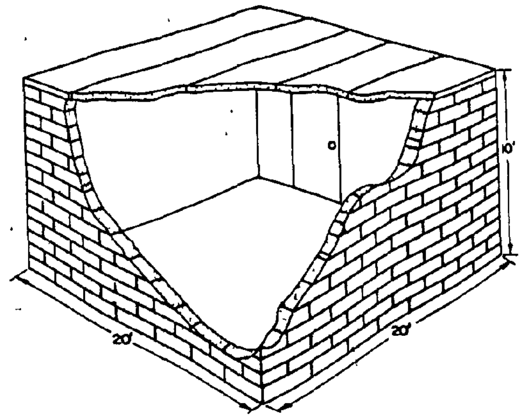


Fig. 2.2 — The Experimental Building Without Windows, Internal Mass, and Insulation.

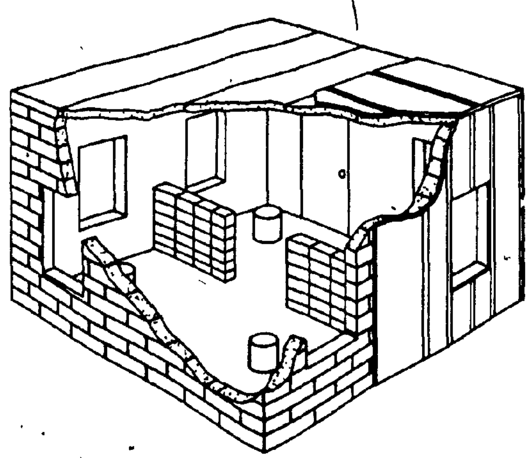


Fig. 2.3 — The Experimental Building Showing Windows, Internal Mass, and Insulation on the Outside of the Building.

then the masonry building.

2. It was verified that placing insulation on the exterior was very effective in reducing and controlling variation of the indoor temperature. The indoor temperatue was reduced by one half when insulation was placed on the outside as compared to the inside.

3. The use of windows affected the thermal behaviour of the building significantly. The maximum heating load with windows was 57 percent higher than the same building with no windows. Use of double panes reduced the maximum heating load by 9%.

Table 2.16 shows the location of insulation and windows as well as the results of the tests. The table shows also an interesting result which is a comparison of maximum heat flow rates as measured and calculated by 'steady state' methods, and 'dynamic analysis' methods.

Table 2.16 — Comparison of Maximum Heat Flow Rates as Measured and Calculated by Two Methods.

Test No	Insulation	Windows	Maximum Heat Flow Rate - Btu/h: Steady-State Method	Response Factor Method	Measured
6	None	Single pane	15,135	11,558	11,372
7	Inside	Single pane	4,470	2,814	2,748
8	Outside	Single pane	4,748	3,047	2,811
9	Outside	Double pane	4,499	2,525	2,700
10	Outside	Dougle pane	8,150	7,144	6,321

The computer program using dynamic analysis predicted the maximum heat flow rates within an average of 4.3% of the actual measured rates. The maximum heat flow rates calculated by 'steady state' methods were between 29 and 69% higher than measured rates. One of the conclusions of the National Bureau of Standard Tests was: "It was shown that steady state methods of heating load calculations could result in over-sizing heating equipment by 30% and more for this particular building and imposed exterior conditions if the lowest outdoor temperature was selected as the design temperature". Thus it is clear that concrete masonry posesses heat storage capacity which is not taken into consideration by the 'steady state' method.

2.3.3 Solar Energy Walls

Solar energy is the most abundant and permanent source of energy available. Its impinging on the earth's atmosphere is dilute and is received intermittently at any point on earth. It requires different methods of collection and utilization than forms of energy widely used before. It is only within the past 40 years that solar energy has been with us as a technology. During this period, the use of solar energy has progressed very slowly, and the state of the art is definitely in its infancy. But, due to the energy crisis, the use of solar energy received an enthusiastic response from governments, architects, engineers, builders and home owners. Solar energy owners are offered in many universities and colleges, and research programs are underway in many countries.

2.3.3.1 Passive Solar System

Solar energy systems for buildings are divided into two categories: 1) Active solar system, and 2) Passive solar system. The active solar system consists of solar collectors, heat exchangers, thermal storage elements, pumps and electric controls. This system has received the widespread attention which could be due to: 1) the hardware used;

2) heating and cooling the building and for hot water 3) petrofitted to existing buildings 4) special forms and proper orientation of buildings are helpful but not essential.

Little attention has been given to passive solar heating systems, where buildings themselves are the collector, storage device and heat distributer, though passive solar heating is an ancient art. For hundreds of years people who lived in deserts or in North America, intuitively constructed homes with adobe masonry walls or similar materials as solar heat storage receivers. They built thick walls to absorb heat and store it for the entire night. It was overlooked that the passive solar systems may well generate a significant contribution to the energy problem. Passive solar heating is becoming a science and research programs and studies are underway to find effective methods of calculating the proper size of components using computer analysis. It is believed that houses with passive solar energy systems need significantly less energy for heating. One authority on this subject said: "The intriguing thing about these (passive) approaches is not only that they are architecturally more compatible with normal building practices and aesthetically more pleasing, or that they should be lower in cost than active systems, but that they apparently work so well. Not only do they work well, but they appear to work relatively well in crummy climates with a great amount of diffuse solar energy where active systems can hardly perform effectively. This is because a passive system is always working when any sun is shining. It collects every bit of energy, direct or diffuse that comes through the glazing. An active system has a threshold and does not begin to work until a certain temperature is achieved. This largely compensates for the fact that the energy losses in the passive system are undoubtedly greater than they are in an active system. I also feel that passive heating can be made even more comfortable for the occupants of the building, than conventional fossil or active solar systems [13].

2.3.3.2 Design of Solar Wall

Design of a concrete masonry passive solar wall is still at a rudimentary stage. However, concrete masonry walls have been employed as passive solar heating systems and some important facts have been developed. Fig. 2.4 represents a passive solar design [13]. The Trombe wall which was erected by Felix Trombe, consists basically of a heavy concrete masonry wall, thick enough (30 to 45 cm depending on the material) that the heat conducted does not reach the room till the night. The wall was painted black on its exterior south side so that its surface allows a maximum of solar radiation to be absorbed. A pane of glass is installed in front of the wall creating 3" air space.

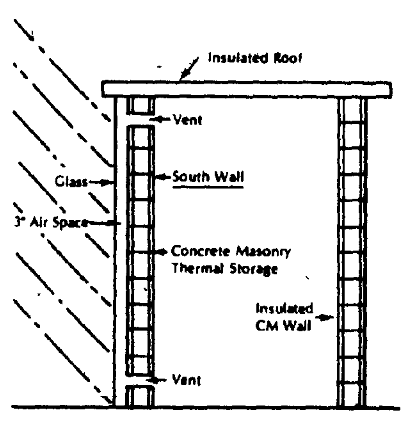


Fig. 2.4 — The Trombe Wall.

Two carefully sized openings, one at the top and one at the bottom of the wall, provide two vents which connect the room to the air space during the heating season. The heat circulates into the room by thermal buoyancy currents. In summer, the interior vents are sealed and exterior vents through the glass are provided to circulate cool air between the glazing and the concrete masonry wall. The Trombe wall system works best in dry regions which never have more than a couple of cloudy days in a row. If it cools excessively, the storage wall will become a source of discomfort.

Design of concrete masonry thermal storage walls require consideration of several factors beyond the normal structural and aesthetic. Two important properties are heat capacity and thermal conductance. These properties are directly related to the dynamic thermal response of the concrete masonry wall. Dynamic thermal calculation can be made by computer simulation. A number of such studies have been made. These studies provide helpful information in the preliminary design of passive solar system using concrete masonry walls. The following paragraphs are taken from ref. 13 and present interesting information from these studies.

Concrete masonry thermal storage walls should be placed so as to receive the maximum solar illumination. South walls are very effective. The mass of a south wall used for thermal storage should leave a heat storage capacity that is an equivalent to about 150 pounds of concrete masonry for each square foot of glazing; a heat capacity of about 30 BTU^{o}F/sf of glass. Better performance can be achieved if the concrete masonry storage wall is located so that it is directly heated by sunlight absorbed within the heated space. When concrete masonry storage wall is placed directly in front of the glass, there is an optimum thermal conductance-heat storage capacity relationship. If wall conductance is too high, storage loses too much heat during the charging period. This prevents storage from attaining higher temperatures and storing greater amounts of heat. If wall conductance is too low, storage attains such high temperatures during charging periods that it loses too much heat through the glazing.

Several studies have indicated the optimum thickness of a concrete masonry thermal storage wall is 12 to 18 inches. Special surface treatments: roughness, blackening surface, grooves, etc., may improve convective and radiant heat exchange of a concrete masonry thermal storage wall.

Passive solar heated building requires more insulation of exterior walls than other heating systems. If insulation is to be employed in conjunction with exterior concrete masonry walls, it could be placed on the exterior side or in the center to take greater advantage of the thermal heat storage capacity of the masonry.

Table 2.17 presents selected properties of some concrete masonry walls that might be considered in the design of a thermal storage wall for passive solar heating. Data included time lag which is defined as the length of time between maximum temperature on one side of the wall and maximum temperature on the other. It gives an idea as to how long it will take to heat up (charge) the various concrete masonry thermal storage walls. Other information of interest in the selection of a storage wall includes conductance and the heat capacity of the wall expressed in BTU^{O}/F/100 s.f.

Table 2.17 — Selected Properties of Concrete Masonry Walls for Use in Solar Heating.

WALL DESCRIPTION	CM Unit Weight, lb/cf	Wall Weight, lb/sf	Conductance BTU/sf/hr °F	R-Value	Thermal Storage BTU/°F/100 sf	Time Lag, hr
8" concrete block with hollow cells filled with bulk insulation	60	30	0.13	7.46	490	4.6
	80	40	0.17	6.06	645	4.2
	100	50	0.21	4.85	800	3.9
	120	60	0.26	3.79	950	3.5
	140	70	0.51	1.98	1140	3.8
Same as above with 2 inches rigid insulation on exterior surface	60	SAME AS ABOVE	0.06	17.46	SAME AS ABOVE	5.9
	80		0.06	16.06		5.5
	100		0.07	14.85		5.0
	120		0.07	13.79		4.6
	140		0.08	11.98		4.6
Same as above with 4 inches rigid insulation on exterior surface	60		0.04	27.46		7.4
	80		0.04	26.06		7.0
	100		0.04	24.85		6.5
	120		0.04	23.79		6.0
	140		0.05	21.98		6.0
Same as above with 6 inches rigid insulation on exterior surface	60		0.03	37.46		8.9
	80		0.03	36.06		8.5
	100		0.03	34.85		8.1
	120		0.03	33.79		7.6
	140		0.03	31.98		7.6
8" concrete block with hollow cells filled with dense grout or concrete with a unit wt. of 144 lb/cf	60	50	0.29	3.42	815	4.9
	80	60	0.39	2.55	970	4.4
	100	70	0.54	1.85	1125	4.3
	120	80	0.73	1.37	1280	4.0
	140	90	0.85	1.18	1465	4.1
Same as above with 2" rigid insulation on exterior surface	60	SAME AS ABOVE	0.08	13.42	SAME AS ABOVE	5.9
	80		0.08	12.55		5.5
	100		0.08	11.85		5.0
	120		0.09	11.37		4.5
	140		0.09	11.18		4.5
Same as above with 4" rigid insulation on exterior surface	60		0.04	23.42		7.4
	80		0.04	22.55		6.9
	100		0.05	21.85		6.4
	120		0.05	21.37		6.0
	140		0.05	21.18		5.9
Same as above with 6" rigid insulation on exterior surface	60		0.03	33.42		8.8
	80		0.03	32.55		8.4
	100		0.03	31.85		8.0
	120		0.03	31.37		7.5
	140		0.03	31.18		7.4
12" concrete block with hollow cells filled with bulk insulation	60	45	0.09	10.98	720	8.0
	80	50	0.12	8.70	945	7.8
	100	70	0.15	6.80	1170	7.3
	120	85	0.19	5.18	1390	6.7
	140	100	0.39	2.59	1545	7.1
12" concrete block with hollow cells filled with dense grout or concrete with a unit wt. of 144 lb/cf	60	80	0.21	4.85	1295	8.3
	80	95	0.27	3.69	1520	8.2
	100	110	0.37	2.74	1740	7.6
	120	125	0.48	2.08	1965	7.1
	140	140	0.56	1.79	2115	7.2

Chapter III

APPLICATION OF HAND-LAID PREFABRICATED CONCRETE MASONRY WALL PANELS

3.1 MSI Panel System

Preassembled panels have been introduced by Masonry Systems Incorporated (MSI) of Westminister, Colorado. This system utilizes factory assembly of panels manufactured in two different ways [14].

One procedure used by MSI involves the dry stacking of panels, without mortar, at the factory site. After six or eight panels have been laid up side by side, steel reinforcement is placed in the hollow cells of the block and the panels are grouted solid, thus creating concrete block panel up to 8 x 30 ft. in size. A three-man crew consisting of one mason, a crane operator, and one labourer build 1800 square feet of panel per day at the factory in this way. Job site erection requires five men: a crane operator, a pickup man on the ground, and a mason and two helpers up on deck. Installation of an 8 x 30 ft. load bearing partition is generally accomplished in 11 minutes.

A more recent development by MSI has been the "Rock-A-Block" procedure which is similar to the dry stacking and grouting method, but which utilizes a rigid steel frame against which blocks are laid up, making it possible to preassemble panels at the job site or at the factory.

A second type of MSI panel consists of factory assembled facing units using one of the organic adhesive mortar, Threadline. The concrete masonry units used with the thin adhesive in these panels required that the bearing surfaces of the block to be ground to a closer tolerance than is normal for machine production. However, the company has since developed a newer mortar which is laid down thick enough that block size irregularities are taken up, in the mortar, and thus avoiding the necessity of grinding blocks. The new mortar consists of portland cement, sand, water, latex and ground limestone [14].

3.2 VET-O-VETZ MASONRY SYSTEMS

Vet-O-Vetz Masonry Systems, a division of Vet-O-Vitz Bros. Inc., Cleveland, Ohio, [5] has been a major pioneer in the field of masonry panelization for the past decade. They have innovated and developed numerous new materials, equipment, connection details, erection devices and systems to advance the art of "Prefab Masonry Panels". The panels are preassembled, either in an on-job-site or off-job-site, by semiautomated plant. Vet-O-Vetz Masonry Systems has involved a portable plant for an on-job-site masonry panelization technique. This system allows construction to proceed under the precision controlled environment insuring safe working conditions. In both, on-site and off-site operations, masonry units are hand laid, in vertical orientation along prelevelled setting beds, using mortar of high compressive and tensile strengths.

Veto-O-Vetz Masonry Systems developed an automatic mortar spreader, to lay down controlled bed joints using conventional mortar, and greatly increases the masons' productivity of on-site assembly of panels. The first job utilizing this system is in Southfield, Michigan, just outside Detroit. It is a 14-story office tower designed in the load bearing engineered concept. Panels are placed one on top of the other like vertical columns with stacked windows in between. Five brick layers laid all the bricks and blocks for this job. A 2400 sq ft building with 4000 sq ft of wall area was completely enclosed in just 2-1/2 days in sub freezing weather by five men and a crane which was on the job for only 5-1/2 hours [15].

CHAPTER IV

PREFABRICATED TOMAX WALL PANELS

4.1 TOMAX MACHINE SYSTEM

This automated machine for manufacturing masonry wall panels using standard concrete units was invented by Paul Thomas of Phoenix, Arizona. The machine was built by Builders Equipment Company in the same city. It was introduced to the North American market after several years of testing and development. Today, Tomax machines are installed in the United States (California, Arizona, Kansas, Ohio), in Austrialia, and in Canada (Montreal, Toronto, Port Colbourne). The exclusive licence rights for the Tomax system are owned by Pacific Prestressed Products Inc., of New Port Beach, California. They offer exclusive franchises in areas across the United States and around the world.

The machine consists of the following basic parts: [16]

1) The superstructure comprises of the hoisting motor that lifts the elevating platform and also the vertical index or storey-pole which accurately locates the platform for each of the courses of block to be laid.

2) Theelevating platform which carries the layer of blocks on a set of tracks, which allows the layer of blocks to travel from one end of the platform eo the other.

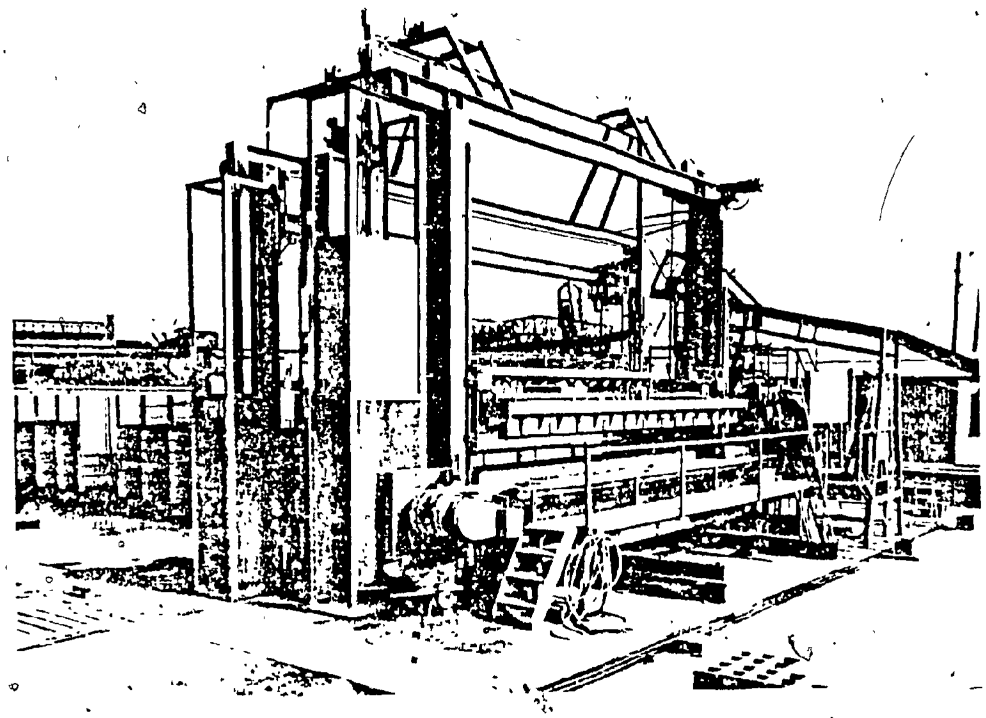


Fig. 4.1 — An overall view of the Tomax wall panel machine [16].

3) The block-layer which performs all the operations required to lay a block. It lays the block, tempers the mortar, inserts the joint reinforcement, makes the head and bed joints, and trowel the joints.
4) The block conveyor brings the block up onto the elevating platform and along the entire length of the platform to the block layer.
5) The mortar mixer — a combination of mortar mixer and skip loader, which brings the mortar up to the block layer as required.
6) The panel conveyor, which consists of a series of pallets 20" wide and 21" long that are fastened side by side, 6'-0" on center, to two I-beams which act as a carriage and allow the pallets to travel directly under the elevating platform and the block layer.
7) The panel stripping beam that removes the cured panels from their pallets and deposits them on the yarding conveyor.

4.2 Tomax Machine Operation

The first step in the Tomax machine operation is the entry of the block to the primary conveyor. (Reference to Figs. 4.2 - 4.8 will be useful in understanding the operation). From the discharge point of the primary conveyor the block makes a 90^o left turn onto the machine's infeed conveyor belt. Along this belt the block layer at whatever position along the elevating platform it happens to be at the time.

At the proper point of the wall course being laid, a single block is engaged by a clamp which lifts it from the infeed conveyor and moves it across to its proper point in the wall course and places it firmly onto the previously mortared and reinforced bed joint. With the wall clamped

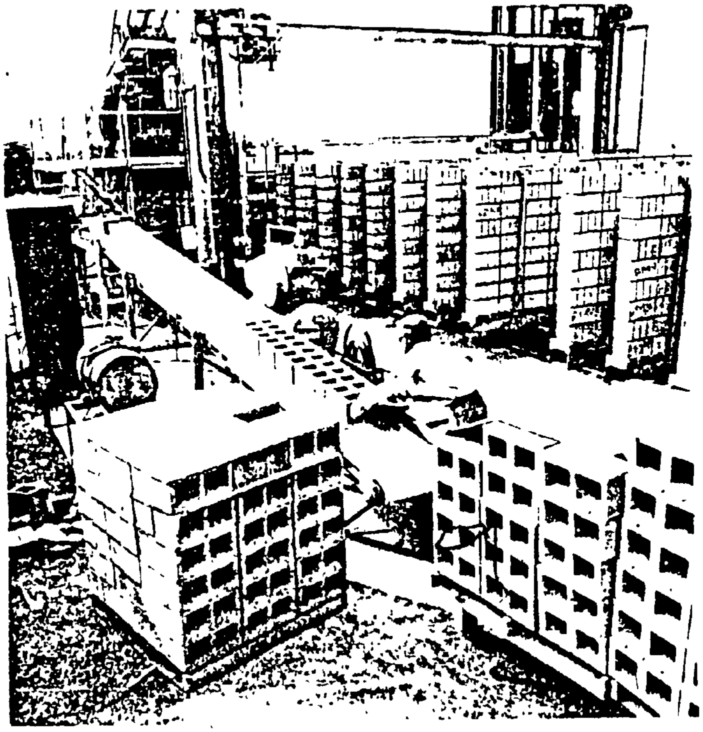


Fig. 4.2 — The blocks begin their journey on the conveyor belt which feeds the machine [16].

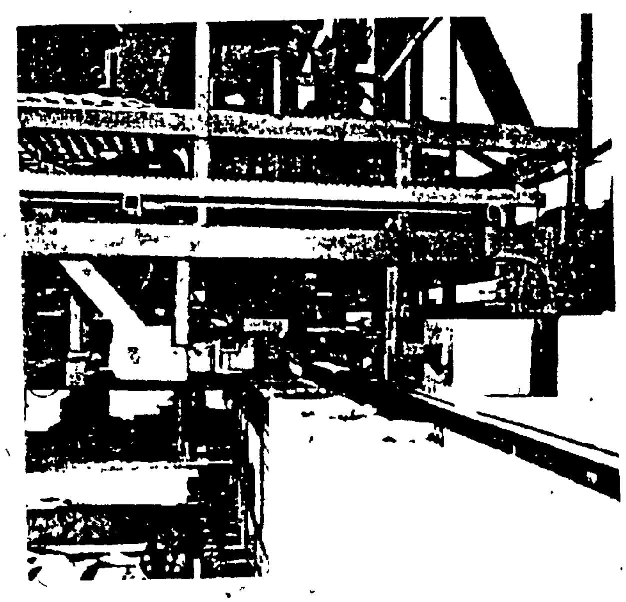


Fig. 4.3 — Individual block are moving along the infeed conveyor. Just beyond the single block entering the machine, the blocks preceeding it is in the clamp and partially lifted for placement [16].

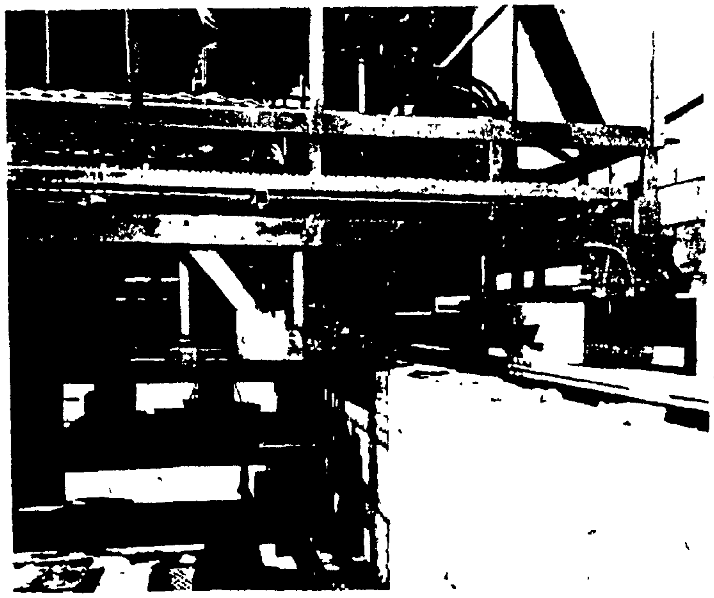


Fig. 4.4 — *Here the clamp has moved the block from the infeed conveyor to its proper position over the wall and is about to put the block in place on the mortared bed joint. A sufficiently sharp eye can also see the joint reinforcing in place* [16].

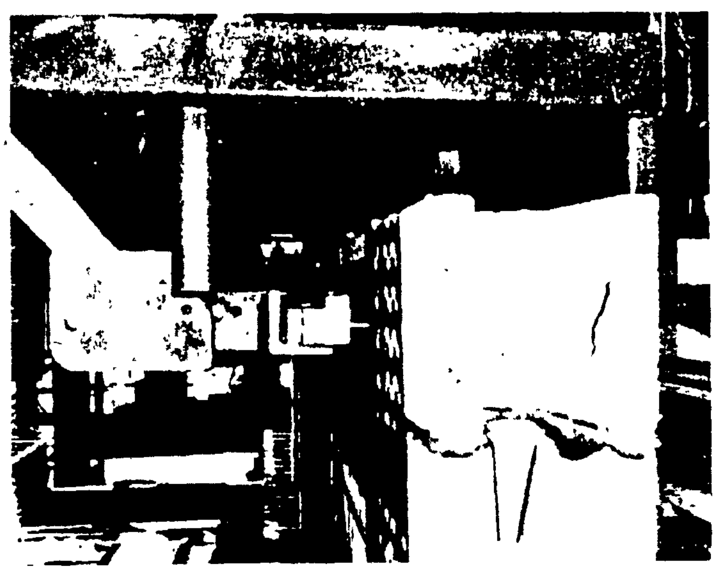


Fig. 4.5 — *With the wall clamped to stabilize it, the head joint-filler (left-center) is in position and filling the joint; the head box, containing mortar in the down position for filling* [16].

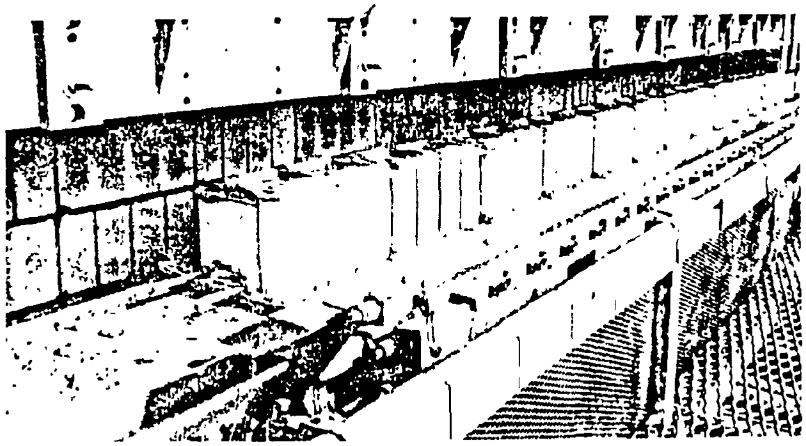


Fig. 4.6 — Changes in panel size, block or style are accomplished by a simple indexing method [17].

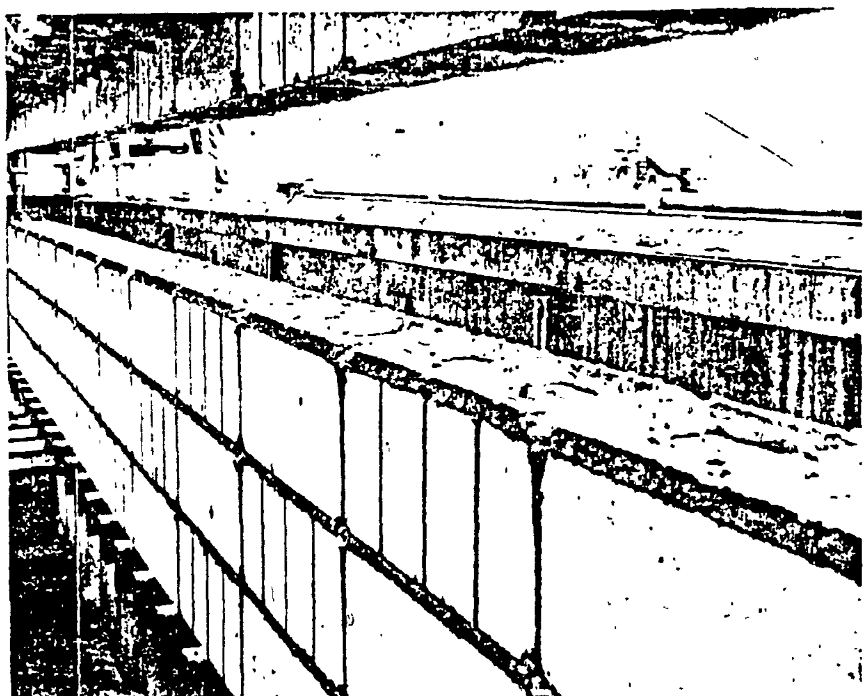


Fig. 4.7 — Each course is vibrated into position. Three courses of blocks about to receive the fourth course from above [17].

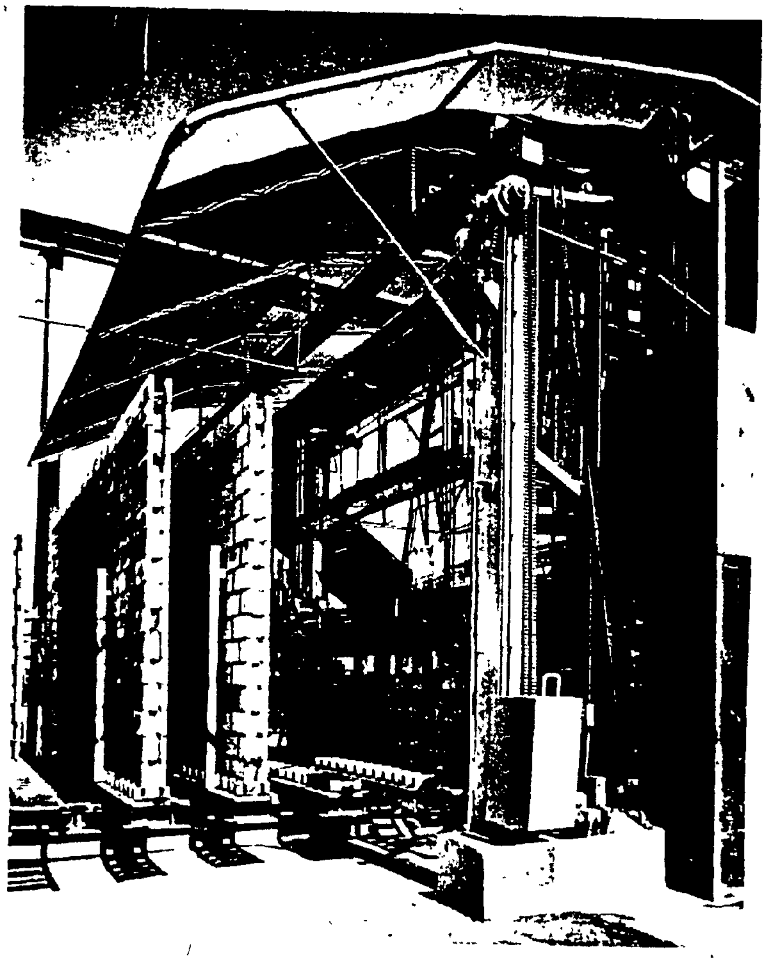


Fig. 4.8 — This view from the discharge side shows how finished walls are indexed out from under the machine while empty pallets move under the machine for laying of succeeding walls [17].

to stabilize it, the head joint filler moves into position at the joint of the block just mentioned and the block that preceded it into the wall, and fills the head joint. This process is repeated until the full length of the course is laid, whereupon the elevating platform is raised to the next course level and the block layer is returned to the starting point of the wall. When the final course of a wall has been completed, the elevating platform is raised sufficiently to permit clearance for the wall to move out on its pallet from under the machine [16].

All of the mentioned steps are fully automatic and automatically programmed in the control console at the machine. The only manual operations are the raising and lowering of the elevating platform, the return of the block layer to the starting point of a course, and the travel of the material bucket to the mortar mixers (one each for the bed and mortar joints), all of which operations are performed by the operator's manipulation of powered controls. Also, horizontal reinforcing is placed by hand. After twenty-four hours, the panels are strong enough to be moved from the pallet by forklift or crane and taken to a storage area for further curing [16].

4.3 Tomax Panels

The machine can produce panels in any size up to 12 ft. in height and 24 ft. in length within the modular width and length of the block. Blocks can be of various thicknesses from 6 to 12 inches in running or stack bond, and can thus lay 270, 8x8x16 in. blocks in 35-40 minutes. This rate is above the national average for a full day's work for a mason.

The Tomax Panels have many advantages and desirable features. The following is a list of main advantages and features: [17].

1. Solid bonded head joints. A head joint filled solid with 5000 psi mortar.
2. Full bed joint. Every cross-well as well as the face shell of the block is meshed completely in a ribbon of mortar.
3. The mortar is 5000 psi sand and cement mortar, with no epoxies, no additives of any kind. These tremendous strengths, combined with bonding under vibration, make a panel that is built out of pieces become a truly monolithic wall.
4. Panels are available in any size up to 24 feet long and 12 feet high, within the modular widths and lengths of the block.
5. The concrete block panel offers a much more economical way for the architect or engineer to express his design capabilities. Because panel sizes can be changed in modular sizes almost at will, versatility to the architect and engineer is not limited to any preconceived mold of form.

6. There are no code restrictions within this panel, because there is nothing new in the materials, only in the concept of handling.
7. The panel has built-in quality control. If a panel factory should decide to save on the mortar, the panels would arrive at the job site in pieces.
8. The panel can be pre-inspected at the plant site, thereby eliminating the disagreeable uncertainty of wondering what the completed job will look like.
9. There is considerable shortening of job time, because panels are built under factory conditions ahead of time. Installation is much faster because panels represent thousands of square feet a day instead of a few hundred square feet a day.
10. Panels can be prefabricated with built-in grade beams.
11. Buildings with panels using built-in grade beams can now go to pier-type construction, thereby saving costly work and cutting days off any construction project.
12. Block panels can be completely waterproofed by immersing the panel in silicone waterproofing agents. Once the waterproofing agent is on the interior core of the block, where it is unaffected by wind and abrasive agents and untouched by the ultraviolet rays of the sun, it is waterproofed for the life of the building.
13. Fire ratings, insulation and sound-proofness have all been vastly increased by the use of solid head joints, instead of a variable amount of mortar.
14. Because of the enhanced strength, THOMAS-WALL panels can be laid flat, and then ground to expose the natural aggregate in the concrete block. Or the panels can be faced with either terrazzo or exposed aggregate.

15. Using the THOMAS-WALL panel with the exposed aggregate facing as a load bearing wall, the steel framework required for conventional precast work is saved.
16. Using the panel system, automatic expansion joints are introduced, eliminating common cracking.
17. These panels are tied to each other, to the floor, and to the roof by any of the conventional practices within any reinforced masonry code.

There is a basic difference in the marketing concept between Tomax Panel and those manufactured by other systems such as MSI and Veto-O-Vitz. With the latter system, panels of concrete block units are selected and scheduled from the architect's plans which may have been conceived without the architect having any intention to use a system of panels for construction. The job is changed to panels by the panel manufacturer, who displays a high degree of flexibility in what he can do in the way of panel sizes, shapes, etc. In the case of Tomax Panels approach, however, the panel manufacturer makes and inventories "Standard Panel Sizes". The architect designs the building for panels from the start with a knowledge of what panels are available and how they are put together on site.

4.4 PANEL STRENGTH TESTS

4.4.1 ENGINEER'S TESTING LABORATORIES

The first Tomax panel strength tests were conducted by Engineers Testing Laboratories, Phoenix, Arizona. The purpose of the test was to determine:

1. The mortar strength being used.
2. The actual strength of the walls.
3. If the panels would meet the safety factor of normal masonry.

Two panels were tested in a horizontal position and supported at the ends by wood 4x4'. Concrete masonry units were applied gradually as a uniformly distributed load.

Panel No. 1 was made with 8"x8"x16" units laid in a running bond. It had 12 feet clear span and 6 feet width. At the time of the tests, the age of the mortar was 28 days and the average strength was 5288 psi.

Loading	Deflections — inches
35 psf, dead load of panel	0.00
52.5 psf, imposed live load	0.025
70.0 psf, imposed live load	0.054
87.5 psf imposed live load	failure

At failure, mortar shear was followed by a diagonal sheer of the block. Panel No. 2 was made with 8"x8"x16" units laid in stacked bond. The panel had a clear span of 12 feet and a width of 4 feet. At the time of the test, the age of the mortar was 7 days.

Loading	Deflections — inches
35 psf, dead load of panel	0.000
44 psf, imposed live load	0.006
61 psf, imposed live load	0.029 (failure)

The failure load remained in place for approximately one minute before failure, but deflection was visibly increasing during this interval. Failure occurred through a vertical mortar joint.

The tests showed that the two panels tested had flexural strength considerably greater than necessary to sustain a wind load of 15 psf.

Tests were also made on two panels in an upright position. The two panels, 2 feet high by 12 feet long, were set 4 feet apart. Wood pallets were placed spanning the 4 feet and the pallets were then loaded to failure. A total load of 5656 lbs or 213 lbs per linear foot were in place for a period of at least one minute before failure occurred.

The results of the tests prove that properties and quality of Tomax wall panels are superior to those of conventional wall panels.

4.4.2 TESTS BY THE UNIVERSITY OF TOLEDO

Tests concerned with the lateral strength of Tomax wall panels were carried by The Research Foundation of the University of Toledo, Ohio. The panels were fabricated a month prior to testing and were 4 feet wide and 8 feet high.

The test device consists of a strong wooden frame which is changed to the top and bottom of the wall panel. Uniformly distributed loading obtained with incremental increases in air pressure in a plastic bag interposed within the frame. Bag pressure was controlled by a manually operated valve in the air compressor tank and was measured by means of mercury "U" tube manometer. This test system, which provides a uniformly distributed load over the face of the wall is considered to be more accurate than beam type tests where load is applied by piling on weights, or where failure is induced by a concentrated load.

Table 4.1 shows the test results as well as it indicates the size of units and type of bonds which the panels were constructed with. It should be noted that some walls included dur-o-wal joint reinforcing while others did not, but since these tests were made in the vertical span, whereas

Wall Description	Failure load (psi)
8" block, stack bond	140
8" block, stack bond	140
8" block, running bond	119
8" block, running bond	161
12" block, stack bond	385
12" block, stack bond	343
12" block, running bond	294
12" block, running bond	238

Table 4.1 – Description of Panels and Test Results.

joint reinforcing is effective in the horizontal span only. Joint reinforcing was of no consequence. It is also important to mention that none of the tested walls included grouted cores. In actual practice, the panels would usually have at least two fully grouted cores which would obviously impart additional strength in the vertical span. At the time of the test the age of the mortar was 28 days and its strength was 4,920 psi.

In all cases the panels failed in bond between mortar and block, usually at or near the mid height of the panel.

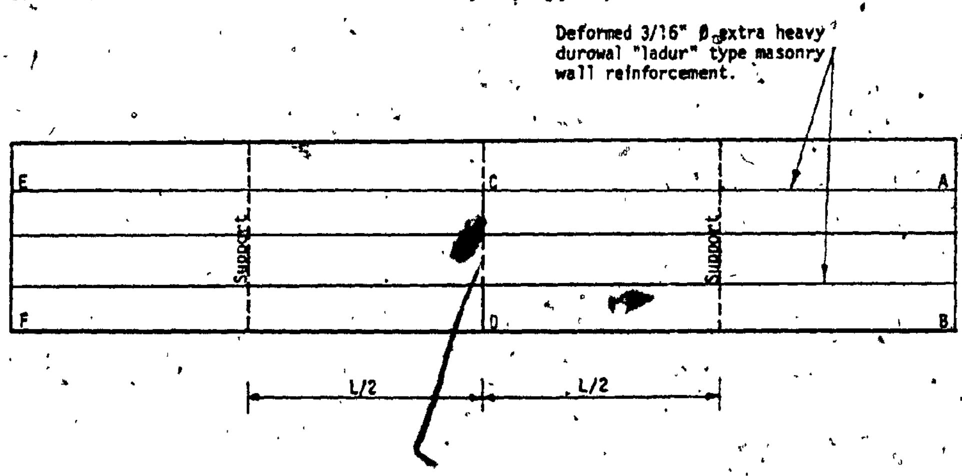
The results show Tomax panels are superior in strength to panels built by a hand laid system. The highest strength achieved by an 8 inch wall constructed with sand and gravel blocks laid up with type M mortar (the strongest mortar covered in ASTM specifications) was 96 psf in comparison to the average 140 psf achieved for 8 inch walls in the test being reported upon [18].

4.4.3 TESTS BY BEST'S BLOCKS, INC.

Load tests on concrete block panels were carried out by personnel from Best's Blocks, Inc. at their plant in Union City, and were witnessed by Hales Testing Laboratories. Loadings were made on 4-6-73 and final deflections measured on 4-9-73 [19].

Panels were either stack or running bond with support points approximately 10'0" apart consisting of one cell filled with grout and a 1/2"Ø bar in the middle of the cell. No other cells were filled with grout. All joints were filled with mortar and two horizontal bed joints in each panel had "Dur-O-Wal" reinforcing placed in them. The panels were 20'0" long with four vertical courses giving a height of 32"±. The panels were laid on their sides with parallel supports under the grouted cells; thus providing a central span of 10'0"± and two cantilever spans of 5'0"± on the outsides.

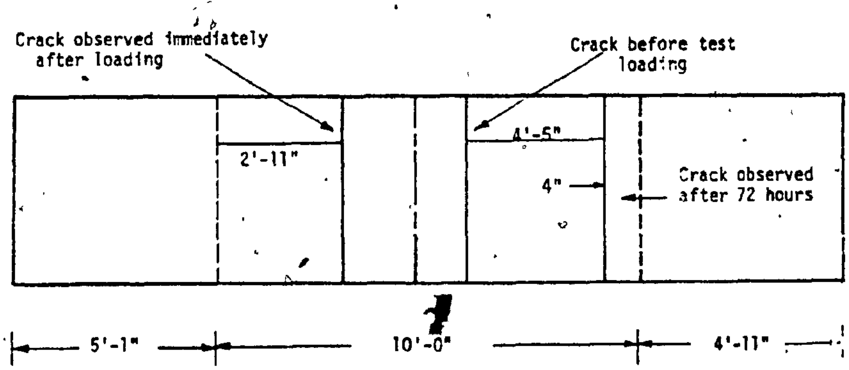
The panels were then uniformly loaded with cement bags (1 layer of bags consisted of 15 bags, 94 lbs) and deflections measured. The deflections were measured after different uniform loadings were applied (a) immediately after the loadings were applied, and (b) approximately 72 hours after the loadings were applied, in order to allow for creep and shrinkage. The panels were examined for cracks before loading, after loading, and after 72 hours under load. Dimensions and physical damages are noted on the sketches for each test. Vertical deflections were measured at the six stations indicated as A, B, C, D, E and F.



Loading from 1 layer of bags	27 lbs/ft^2
Assumed dead load of panel	40 lbs/ft^2
Total uniform loading with one bag	67 lbs/ft^2
Loading from 2 layers of bags	54 lbs/ft^2
Total uniform loading with two bags	94 lbs/ft^2
Loading from 3 layers of bags	80 lbs/ft^2
Total uniform loading with three bags	120 lbs/ft^2

Table 4.2—Loadings.

Panel #1. Stack Bond. This panel was damaged before the test began and therefore only loaded with one layer of cement bags. We understand it was dropped prior to testing.

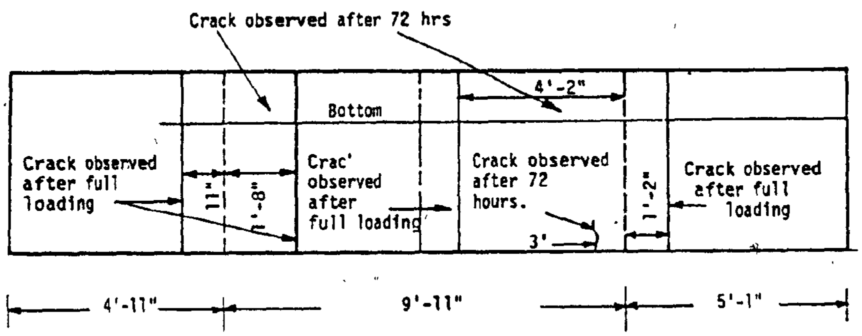


Note: all cracks were in the mortar joints and were hairline cracks and were barely observable.

	Deflections — inches					
Loading	A	B	C	D	E	F
1 bag 67#/ft^2 incl. dead load	+1/16	0	+1/8	+1/8	+1/16	0
after 72 hrs. with above loading	+3/16	+1/16	+1/8	+1/8	+3/16	+1/16

Note: Deflections indicated as + are toward the ground.

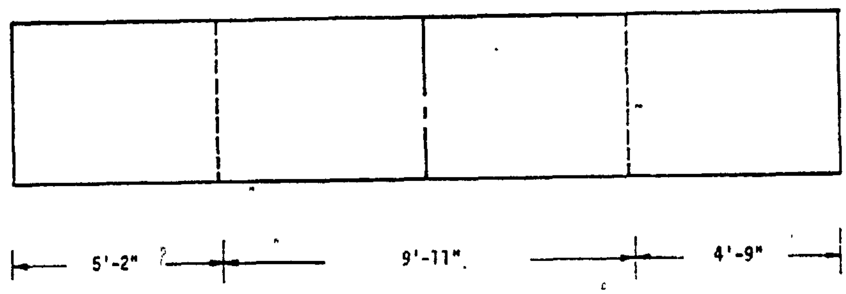
Panel #2. Stack Bond. This panel did not have any grouted cells with reinforcing. This panel was loaded with two layers of cement bags. Deflections were measured after loading with one layer of cement bags, after loading with two layers of cement bags and after 72 hours under the two layer loadings.



Note: all cracks were in the mortar joints and were hairline cracks and were barely observable.

LOADING	Deflections — inches A	B	C	D	E	F
1 bag 67#/ft² incl. D.L.	0	0	0	0	0	0
2 bags 94#/ft² incl. D.L.	0	0	+1/8	+1/16	+1/8	+1/8
after 72 hours with above loading	+1/8	+1/8	+1/8	+1/16	+3/16	+3/16

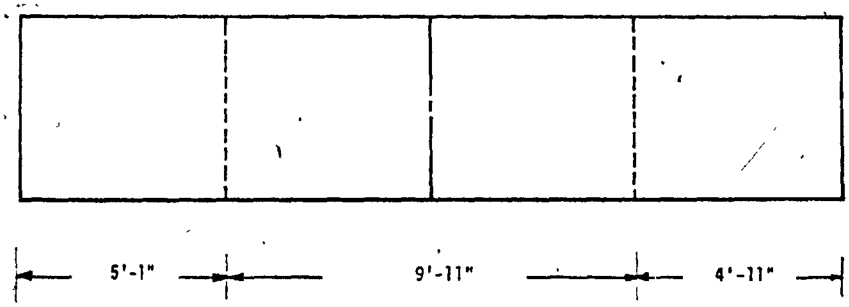
Panel #3. Running Bond. This panel was loaded with two layers of cement bags. Deflections were measured after loading with one layer of cement bags, after loading with two layers of cement bags, and after 72 hours under the two layer loading.



Note: no cracks were observed in this panel after 72 hours under load.

LOADING	Deflection — inches A	B	C	D	E	F
1 bag 67#/ft^2 incl. dead load	0	0	0	0	0	0
2 bags 94#/ft^2 incl. dead load	0	0	0	+1/16	+1/16	+1/8
after 72 hours with above loading	+1/16	+1/8	+1/8	+1/8	+1/8	+1/4

Panel #4. Running Bond. This panel was loaded with three layers of cement bags. Deflections were measured after loading with one layer of cement bags, after loading with two layers of cement bags, after loading with three layers of cement bags and after 72 hours under the three layer loading.



Note: no cracks were observed in this panel after 72 hours under load.

LOADING	Deflections - inches A	B	C	D	E	F
1 bag $67\#/ft^2$ incl. dead load	0	0	0	0	0	0
2 bags $94\#/ft^2$ incl. dead load	0	0	0	0	0	0
3 bags $120\#/ft^2$ incl. dead load	+1/16	+1/16	+1/16	0	0	0
after 72 hrs. with above loading	+3/16	+3/16	+1/8	+1/16	+1/8	+1/8

Considering Panel 4, loaded at 120 psf, developed a resisting moment of 2,000 ft.-lbs. for a 16 inch width $\left(M = \frac{w}{8} = \frac{120(16/14)(10)^2}{8}\right) =$ 2000 ft-lb. This is the equivalent of 8 times the normal design wind load of 15 psf required to be resisted by this panel. It also far exceeded the allowable moment at the yield point of the wire of 870 ft.-lbs. without cracking and with negligible deflection after 72 hours. These tests show no apparent difference whether the bond is stacked or running. These high test loads must be attributed to the high compressive strengths of the mortar and the Tomax machine's techniques of joining the blocks with proper pressure and vibration so as to obtain excellent bond values [19].

4.4.4 TEST BY QUICKSPAN

Quickspan engineers carried out a test on a load bearing wall panel of Tomax type. The test was concerned with the lateral strength of the panel. The panel had 8" width x 8'-8" height x 9'-4" length. It was reinforced every two rows with amsonry ties of block-lok type. At the time of the test the age of the mortar was 28 days at ambient temperature inside the plant. The panel was placed in a horizontal position, simply supported at two points each 16" from the end of the panel as shown in Fig. 4.9.

Concrete masonry units were applied gradually as a uniformly distributed load till failure occured.

The first crack was noticed when a load of 115 psf was applied. Assuming a wind load of 20 psf, then the factor of safety would be:

$$f.s. = \frac{115}{20} = 5.75$$

the maximum load that caused failure was 135 psf which gives a factor of safety:

$$f.s. = \frac{135}{20} = 6.75$$

It can be concluded that the Tomax panel has a sufficient lateral strength to resist a wind load of 20 psf with a factor of safety equal to about 6 [20].

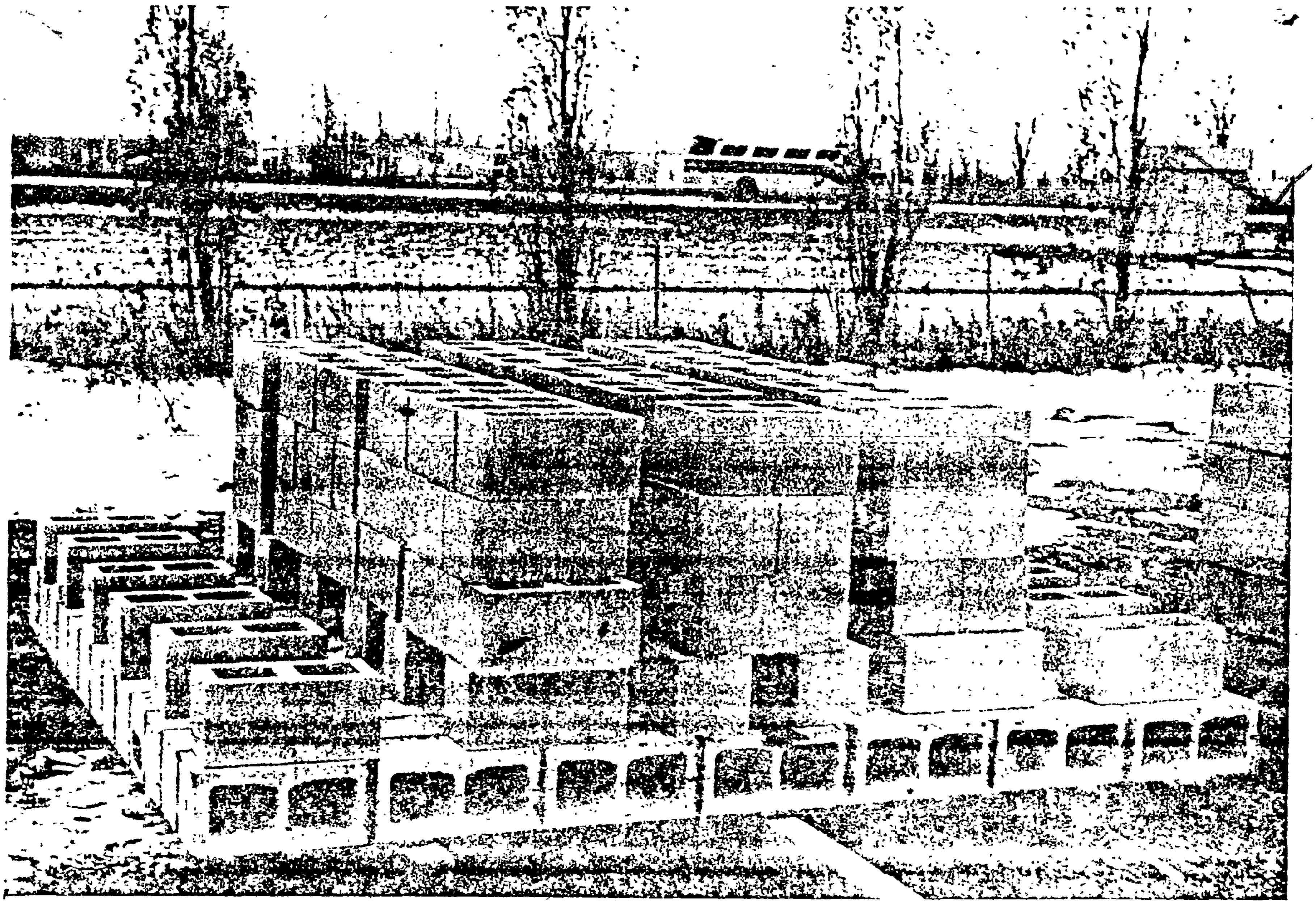


Fig. 4.9 — General arrangement of Quickspan panel strength test.

4.5 Panel Connections

A disadvantage of panelization is introduction of joints. A panelized structure would be equivalent to a monolithic one if the joints were of equal properties to those of the panels. This is not easy to achieve and consequently the structure is dependent on quality of its joints.

Joining the panels together attracted the greatest publicity since the accident at Roman Point. Roman Point was a 22-storey pre-cast concrete panel construction resting on an in-situ podium. A gas explosion, in a corner apartment of the 18th floor removed the external load bearing flank panel of the living room and bedroom. As a consequence the floor slab above colapsed and the flank walls and floors above followed, causing progressive collapse.

The classic gas explosion in Copenhagen in January 1969 showed that masonry wall, unlike precast concrete panel, does not tend to be blown out as an entity. Lateral forces tend merely to punch a hole in it [21].

During the many years that concrete masonry has been built, walls and floors have been joined with relatively simple connections. However, the development of prefabricated concrete masonry panels initiated the need for very strong connections between adjoining walls and floors.

On the following pages are details of different combinations of Tomax elements and of a number of connections that might be encountered in buildings using prefabricated concrete masonry panels. These details are offered only as a guide for design and are not meant to be complete or suitable for all buildings in all places [20, 17].

These connections are based on those used for conventional on-site masonry construction, and incorporate revisions for adaptation to panel construction.

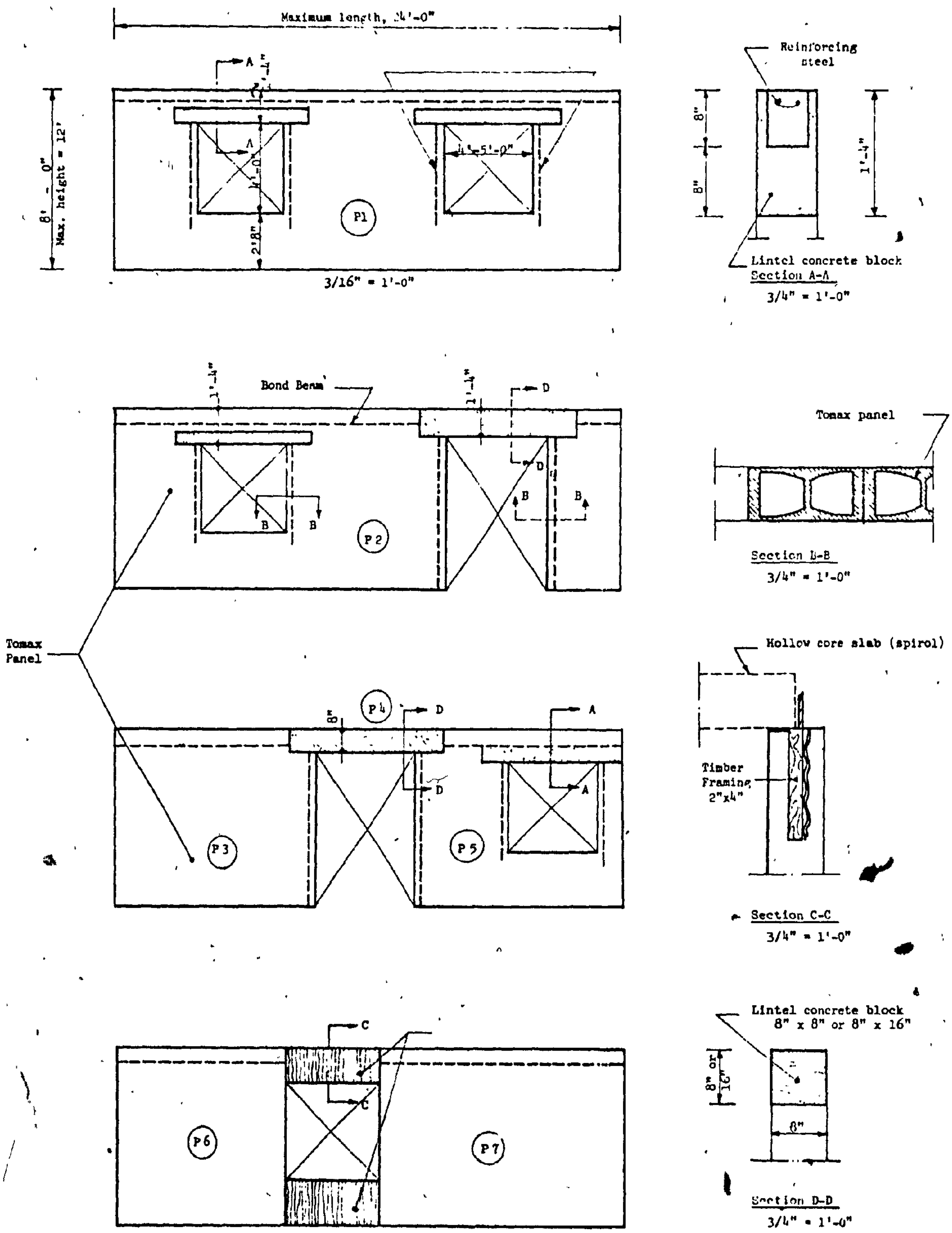


Fig. 4.10 — Different Combinations of Tomax Elements.

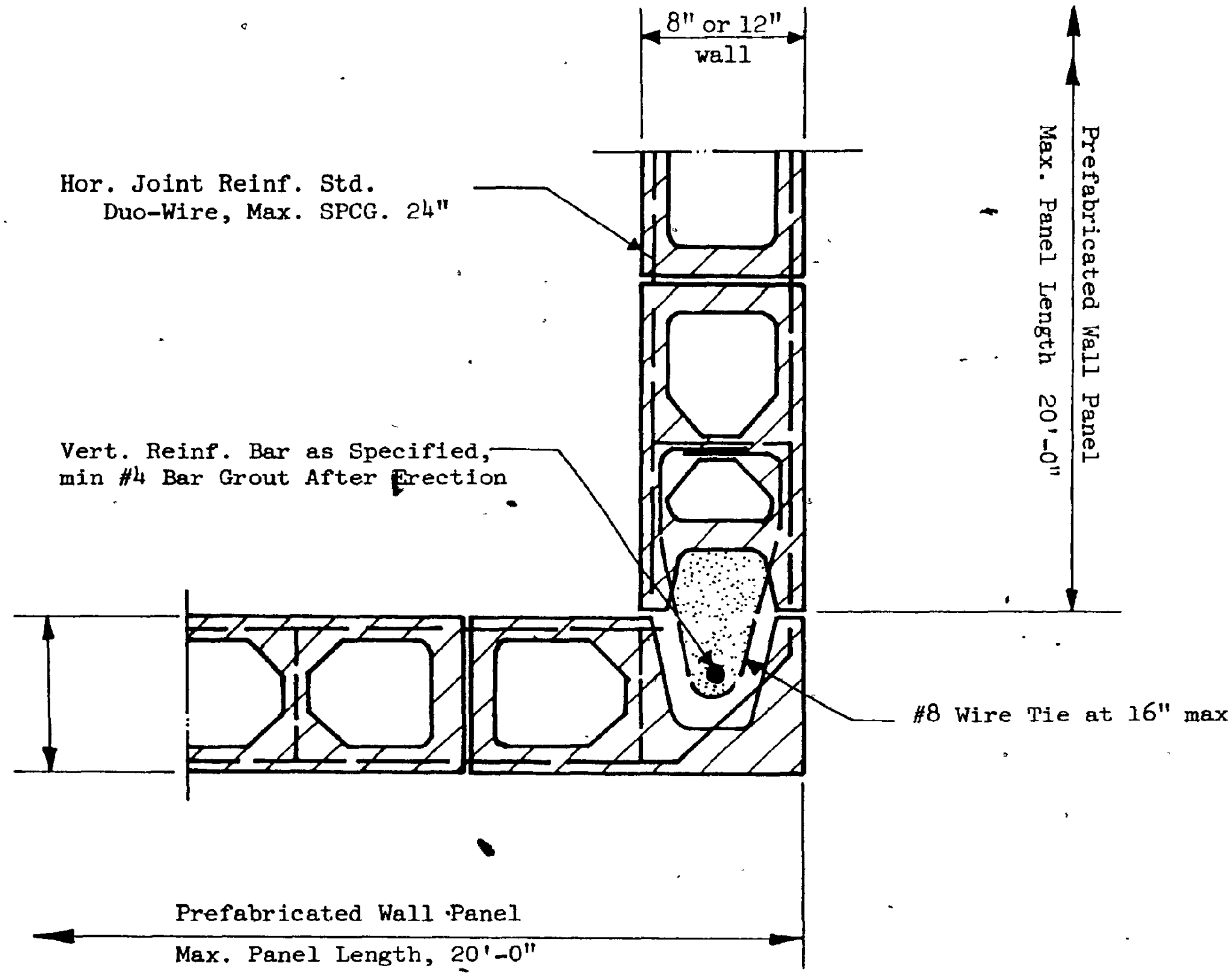


Fig. 4.11 — Plan of Corner Connection.

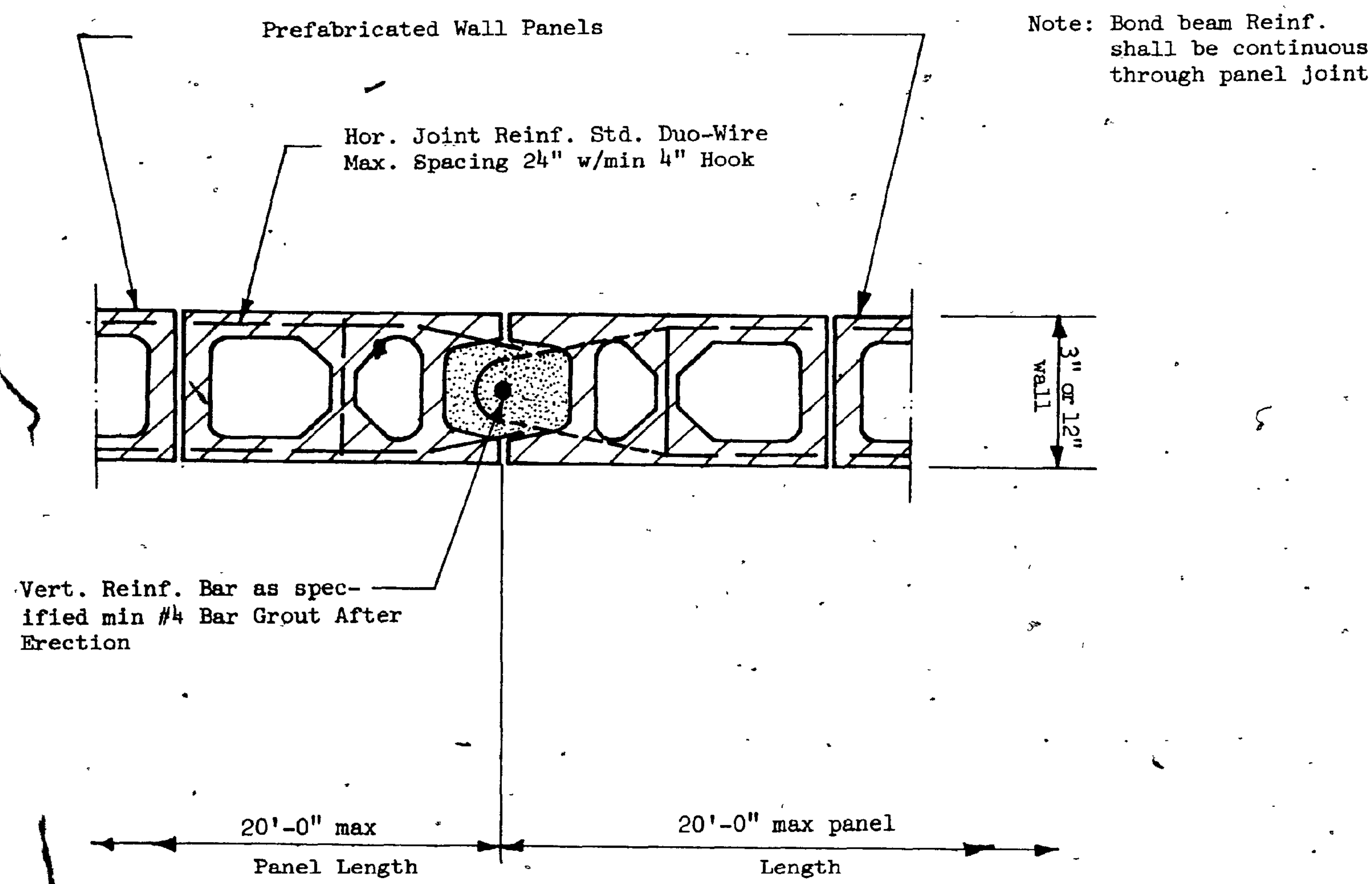


Fig. 4.12 — Plan of Typical Panel Connection.

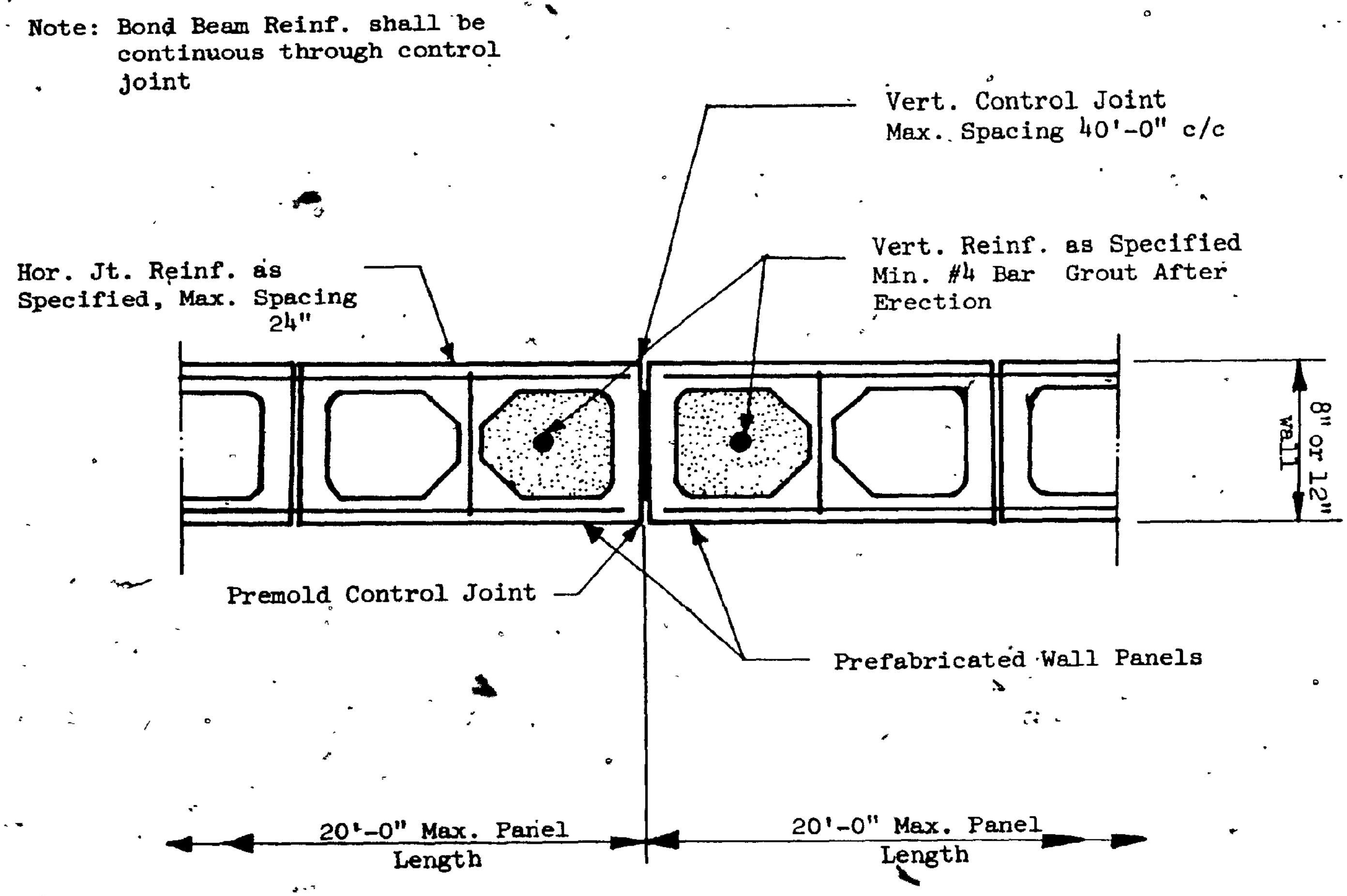


Fig. 4.13 — Plan of Typical Masonry Control Joint.

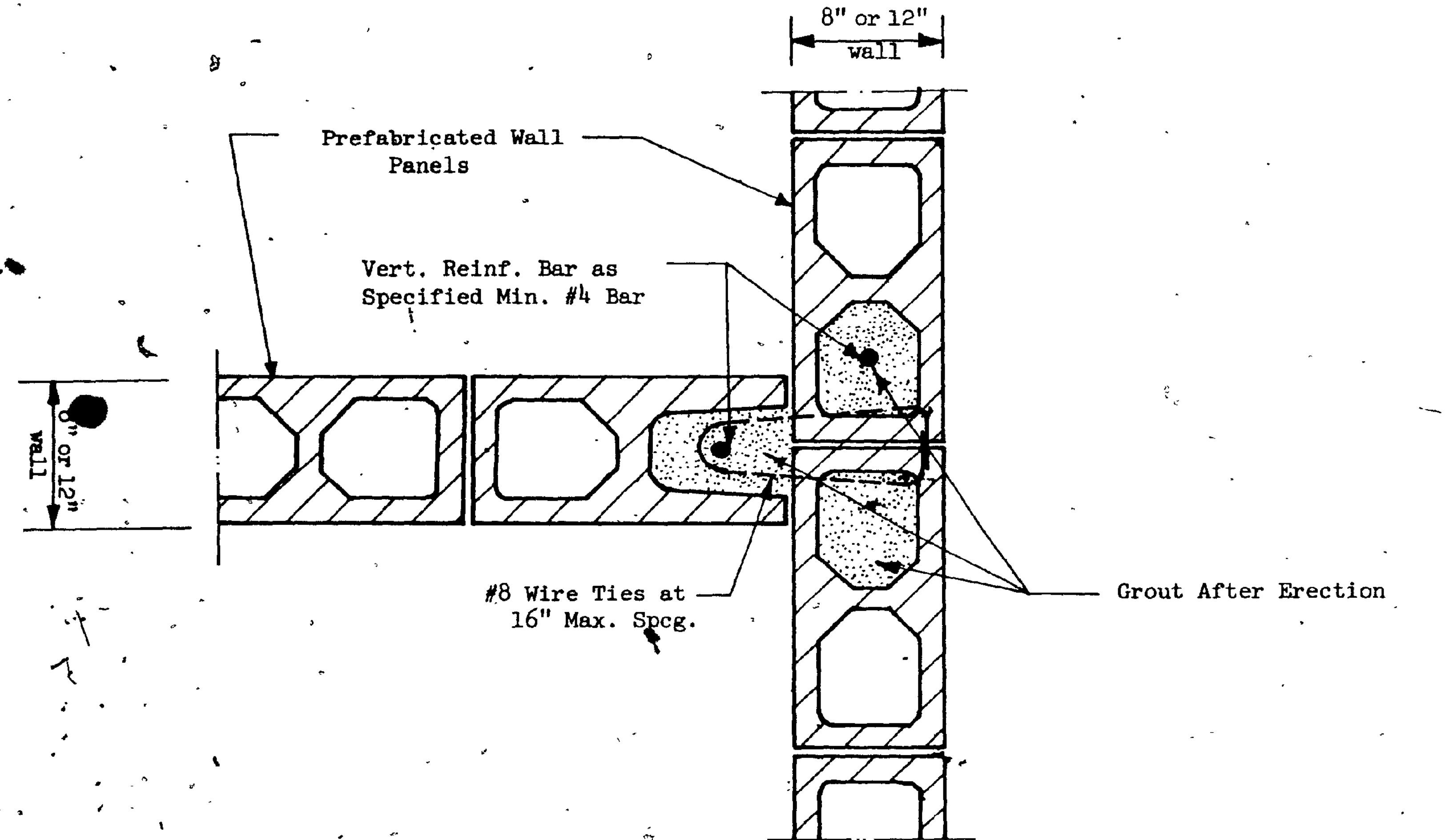


Fig. 4.14 — Plan of Typical Intersection Connection.

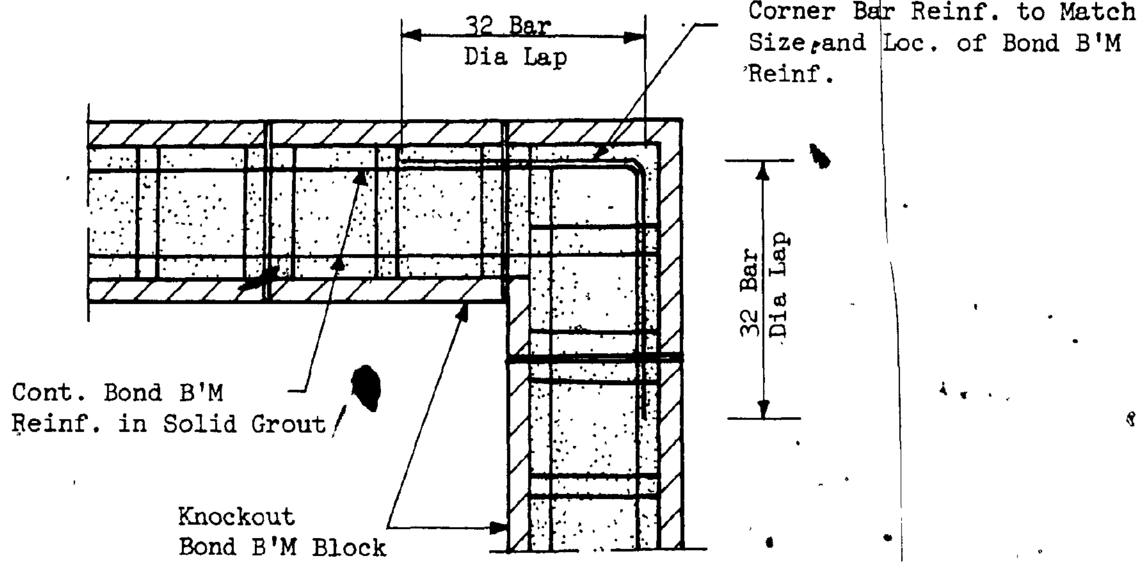


Fig. 4.14 — Typical Bond Beam Reinforcement at Corners.

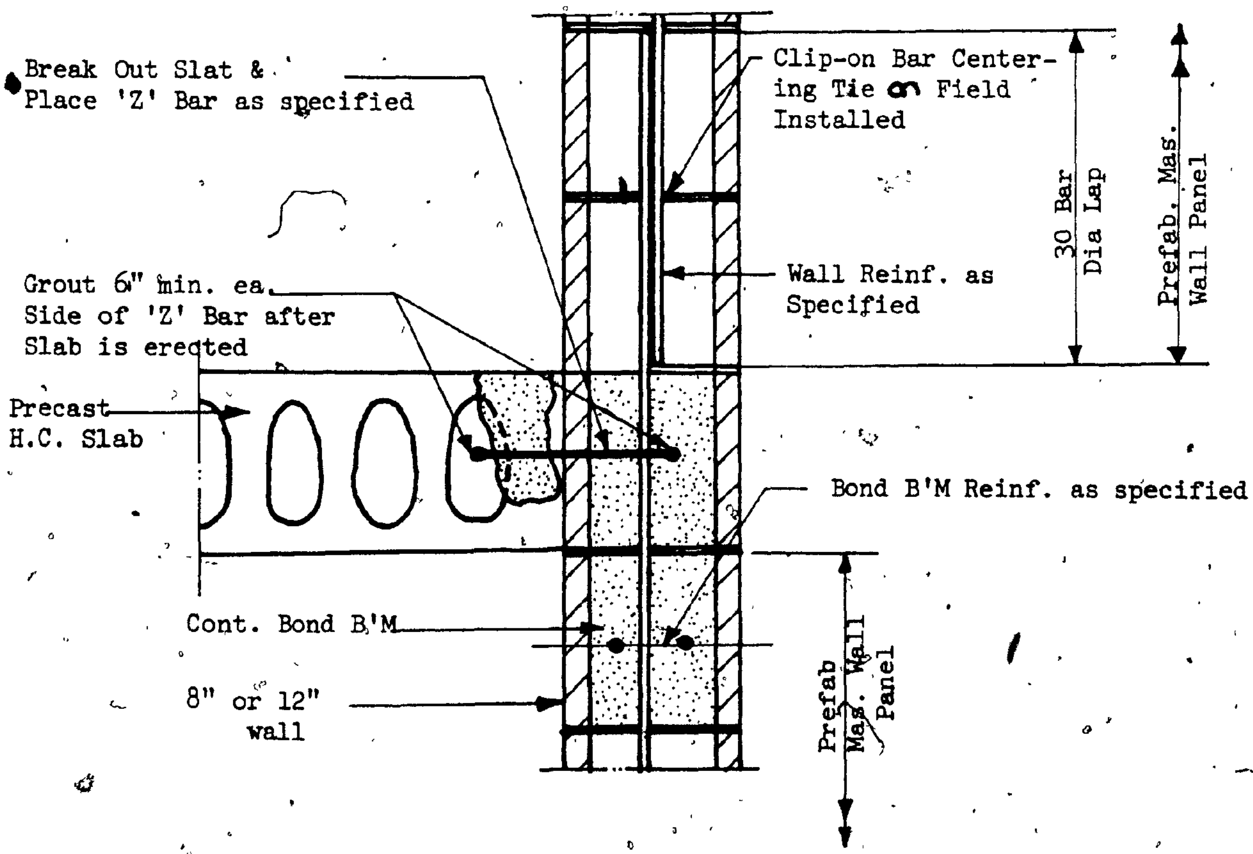


Fig. 4.15 — Typical Connection of Slab Parallel to Wall.

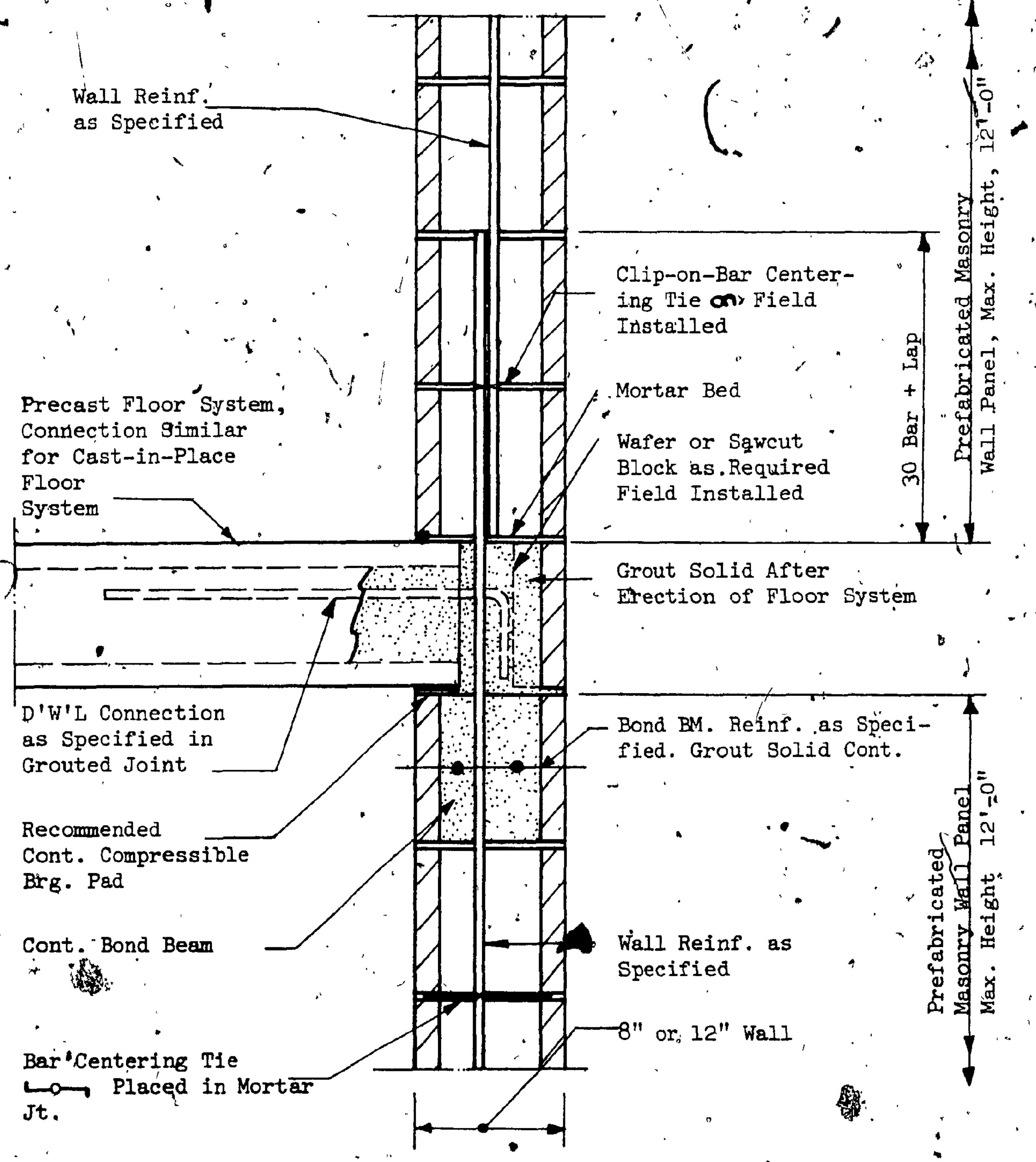


Fig. 4.16 — Typical Floor Connection to Exterior Wall.

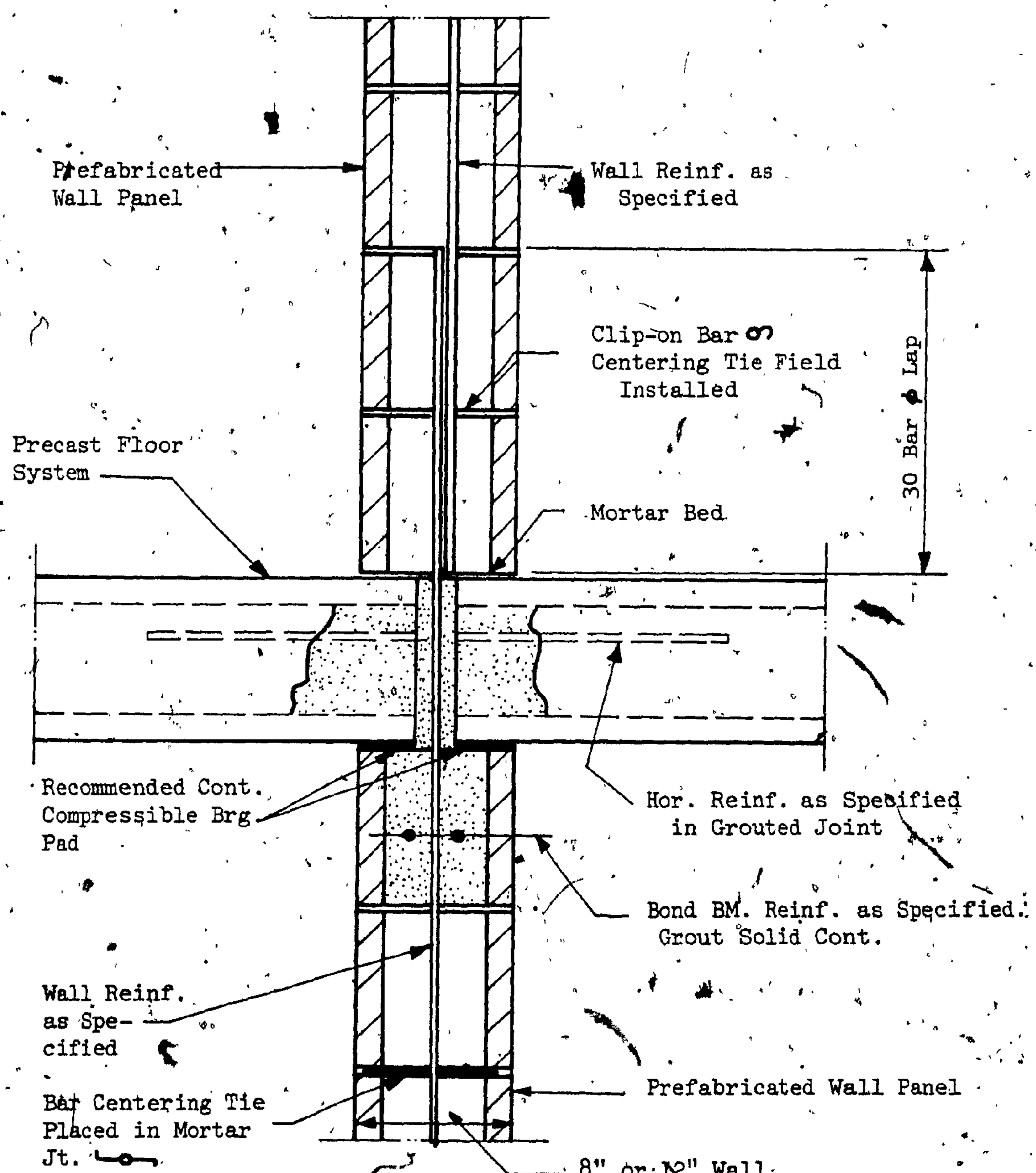


Fig. 4.17 — Typical Panel Connection at Floor.

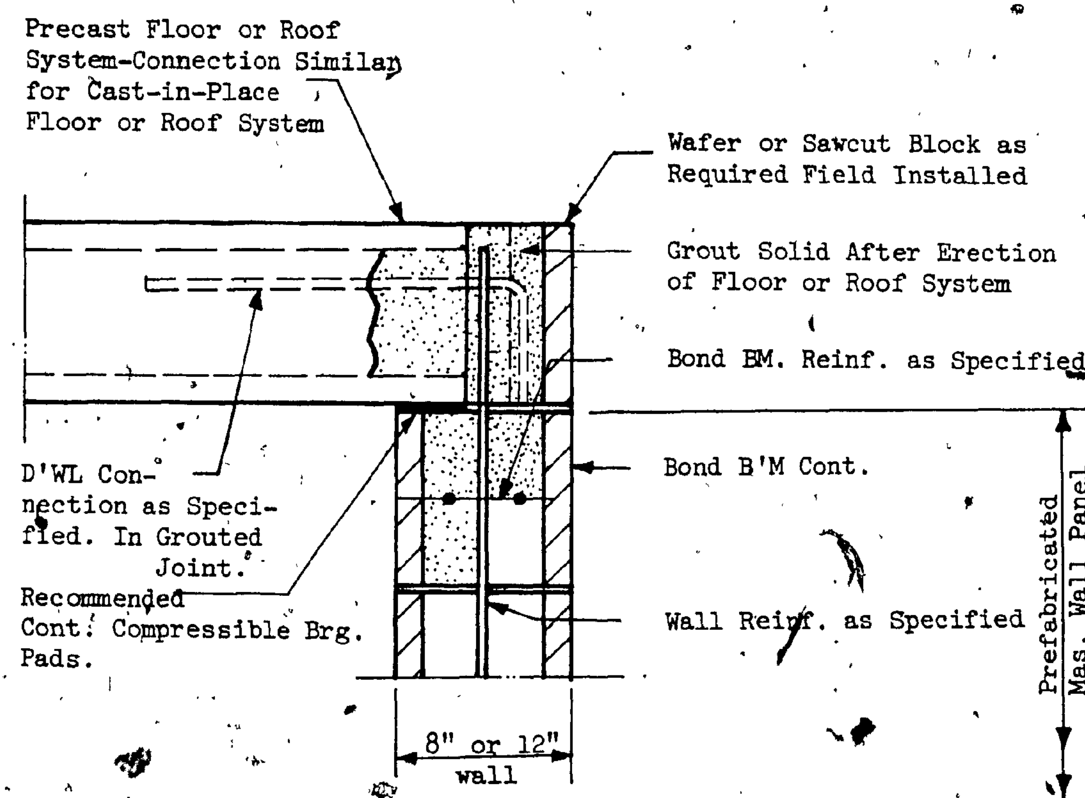


Fig. 4.18 — Typical Exterior Floor or Roof Connection.

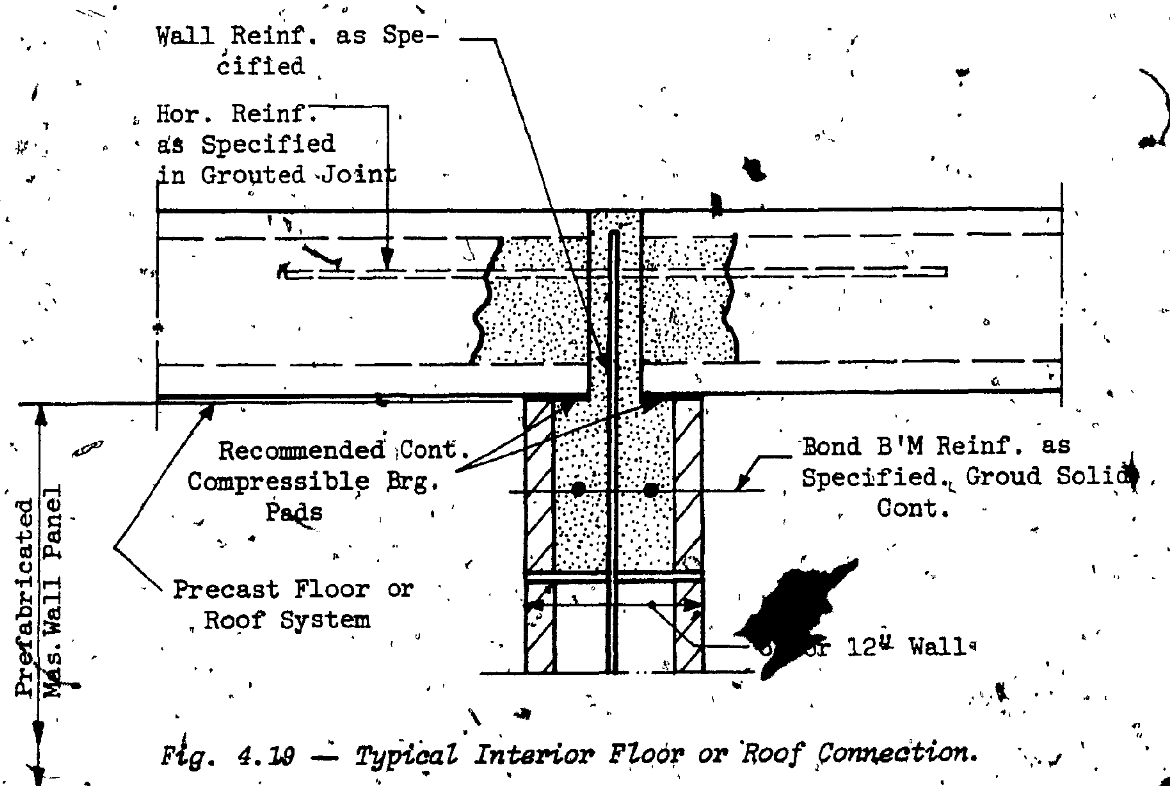


Fig. 4.19 — Typical Interior Floor or Roof Connection.

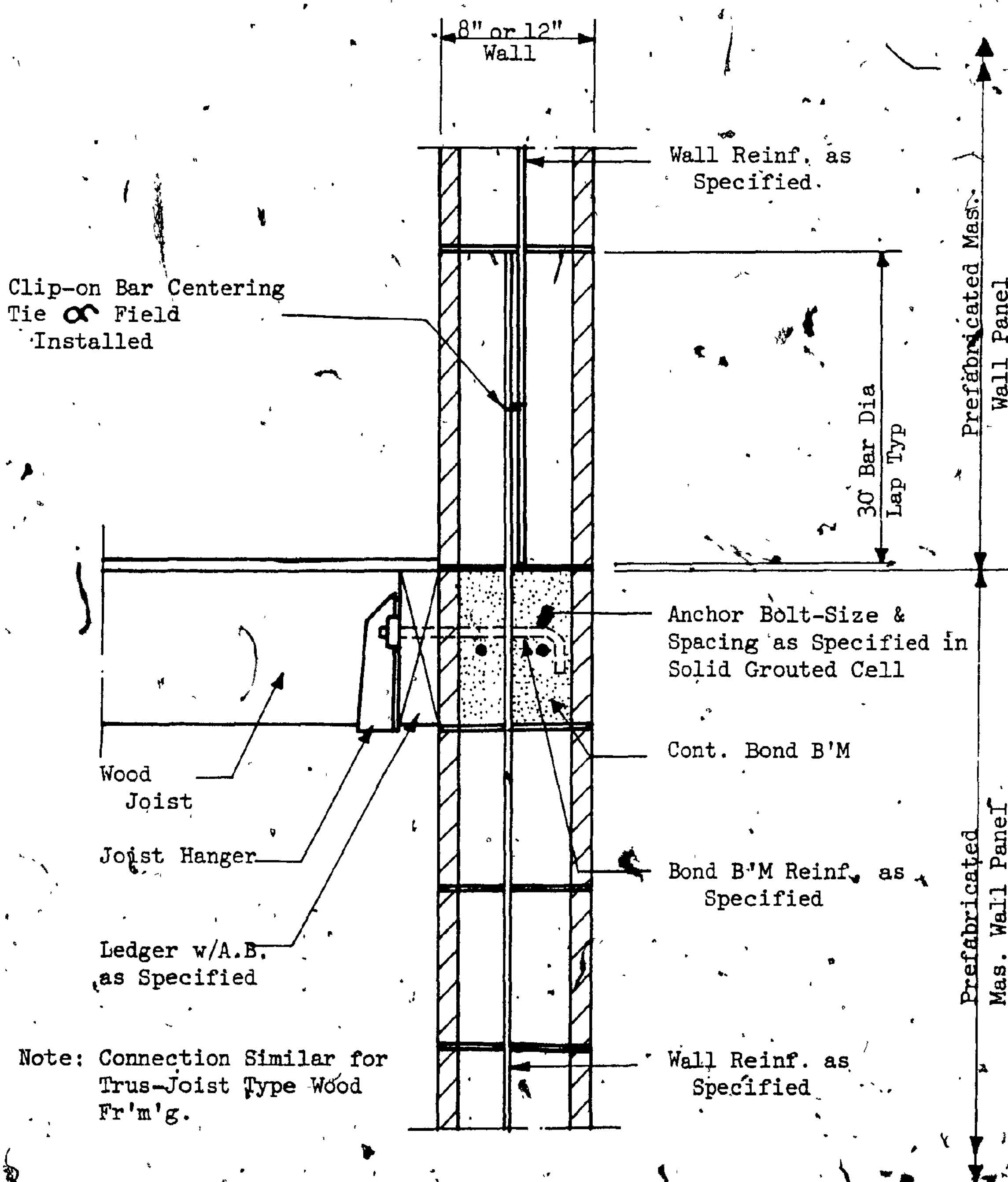
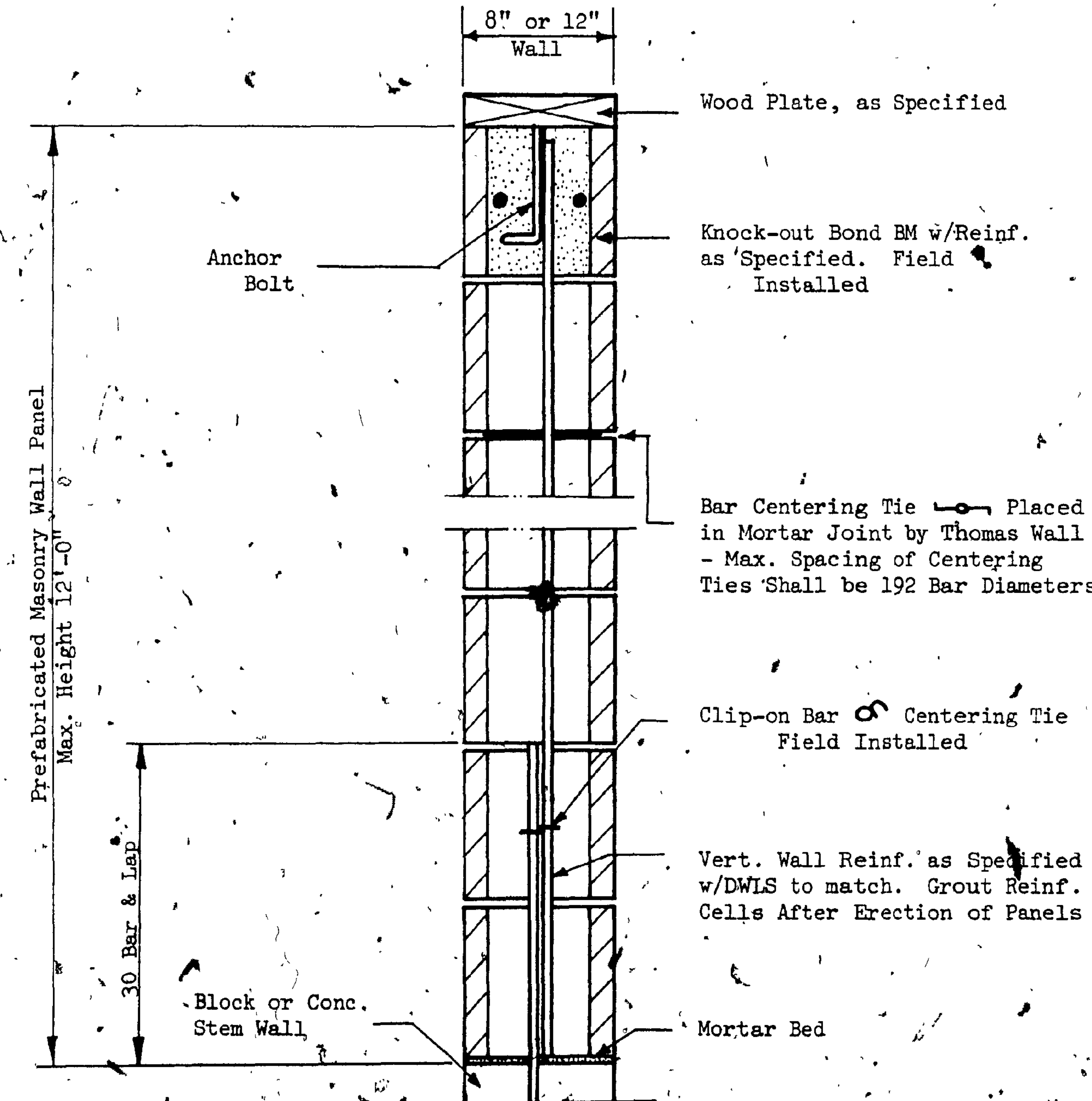


Fig. 4.20 — Typical Connection Detail Wood Floor Framing.



Note: All field grouting shall be done after erection of panels. Panels shall have temporary bracing until connected to floor or roof system.

Fig. A.21 — Typical Panel Section (exterior & interior walls - top bearings).

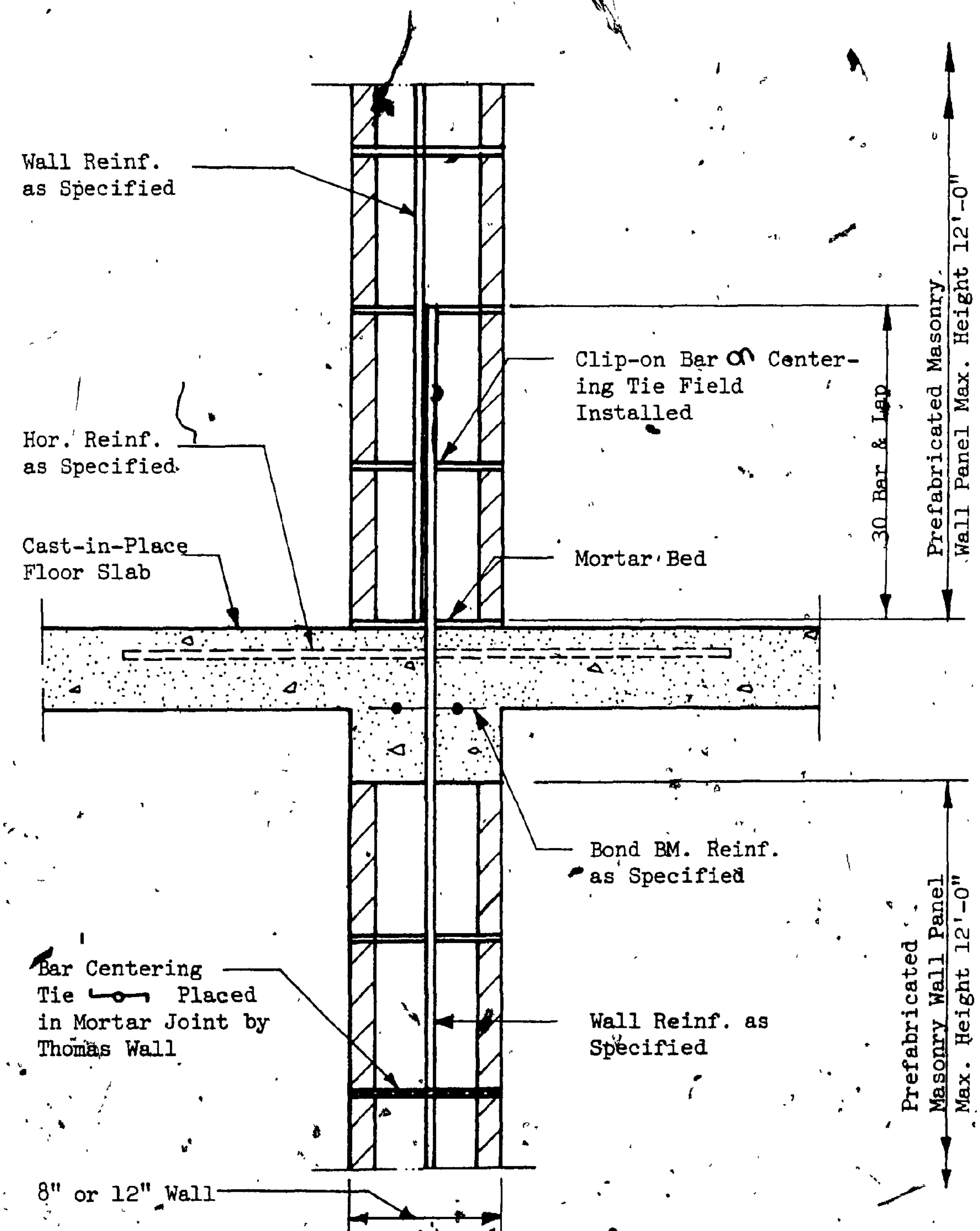


Fig. 4.22 — Typical Panel Connection at Floor (cast-in-place floor system).

4.6 TRANSPORTATION AND ERECTION OF TOMAX PANELS

Handling the finished panels of the plant does not require overly cautious methods due to the high tensile and high bond qualities of the mortar used in Tomax panels. When panels are completed, overhead crane or a conventional forklift can be used for yarding and loading the panels. The only requirement for handling finished panels with a forklift is that it be fitted only with a device which will hold the top of the panel from falling over as it is transported.

Tomax panels are transported from the fabrication plant to the job by conventional trucks with simple modifications or by especially designed low bed trailers. When using conventional trucks, two 3" pipes are installed vertically on the center line of the truck protruding a little above the panels, which act as a temporary support, to keep the individual panels from falling off while loading the truck. After loading, the panels are tied down according to standard practices.

At the building site lifting devices and provisions to prevent breakage are attached to the panels which are swung to approximate position in structure by a crane.

One drawback to any prefabricated system is the tremendous amount of labor, material and planning that must be devoted to support the system while incorporating it into the structure. This additional cost does not benefit the final structure. A method of installing concrete masonry panels was devised minimizing labour, material, and planning. The first approach to the problem of supporting the panels vertically while they are being aligned, plumbed and grouted, is steel or wood kickers, as used in any precast panel system. Vertical reinforcing which is grouted in the finished panel supports the panel while it is being installed. As a result, the wall is supported by its own bootstraps until it is incorporated into the finished structure.

The steel reinforcing, protruding from the foundation or the pier that the core of the panel is placed over; has a hood in the top end. A 3/8" pencil rod, with a hook in its bottom end, is put down in the same core from the top and the two hooks engaged. A steel holding bar 2" wide and 8" long with a 1/2" hole through it is slipped over the top of the 3/8" pencil rod with the rod projecting through 1/2" hole. The holding bar with the pencil rod through it is laid down on top of the panel. A pencil rod tightener or button tightener as used in reinforced concrete framework, is slipped down over the pencil rod and rests against the steel holding bar on the top of the panel. The pencil rod is then tightened, driving the steel holding bar on the top of the panel, on a set screw in the steel holding bar is tightened against the pencil rod. Therefore the stress built up in the reinforcing bar and pencil rod, and held by steel holding bars, holds the panel down tight to the foundation, thereby holding it up until it is grouted. After it is grouted the pencil rod and steel holding bar are removed and reused [17].

The monolithic like soundness of the panels assist erection of the units in numerous ways. Structural connection of one panel to another is easily accomplished by breaking through the end or the face of the panel in any desired location without fracturing or damaging the adjacent block and inserting reinforcing steel and grouting. The soundness of the panel also allows the panel to be field cut for possible special openings and special length of panel. This is readily done with a hand power saw and a carborundum blade with the panel either in its normal vertical position or by laying it down flat and sawing it as one would work on a section of plywood [17].

The installation procedure is repeated from floor to floor. If preassembled mechanical core units, plumbing trees, or preassembled kitchen or bath units are to be used, they are placed by crane before the floor above. Otherwise the erection continues with the mechanical and finishing work in a generally conventional manner. Exterior enclosure is keyed to follow close behind the structural work to ensure the building is closed as quickly as possible. This rapid procedure employs a minimum of field labor, which shortens construction time and allows for early occupancy.

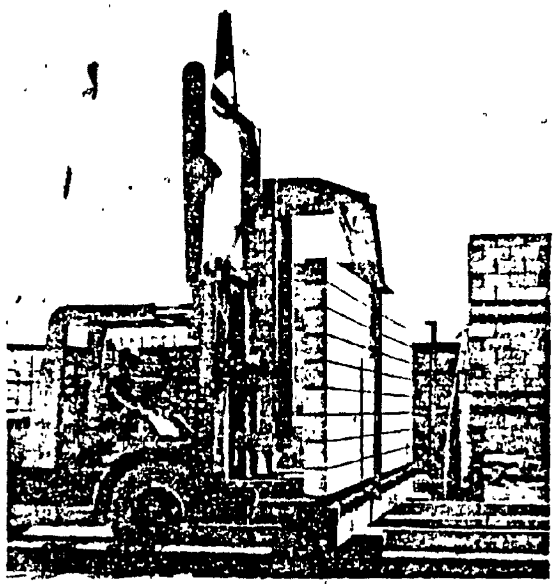


Fig. 4.23 — Tomax panels can be handled by conventional equipment [17].

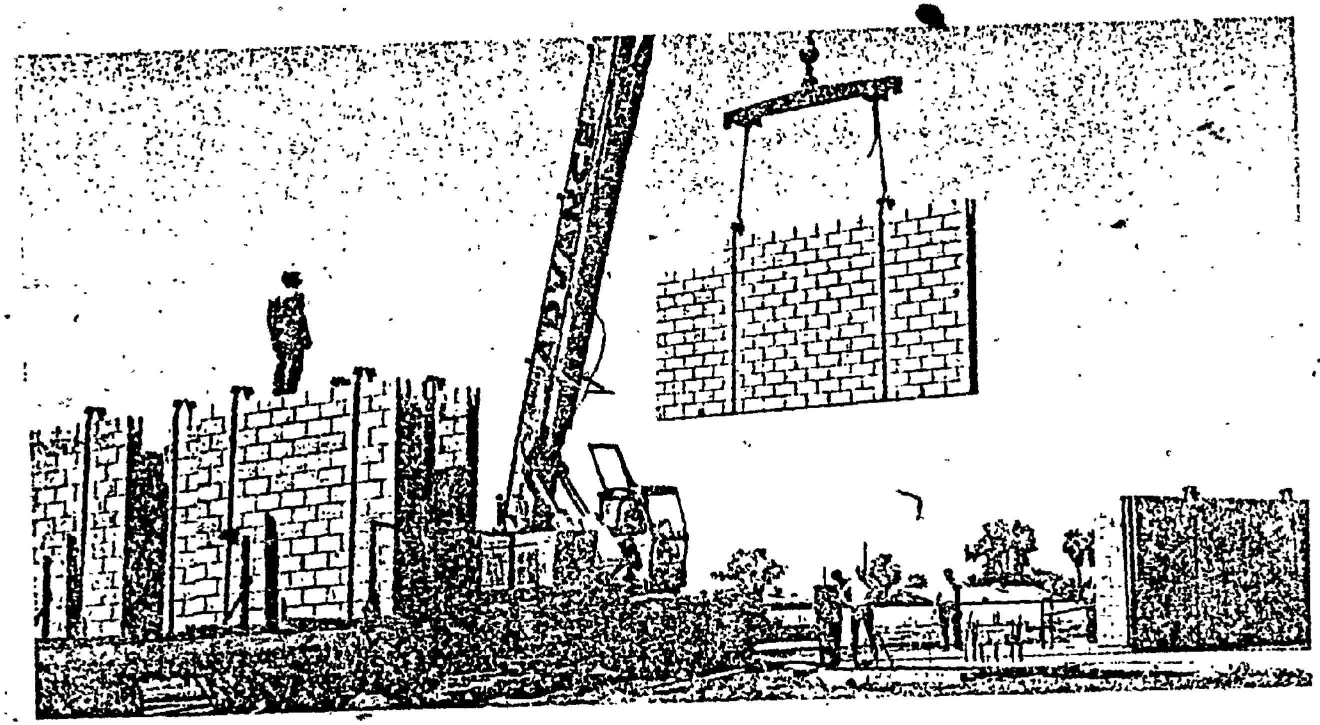


Fig. 4.24 — Tomax panels are transported, delivered, and installed easily and efficiently [17].

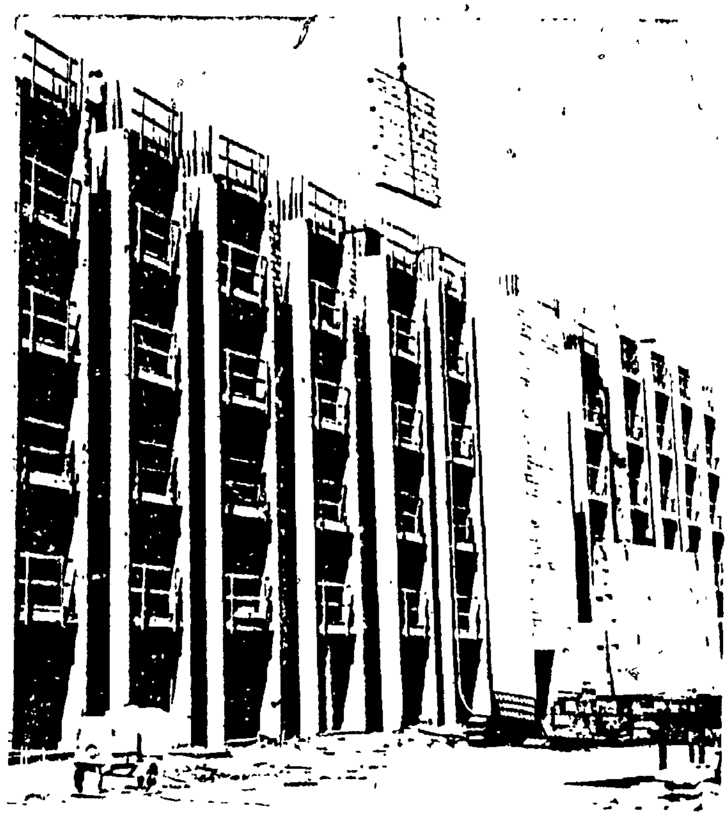


Fig. 4.25 — Tomax Panel is lifted to upper stories of an apartment project [17].

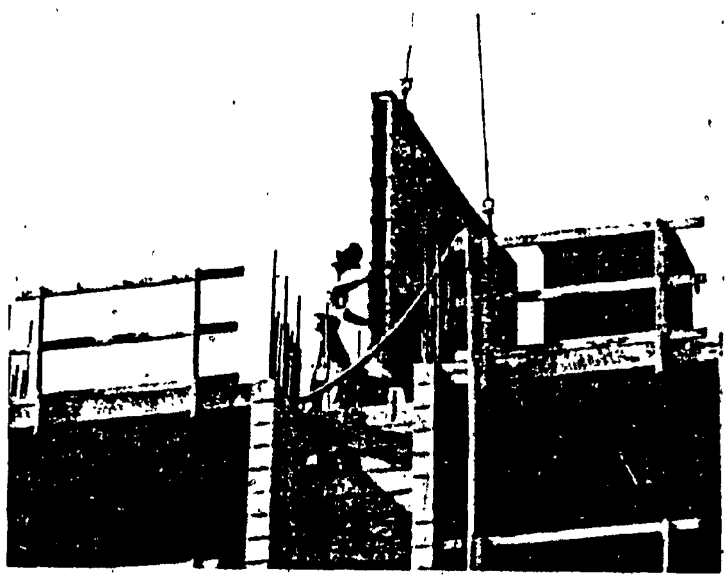


Fig. 4.26 — Tomax panel is gently lowered into place by crane [17].

4.7 APPLICATION OF TOMAX PANELS

Tomax wall panels were introduced to the market in North America at a time when:

1. Building owners, developers, architects and engineers were accustomed to using large scale construction units such as glass, metal, plywood and precast concrete panels.
2. Construction industry was suffering a setback mainly because of inflation.

For these reasons we find masonry in general and concrete masonry panels, in particular, are struggling to reach their rightful place. Today, Tomax wall panels are applied for construction of homes, industrial buildings, apartments, and commercial buildings. Erected buildings are located in desert country, subjected to temperatures 110°F, are also located 8000 feet above sea level in winter temperatures of 20°F below zero, and after more than five years there are no cracks in any of these panels [17].

4.7.1 Example 1 - Commercial Building in Ville d'Anjou

By using prefabricated block walls, Tomax panels, Quickspan, a Longueuil, Quebec, firm has been able to erect a seven-storey commercial building in Ville d'Anjou in only 18 working days [22].

Quickspan has brought the speed, economy and quality control by using Tomax prefabricated block walls and Spiroll prestressed extruded concrete slabs for floors. The job involved putting into place 56,000 square feet of 8-inch hollow core slabs and 26,000 square feet of block wall panel.

In using Tomax panels, speed is not the only advantage. Prefabrication of the block panels at Quickspan's plant virtually eliminates the problem of freezing mortar at the construction site. Consequently, the builder's heating costs are greatly reduced.

Prefabrication also means increased quality control, greater strength and reduced maintenance costs. Since mortar is spread on the blocks by machine, evenly and under carefully controlled conditions, and since the wall panels are mechanically vibrated to obtain optimum adhesion, the resulting joints are much stronger than in conventional masonry walls.

The wall is trucked to the construction site and then lifted into place by crane. Tomax panels resist the kind of cracking often seen in other kinds of block walls.

The use of both the Tomax walls and Spiroll floors gives the bulider two other advantages: cost control and reduced reliance on hard to find masons. It was estimated that 20 masons would normally be required to work on a building like the one erected in Ville d'Anjou. Instead, only about four or five were needed to put in place the non-bearing block elements such as longitudinal walls. Because of the need to increase the bearing strength of walls, after eight storeys, Quickspan buildings begin to lose their economic advantages over conventional concrete buildings.

According to Quickspan's experience, prefabricated structures can save $500 to $600 in labor and materials on each unit of an apartment building. In addition such buildings offer the prospect of reduced heating costs during cold weather construction as well as reduced interim financing costs and earlier generation of income, due to the speed of erection.

4.7.2 Example 2: Residential Building in Longueuil

Quickspan built in Longueuil a six-storey building for senior citizens in less than six months only at a cost of $2.4 million. It took Quickspan only 13 days to produce 95% of the elements needed for the project. The elements were 85,000 cubic feet of floor slabs, and 35,000 cubic feet of Tomax wall panels. The building consisted of 99 units, 11 units with two bedrooms and 88 units with one bedroom only [23].

According to Quickspan, using their system, about $3,000 can be saved on each unit, which is about $3,000,000 in this project.

La Societe d'Habitation du Quebec, SHQ, which was late in providing 7,000 living quarters for senior citizens, was impressed by the construction technique using prefabricated floor and wall panels by Quickspan. The SHQ hoped that this will enable them to provide the needed units in a short time and at the same time reducing the cost of the projects [23].

4.7.3 Description of Projects Accomplished by Quickspan

Quickspan has accomplished many projects in the Province of Quebec using Tomax wall panels and hollow core prestressed slabs (Spancrete and Spiroll). In all of them, quality and economy were achieved and their customers were satisfied. Fig. 4.27 and 4.28 are two of these projects. Other projects are described in the list on the following pages [20].

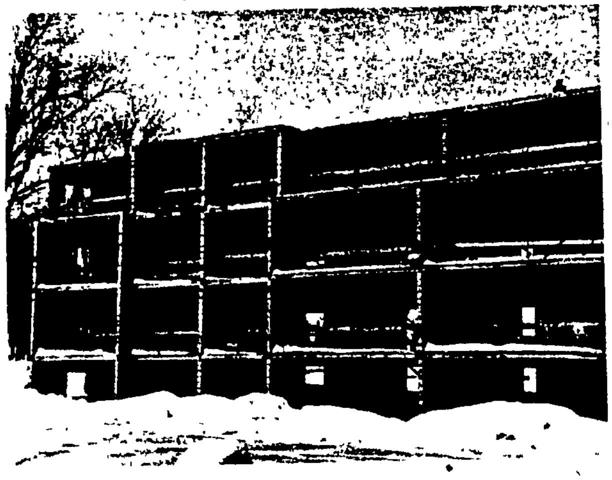


Fig. 4.27 — Apartment building, Nun's Island.

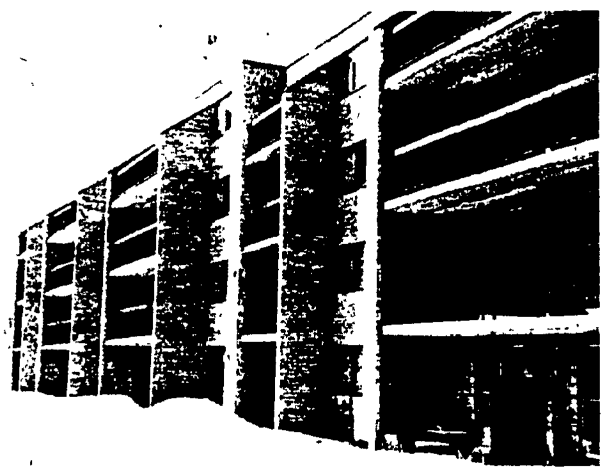


Fig. 4.28 — Residence for retired people, Laval.

CONTRACTOR	ADDRESS	ARCHITECT	STRUCTURAL ENGINEER	# of UNITS
SUMMIT IMPERIAL DEV. LTD. Edifice Appartements	rue Lafayette Longueuil, P.Q.	Maurice Bergman	Maurice Levis	90
J.B. CAYOUETTE LTEE. Motel St-Hyacinthe	route Trans-Canad. St. Hyacinthe	Cayouette, Saia & Leclerc	Eric Leblanc	
METROPOLITAN CONST. INC. Edifice Appartements	700 rue DeGaspe Ile des Soeurs	Tolchinsky & Goodz	Marc Denis & Associates	186
DOMAINE DES HAUT-BOIS INC. Edifices Appartements	Bl. des Haut-Bois Ste-Julie	Desmarais & Tornay	Regis Trudeau & Assoc.	52
DOMAINE DES HAUT-BOIS INC. Edifices Appartements	Bl. des Haut-Bois Ste-Julie	Cayouette & Saia	Horvath & Assoc.	52
ROGER GAGNE LTEE. Edifice Appartements	525 rue Bruges Longueuil	J. P. Bayard	Plante, Laurin & Assoc.	128
FAREL CONSTRUCTION INC. Edifice Appartements	605 rue Bruges Longueuil	J. P. Bayard	Plante, Laurin & Assoc.	128
GIOVANNI DILILLO CONST. Bungalo Duplexes	rue Charles-Faulkner rue Allard Montreal-Nord	Giovanni Dilillo Giovanni Dilillo	Marc Denis & Assoc. Marc Denis & Assoc.	32
A. PREVILLE & SONS INC. Anachemia Chemical Plant (usine produits chimiques)	135 rue Richer Ville St Pierre	Anachemia Chemi- cals	Marc Denis & Assoc.	
POULIN & MERCIER (1968) INC. Habitations à loyers Modiques	5905 Chemin Chambly St Hubert	Jean da Keresztes	Jean Horvath & Assoc.	32
CARTIER BUILDING INC. Edifice Appartements	Boulevard du Sou- venir, Laval, P.Q.	Par proprietaire	Marc Denis & Assoc.	198
CARTIER BUILDING INC. Edifice Appartements	Avenue Brookdale Cornwall	Par proprietaire	Ivan Varkay	99
COLOSSUS CONSTRUC. LTD. Edifice Appartements	rue Laflin Cornwall	Dominik, Thompson broise, Architectes	Malette, Lafram- & Ingenieurs	90
L.S.R. CONSTRUC. LTEE. Edifices Appartements	rues Beaubien & Boulogne, Longueuil	Jean-Guy Brodeur	Raymond Bolduc	36

BRANDON CONSTRUC. INC. Edifice Appartements	rue Cartier Pointe Claire	Angers & Perron	Dupuis, Morin, & R Routhier & Assoc.	140
DELTAIR CORPORATION Edifices Appartements	343 rue Querbes Vaudreuil	Desmarais & Tornay	G. Horvath & Assoc.	100
CHAURET & DUFOUR CONST. Habitat St-Sauveur	Chemin du Lac Millette, Mont Saint-Sauveur	Boudrias, Bourdeau & St Jean	St-Amant, Vezina, Vinet & Brassard	72
BOROIS LIMITEE Résidence pour rétraités	Chemin St-Francois Rigaud	George Marois	Plante, Laurin & Assoc.	64
CONST. EMERY PAQUETTE INC. Résidence Gabriel	Route 3 St Timothee	Boudrias, Boudreau & St Jean	Maurice D'Arcy & Assoc.	51
A.L.V. CONSTRUC. INC. Ed.Comm. & Res. Bât. A.	rue Beaubien Ville d'Anjou	Jean-Paul Breton	Joseph Karaguesian	48
STEPHEN SURA INC. Beaconsifled Garden Apts.	rue Elm Beaconsfield	Jacques Beique Jr.	Real Deschenes & Assoc.	82
DORILAS GRENIER LTEE. Les Habitations Bourg-Chemin	8105 Marie Victorin, Tracy	Boudrias, Boudreau St Jean	Maurice D'Arcy & Assoc.	90
MOTEL COLIBRI INC. (1967) Môtel Cilibri	Route 116 Ste Victoire d'Arthabaska, Victoriaville		Gerard Berlinguette	
SOCIETE d'HABITATION du QUEBEC, Résidence pour Personnes Agées	rue Boulogne Longueuil	Jean-Guy Brodeur	Marc Denis & Associes	99

TOTAL — 1,869

Table 4.3 — List of Projects Accomplished by Quickspan.

CHAPTER V

DESIGN OF PLAIN HOLLOW CONCRETE MASONRY WALLS

The design of a concrete masonry wall needs careful planning if the wall is to successfully serve its intended purpose. Prefabricated concrete masonry wall panels are designed by engineering analysis method in accordance with building codes and manuals used in designing conventional hollow concrete masonry walls.

Through its history masonry has depended on mass for structural performance. The design of masonry walls was a matter of the state of the art. This design appraoch is referred to as the "empirical" design method. It was generally based on simple rules concerning minimum thickness and maximum spacing of supports. These design rules were very conservative since they had to take care of such adverse factors as poor quality control, variations in materials, loads, and other factors affecting masonry strength. Though allowable stresses for axial compression were given in building codes, they were rarely used since the design was governed by the conservative minimum required thicknesses. Allowable stresses in shear and tension were not given because a general structural analysis design was not expected.

In the sixties, building codes recognized the high standard of quality control in modern concrete and that concrete masonry is susceptible to all conventional mathematical analysis. Specifications for the Design and Construction of Load-Bearing Concrete Masonry were approved. The section on masonry design in the National Building Code of Canada, 1965, resulted in the inauguration of engineered masonry in Canada.

5.1 LOAD BEARING WALLS

5.1.1 Engineered Concrete Masonry

By application of sound engineering principles and adequate quality assurance programs, concrete masonry walls can be designed using stresses much higher than those previously allowed with equal or greater safety. The allowances for greater masonry stresses permit structures to be built with thinner walls. Instead of requiring 12 to 16 in. thick walls of nonreinforced concrete masonry, the new codes allow the same load-bearing walls to be only 6 in. thick.

The design of engineered concrete masonry load-bearing wall buildings is based on the combined structural action of floor, bearing walls, and shear walls in resisting lateral forces. The floors transmit vertical dead load and live load to the bearing walls, and function as diaphragms to distribute the lateral loads to the walls. These lateral loads due to lateral forces such as wind or earthquake are resisted by shearwalls which carry them to the foundation, Fig. 5.1 [24].

Floor systems for use with load-bearing concrete masonry walls must perform without exceeding deflection which would cause excessive stress in any vertical element. Stiffness of a roof or floor diaphragm affects the distribution of lateral forces to the shear walls. Many types of the floor systems in use today such as prestressed hollow core, precast concrete solid slabs and cast-in-place floors, are designed so that they can satisfy the requirements.

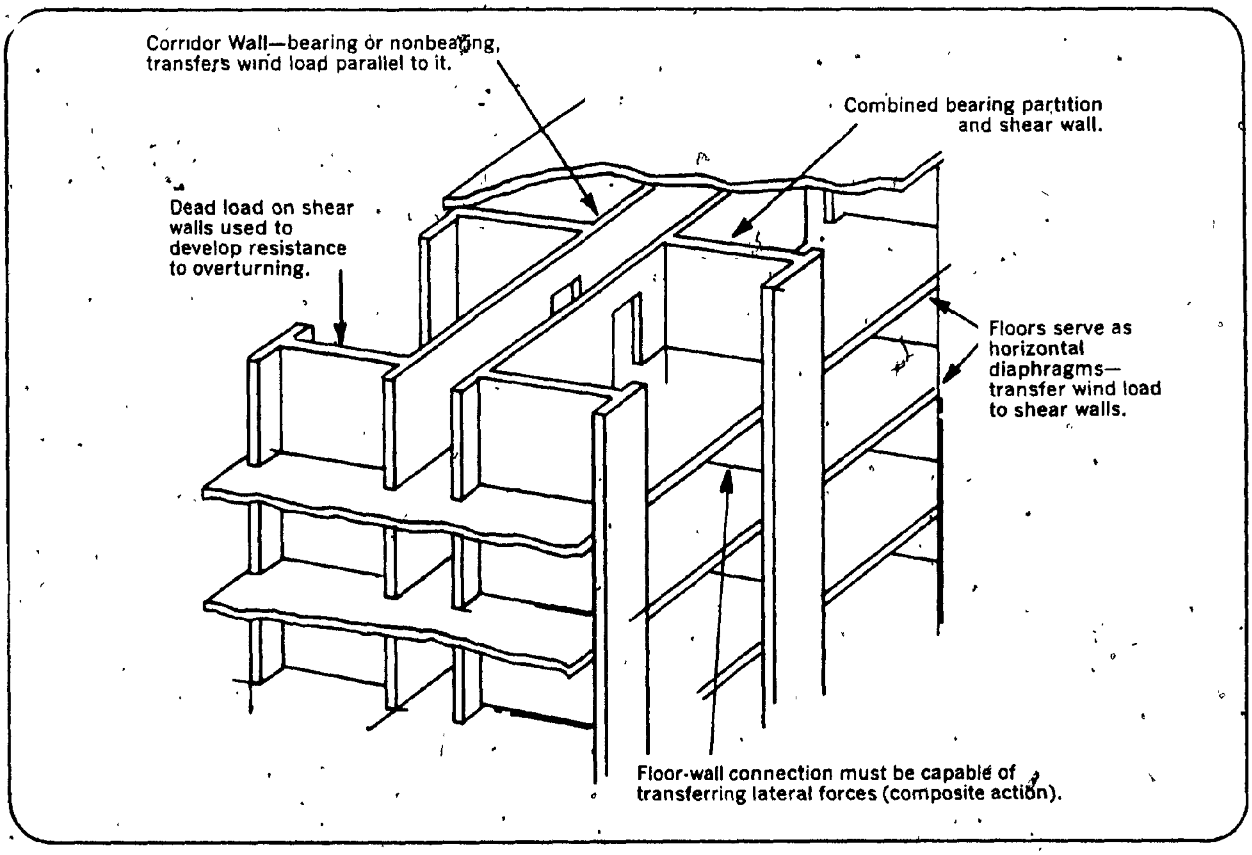


Fig. 5.1 — Engineered Concrete Masonry Load-Bearing Wall Building.

With engineered masonry design concept, it is possible to employ loadbearing concrete masonry in multistorey buildings that range in height from 3 to 18 stories. These buildings are designed and constructed either as reinforced concrete masonry or as nonreinforced, depending on building height, local code, and seismic conditions. The Canadian Building Codes and Specifications allow the use of plain (nonreinforced) concrete masonry in multistorey buildings in Zones 0 and which have little or no seismic activity.

5.1.2 Design Criteria

Design criteria for concrete masonry is contained in all building codes in the United States and Canada. Recently, the Canadian Standards Association published the first edition of CSA Standard S-304, Masonry Design and Construction for Buildings [25]. This new standard consists of four main clauses, namely:

"General Design and Construction Requirements".

"Design of Plain and Reinforced Masonry Based on Engineering Analysis".

"Empirical Rules for Plain Masonry Design Not Based on Engineering Analysis".

"Masonry Veneer".

In order to provide a sound basis for a specific concrete masonry structural design, based on engineering analysis, it is first necessary to determine f'_m (ultimate compressive strength of masonry) to be used in the design. According to CSA Standard S 304, f'_m may be determined by two methods: 1) Prism Method — on the basis of strength tests performed on prisms built from the materials to be used in the structure and take the value f'_m as determined by the tests, or 2) Unit Strength Method — an assumed value based on the compressive strength of the individual masonry units and mortar. The specified strength of the individual units is reduced by appropriate safety factors. Table 5.1 shows assumed compressive strength of concrete block masonry (f'_m) for various compressive strength of the units.

Allowable stresses determined by the prism test method are generally higher than those in Table 5.1. In addition, since the prism test incorporates the unit, the mortar, the grout, and the workmanship, it affords the designer the opportunity to know how the masonry can be expected to perform in practice. It gives him confidence to design with a more favorable safety factor than in the unit strength method.

Table 5.1 — Value of F'_m for Concrete Block Masonry or Structural Clay Tile Masonry (CSA Standard S 304-1977).

Compressive Strength of Units,**, psi	Ultimate Compressive Strength of Concrete Block Masonry or Structural Clay Tile Masonry (f'_m)*, psi	
	Types M & S Mortar	Type N Mortar
6,000 plus	2,400	1,250
4,000	2,000	1,250
2,500	1,550	1,050
2,000	1,350	950
1,500	1,150	800

* Compressive stress of hollow units is based on their net area.

** Linear interpolation is permitted.

5.1.3 Allowable Stresses

As with any wall, the stresses resulting from the various loads acting singularly or in combination are: 1) compression 2) tension, and 3) shear. All major codes provide criteria for obtaining allowable compressive, f_m, from the ultimate compressive stress, f_m. Table 5.2 shows maximum allowable stresses for plain concrete masonry.

Table 5.2 — Maximum Allowable Stresses and Moduli for Plain Concrete Block Masonry and Structural Clay Tile Masonry (CSA Std. S304)*

Type of Stress or Modulus	Designation	Maximum Allowable Stress or Modulus, psi: Units Without Voids of Filled Hollow Units Based on Gross Cross-Sectional Area	Maximum Allowable Stress or Modulus, psi: Units with Voids Based on Net Cross-Sectional Area
Compressive, axial			
Walls	f_m	0.20 f'_m	0.225 f'_m
Columns	f_m	0.18 f'_m	0.20 f'_m
Compressive, flexural			
Walls	f_m	0.30 f'_m	0.30 f'_m*
Columns	f_m	0.24 f'_m	0.24 f'_m*
Tensile, flexural			
Normal to bed joints			
M or S mortar	f_t	36	23*
N mortar	f_t	28	16*
Parallel to bed joints			
M or S mortar	f_t	72	46*
N mortar	f_t	56	32*
Shear			
M or S mortar	v_m	34	34*
N mortar	v_m	23	23*
Bearing on Masonry	f_b	0.25 f'_m	0.25 f'_m
Modulus of elasticity	E_m	1000 f'_m but not to exceed 3,000,000 psi	1000 f'_m but not to exceed 3,000,000 psi
Modulus of rigidity	E_v	400 f'_m but not to exceed 1,200,000 psi	400 f'_m but not to exceed 1,200,000 psi

*Shear and flexural calculations shall be based on net mortar bedded area.

Plain concrete masonry shear walls must be designed in such a manner that no part of the wall is in tension. According to CSA Standard S304, Sec. 4.7.3, the maximum horizontal shear stress in a shear wall, V_{sw} shall not exceed the value:

$$(V \text{ or } V_m) + 0.3\, f_{cs}$$

where -

V or V_m = the allowable applicable shear stress

f_{cs} = compressive stress due to dead loads.

5.1.4 Allowable Vertical Loads

While there are no changes in the CSA Standard S304 regarding allowable stresses, the criteria for the design of walls have been significantly changed mainly relative to the eccentricity and slenderness (CSA Standard S304, Sec. 4.6.7.1 and 4.6/7.2).

The allowable vertical load on a plain masonry wall is determined by the following formulae:

1) If the wall is subject to bending about one principal axis, and

a) $e \leq t/3$, where - e = the maximum virtual eccentricity

t = effective thickness of a wall.

Then, $P = C_e C_s f_m A_n$.

where – P = allowable vertical load

C_e = eccentricity coefficient

C_s = slenderness coefficient

f_m = allowable axial compressive stress

A_m = net cross-sectional area.

b) $e > t/3$, the allowable flexural tensile stress, f_t, normal to the bed joints, shall not be exceeded.

2) If the wall is subject to bending about both principal axis and

a) $e_t b + e_b t \leq \frac{bt}{3}$

where — b = width of a rectangular bar or width of flange of a T-beam

e_t = virtual eccentricity about the principal axis, which is normal to the effective thickness, t, of the member

e_b = virtual eccentricity about the principal axis which is normal to the width, b, of the member.

b) $(e_t b + e_b t) > \frac{bt}{3}$, then p shall be determined on the basis of a transformed section and linear stress distribution, and shall be modified by the slenderness coefficient, C_S. The maximum compressive stress in the masonry shall not exceed the allowable flexural compressive strength.

5.2 NON-LOADBEARING WALLS

Plain concrete masonry non-loadbearing walls are curtain and panel walls used to enclose a structural frame of concrete or steel, or inside partition walls. These walls are designed to resist lateral wind or seismic forces and transmit them to adjacent structural members. For such walls the reinforcement specified in the new standard should, in most cases, provide sufficient strength and ductility to prevent failure and minimize damage in the event of earthquakes. It is required that such walls shall be reinforced in one or more direction with reinforcing steel having a minimum area of 0.0005 Ag (where Ag is gross cross section area) in Seismic Zones 0, 1 and 2, and 0.001 Ag in Seismic Zone 3 at a maximum spacing of 16 inches (CSA Standard S304, Sec. 4.6.8.2.1).

The new standrad allows the use of "Empirical rules" for plain masonry design in buildings in Zones 0 and 1. These rules depend upon "rules of thumb which are known from experience to result in safe structures, generally having factors of safety, for higher than necessary. For load bearing structures of major importance, the "Empirical rules" would result in cumbersome and wasteful construction.

CHAPTER VI

DESIGN OF REINFORCED HOLLOW CONCRETE MASONRY LOAD-BEARING WALLS

Reinforced concrete masonry walls are used in some areas of high stress concentrations, or in areas of high winds or earthquake probabilities because steel provides for the excellent ductility and utilization of damping and energy absorption. Reinforced concrete masonry is constructed by embedding steel reinforcement in such a manner that the component materials act together in resisting forces. The concrete masonry hollow units are laid to form continuous, unobstructed vertical cavities. Required steel reinforcement is placed in these cavities which are then filled with grout to form a bonded composite construction in resisting compressive, tensile, and shearing stresses. This permits the use of higher design stresses and an increase in the distance between lateral supports.

6.1 Design Principles

The design of reinforced concrete masonry is similar in many aspects to its counterpart reinforced concrete. The design principles are similar to the elasticity assumptions used in early development of reinforced concrete. There is reluctance to accept the ultimate design principles, presently used in design or reinforced concrete and steel, in masonry because so far there is no adequate knowledge of the strength of masonry as well as of the adverse such as variation in building materials and workmanship. The assumptions of elastic design used in reinforced masonry are: [26].

1. Plane sections before bending remain plane and stress is directly proportional to strain.
2. The modulus of elasticity of the masonry, mortar and grout are constant within the member in the range of working stresses.
3. Stress in reinforcing is uniform over its area.
4. The member is straight and of uniform cross-section.
5. External forces are in equilibrium.
6. In reinforced masonry the masonry carries no tensile stress.
7. For bending, the span of the member is large compared to the depth.

6.2 Design Stresses

6.2.1 Effective Wall Thickness

It was pointed out in Table 6.1 that compressive strength of hollow unit masonry (f'_m) is based on net area. Since hollow concrete masonry units are of uniform consistent configuration, the net area used in design calculations is predictable and will vary only with the extent of grouting of vertical cells. Table 6.1 [27] shows these equivalents for walls of nominal 6-inch, 8-inch, and 12-inch block, and for various combinations of grouted cells.

Table 6.1 — Equivalent Wall Thicknesses.

WALL CONSTRUCTION		Wall Thickness 6"	8"	12"
		Equiv. Net Solid Thickness		
Solid Grouted Wall		5.6	7.6	11.6
Vertical Cores Grouted at	16" o.c.	4.5	5.8	8.5
	24" o.c.	4.1	5.2	7.5
	32" o.c.	3.9	4.9	7.0
	40" o.c.	3.8	4.7	6.7
	48" o.c.	3.7	4.6	6.5
No Grout in Wall		3.4	4.0	5.5

For example, the wall of 8-inch three core shown in Fig. 6.1(a) [27] with vertical reinforcement 32 inches o.c., has an equivalent solid (net area) thickness 4.9 inches. In flexural compression, the effective design section of a reinforced hollow unit concrete masonry wall is similar to a T-beam, Fig. 6.1 (b). Width of the compression flange, b, is assumed equal to six times the nominal wall thickness, t, when the wall is laid in running bond, and three times the thickness when laid in stack bond. The width of the T-beam stem, b', is equal to the width of the filled core plus the adjacent cross webs (normally 6 inches for a three-core block). Location of a T-beam neutral axis and computations then follow procedures standard for reinforced concrete [27].

Wall = 8 inches in thickness.

Vertical cores grouted at 32" o.c.

Equivalent net solid thickness, from Table 4.2 = 4.9 inches.

Fig. 6.1(a) — Area of Reinforced Hollow Concrete Masonry Wall Assumed Effective in Axial Compression.

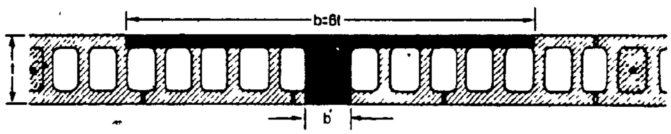


Fig. 6.1(b) — T-Beam Section Assumed in Flexural Compression (Masonry Laid in Running Bond).

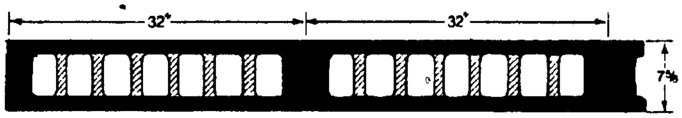


Fig. 6.1(c) — Area Assumed Effective in Longitudinal Shear.

Methods for determining masonry strength (F'_m) are the same as those described previously in Section 5.1.2, that is: F'_m may be based on prism tests, or on an assumed value, depending on strength of individual units as in Table 6.1. Table 6.2 shows the maximum allowable stresses (f_m) in reinforced concrete block.

6.2.3 Shear Stresses

Shear calculations in reinforced concrete masonry walls follow reinforced concrete design procedures and such reinforcement is provided to carry the entire shearing stress where the value of the calculated shear

Table 6.2 — Maximum Allowable Stresses in Reinforced Concrete Block and Structural Clay Tile Masonry (CSA Standard S304)

Type of Stress or Modulus	Designation	Maximum Allowable Stress or Modulus, psi
Compressive, axial		
Walls	f_m	$0.225\ f'_m$
Columns	f_m	$0.20\ f'_m$
Compressive, flexural		
Walls and beams	f_m	$0.33\ f'_m$
Columns	f_m	$0.28\ f'_m$
Shear		
No shear reinforcement		
Flexural members	v_m	$0.02\ f'_m$ but not to exceed 50
Shear walls	v_m	$0.015\ f'_m$ but not to exceed 75
With shear reinforcement taking entire shear		
Flexural members	v	$0.05\ f'_m$ but not to exceed 150
Shear walls	v	$0.04\ f'_m$ but not to exceed 75
Bond		
Plain bars	u	80
Deformed bars	u	160
Bearing on masonry	f_b	$0.25\ f'_m$
Modulus of elasticity	E_m	$1000\ f'_m$ but not to exceed 3,000,000 psi
Modulus of rigidity	E_v	$400\ f'_m$ but not to exceed 1,200,000 psi

stress exceeds that permitted on plain masonry (CSA Standard S304 - Sec. 4.8.3.1):

$$v = \frac{V}{bd}$$

where — v = shear stress

V = total shear

b = width of compression face of flexural member (for member of I- or T-sections b', width of web, shall be substituted for b).

Of special interest in the design of multistorey load-bearing structues, Fig. 6.1(c) shows the wall section assumed effective in longitudinal shear walls.

6.2.4. Tensile Stresses

Building codes do not allow concrete masonry to carry any tensile stresses. Tensile stresses are carried by the reinforcing steel and in accordance with CSA Standard S304, Sec. 4.5.2.1. The allowable tensile stress in reinforcement shall not exceed:

a) 18,000 psi for billet-steel or axle steel reinforcing bars of structural grade

b) 24,000 psi for deformed bars with a yield strength of at least 60,000 psi and not exceeding No. 11 size; and

c) 20,000 psi for all other reinforcement.

6.2.5 Allowable Vertical Loads

The allowable vertical load on a reinforced concrete masonry load-bearing wall is determined by the following formulae (CSA Standard S304, Sec. 4.6.7.2 and 4.6.7.3):

1) If the wall is subject to bending about one principle axis and

a) $e \leq t/3$, where e = the maximum virtual eccentricity

t = effective thickness of a wall

Then,

$$p = C_e C_s F_m A_m$$

where — p = allowable vertical load

C_e = eccentricity coefficient

C_s = slenderness coefficient

f_m = allowable axial compressive stress

A_n = net cross-sectional area.

b) $e > t/3$, or a value that would produce tension in the reinforcement, then p shall be determined on the basis of a transformed section and linear stress distribution and shall be modified by the slenderness coefficient, C_s. The maximum compressive stress in the masonry shall not exceed the allowable flexural compression given in Table 6.3.

2. If the wall is subject to bending about both principal axis, and

a) $e_t b + e_b t \leq \frac{bt}{3}$

where — b = width of a rectangular bar or width of flange of a T-beam

e_t = virtual eccentricity about the principal axis which is normal to the effective thickness, t, of the member

e_b = virtual eccentricity about the principal axis, which is normal to the width, b, of the member.

Then,

$$p = C_e C_s f_m A_n$$

b) $(e_t b + e_b t) > \frac{bt}{3}$, then p shall be determined by the same way as in 1(b).

6.2.6 Reinforcement

For load bearing and shear walls the reinforcing steel must be distributed horizontally and vertically with steel having a minimum area calculated in conformance with the following formulae (CSA Standard S304, Sec. 4.6.8.1):

$$A_v = 0.002\, A_g \alpha$$

$$A_h = 0.002\, A_g (1-\alpha)$$

where —

A_v = area of vertical steel per foot of wall, sq. in.

A_h = area of horizontal steel per foot of wall, sq. in.

A_g = gross, cross-section area, square inches

α = reinforcement distribution factor varying from 0.33 - 0.67 as determined by the designer.

The horizontal and vertical reinforcement shall be spaced not more than six times the wall thickness nor more than 48 inches apart, which ever is less. Horizontal reinforcement shall be provided at the bottom and the top of every wall opening and in the course immediately below the roof and floor levels (CSA Standard S304, Sec. 4.6-8.1.2 and 4.6-8.1.3).

CHAPTER VII

CONCLUSION

In order to provide for the growing needs of the future, to improve environmental standards, and to provide for improved living standards, a greatly increased volume of building will be needed. This will be possible only by increased productivity achieved through the use of industrialized building and generally increased efficiency.

Among the many building systems, the panelized system is the preferable system. It has considerably fewer design constraints and it provides maximum design flexibility. It is used almost throughout the world in numerous variations and it can lead to higher standardization, to better building design and performance.

The Tomax panel system is an impressive development that converts concrete masonry to a modern building material. It produces concrete masonry wall panels under factory controlled conditions and at an economical cost. These panels, due to the solid bonded head and bed joints have superior fire protection, better insulation and soundproofing, greater strength, and fewer maintenance problems. Test data indicates wall panel strengths up to five times greater than blocks laid by hand. Tomax panel connections are abased on those used for conventional on-site masonry construction with some modifications for adaptation to panel construction. Transportation and erection of panels can be accomplished by conventional methods.

A consideration of prefabricated buildings built with Tomax wall panels indicates that prefabrication is beyond the feasibility demonstration stage and is succeeding by offering higher strength walls and faster building enclosure with attendant savings. Tomax walls in conjunction with hollow prestressed floor and roof slabs give a superior structural shell with less coordination problems than other construction methods for modular buildings.

A concrete masonry wall has a heat sink capability and a thermal inertia capability that lightweight insulating materials do not have. It is an excellent thermal mass material for use in passive solar buildings. Also, when we consider the total concept from beginning of manufacture of concrete masonry units to construction and service of building to demolition of the building, the total energy consumed by a concrete masonry structure is far less than other materials.

The Tomax panel system did not revolutionize the concrete masonry industry and has not yet turned things around as dramatically as predicted, but the technique is still going on. The formulation of specifications for the Design and Construction of Load-Bearing Concrete Masonry brings an engineered concept to high-rise buildings, allowing taller and thinner load bearing concrete masonry structures. This concept and the advantages of Tomax panels make the future of these panels a bright one.

Tomax wall panels may not have all the advantages and are not the cure-all for all things; — there are some drawbacks. It is a heavy structure, therefore increasing foundation costs. Some concrete masonry walls should be waterproofed depending upon exposure and the type of masonry that is used. More research and better design criteria are needed. Design and engineering of concrete masonry must be introduced into the architectural and engineering colleges and universities in order to make architects and engineers familiar with the characteristics of masonry performance and the quality of materials which are being used, and to help in fulfilling the needs and knowledge which is demanded in today's technology.

Little has developed in the new CSA Standard S304-1977, that is relative to prefabricated concrete masonry panels, which began to emerge in Canada in the present decade. There is little doubt that with the growth of this industrialized medium, much work will be required to expand and improve the related provisions.

REFERENCES

1. HUGGINS, M.W. "Keynote Address", published in "Proceedings Symposium Panelized Structure Assemblies", 11-12 May, 1972, Sir George Williams University, Montreal, pp. 1-9.

2. Estimating the Fire Resistance of Concrete Masonry, National Concrete Masonry Association, Maclean, Virginia, 1966.

3. Fire Performance Ratings 1975, Supplement No. 2, of the National Building Code of Canada, NRCC No. 13987, National Research Council, Ottawa, Canada 1975, pp. 7-10.

4. Fire Safe Apartment Construction with Concrete Masonry, National Concrete Masonry Association, Mclean, Virginia, 1976.

5. Fire Safe Concrete Masonry Construction, National Concrete Masonry Association, Maclean, Virginia, 1972.

6. Noise Control with Concrete Masonry, National Concrete Masonry Association, Va., 1972.

7. New Data on Sound Reduction with Concrete Masonry Walls, National Concrete Masonry Association, Mclean, Virginia, 1978.

8. National Building Code of Canada, Ottawa, 1977, sec. 9.11, p. 255.

9. Measurements for Energy Conservation in New Buildings, 1978; NRCC No. 16574, National Research Council of Canada, Ottawa, p. 5.

10. Tables of U-values for Concrete Masonry Walls, National Concrete Masonry Association, Mclean, Virginia, 1975.

11. Energy Conservation with Concrete Masonry, National Concrete Masonry Association, Mclean, Virginia, 1974.

12. New Insights into Energy Use and Conservation in Structures, National Concrete Masonry Association, Mclean, Virginia, 1975.

13. Design of Solar Energy Walls with Concrete Masonry, National Concrete Masonry Association, Va., 1978.

14. Henry Toennies, "New Mortars, Prefab Walls Transform Concrete Masonry". Published in Civil Engineering, ASCE, New York, July 1973, pp. 46-49.

15. Henry Toennies, "New Techniques for Concrete Block Walls", published in Building Research and Practice, London, May/June 1974, pp. 139-143.

16. Builders Equipment Company, Phoenix, Arizona, Technical Literature.

17. Thomas Masonry System, Phoenix, Arizona, Technical Literature.

18. Anonymous, "Prefabricated Block Wall Panels", published in Concrete, London, January, 1974, pp. 28-29.

19. Best's Blocks Inc., Union City, California, Technical Literature.

20. Quickspan Inc., Longueuil, Quebec, Technical Literature.

21. H.W.H. West and H. R. Hodgkinson, "The Resistance of Brick to Lateral Forces", published in Proceedings First Canadian Masonry Symposium, University of Calgary, Calgary, Canada, June 7-10, 1976, pp. 274-283.

22. John Gormley, "With Préfab Block Wall a 7-Storey Building Goes in Only 18 Days", published in Engineering and Contract Record, Don Mills, Ontario, Canada, November, 1977, p. 65.

23. Gilles Pillon, On constuira 99 logements pour perssones agées en moins de six mois grâce à la préfabrication lourde", published in Le Courrier Du Sud, South Shore, Quebec, March 1978, p. C11.

24. Load-Bearing Block in High-Rise Buildings, National Concrete Masonry Association, Mclean, Virginia, 1972.

25. CSA Standard S304-1977, Masonry Design and Construction for Buildings, Canadian Standards Association, Rexdale, Ontario, Canada.

26. Walter L. Dickey, "Concrete Masonry Construction", Handbook of Concrete Engineering, ed. Mark Fintel, Van Nostrand Reinhold Company, New York, N.Y., 1974, pp. 536-571.

27. Reinforced Concrete Masonry - An Established Structural System, National Concrete Masonry Association, Mclean, Virginia, 1968.