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PRECAST-PRESTRESSED BUILDING
SYSTEMS AND ELEMENTS

by

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A MASTER'S REPORT

submitted in partial fulfillment of the
requirements for the degree

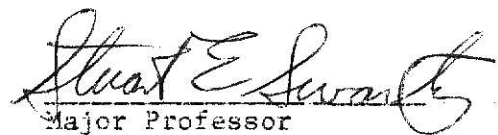
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CHAPTER I - BACKGROUND

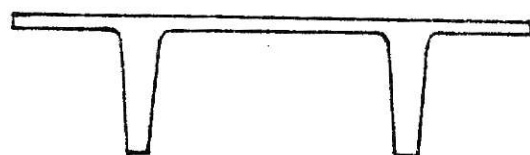
The Precast industry probably started in the United States in 1950, with one plant producing precast-prestressed products. In 1954, there were 34 plants, and a survey by the Prestressed Concrete Institute indicated at least 229 plants were operating in 1961 (19). Now, precast plants are operating in virtually every geographic area of the United States, making the accessible use of precast products economical for many structures.

Starting in the early sixties, building and construction costs have escalated at a considerably faster rate than those of most industrialized products. One of the main reasons for these high building costs is the large amount of site labor involved in the traditional construction processes. The demand for skilled, on-site building labor is quickly out-running the supply and will increasingly do so in most industrialized and developed nations. This trend can be slowed or halted only through greater utilization of centralized plant production, or greater industrialization of construction; and the maximum replacement of expensive mobile skilled construction labor, by centralized local unskilled labor and a few skilled workers.

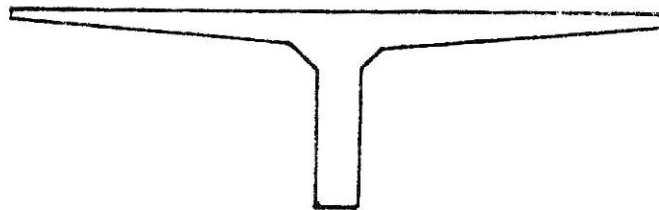
Precast-concrete construction has answered this need for industrialization by mass producing repetitive and standardized units which include beams, columns, floor and roof elements, and wall panels. See Figure 1 for typical standardized precast sections. These elements are produced in precasting yards under factory conditions with closer tolerances and higher quality than can be achieved with cast-in-place concrete. Precast concrete construction is used in all major types of structures; industrial buildings, office buildings, both low rise and high rise type, residential applications and also bridges. Precast concrete utilizes both

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CONTAINS
NUMEROUS PAGES
WITH DIAGRAMS
THAT ARE CROOKED
COMPARED TO THE
REST OF THE
INFORMATION ON
THE PAGE.**

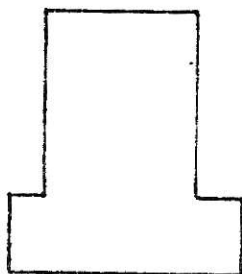
**THIS IS AS
RECEIVED FROM
CUSTOMER.**



Double Tee



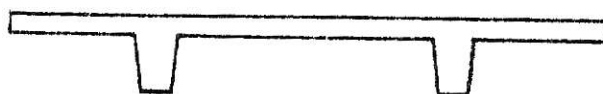
Single Tee



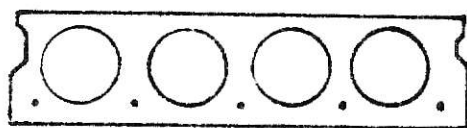
Inverted Tee



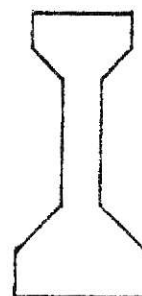
"L" Beam



Double Tee Wall Panel



Hollow Core Slab



AASHTO Beam

Figure 1: Standard Elements

mild steel reinforcement and prestressed reinforcement.

Advantages of precast concrete over cast-in-place concrete include the savings in using unskilled local on-site labor versus skilled mobile construction workers; shorter construction time because elements can be cast while site preparation and foundation work are underway; better quality control and high concrete strengths under the factory conditions, and also less dependence on weather seasons for construction. Member selection is also simplified by using the producers load tables and charts, or using the tables in reference (12).

Disadvantages include the greater cost of transporting materials (members), and the additional design problems associated with site connections of the precast elements.

With precast concrete being available to almost every part of the United States, the producers usually fall into two categories. First, there is the specialized producer, dealing with large individual custom jobs, where special shapes are developed for particular projects. The number of repetitive units must be sufficiently high and are usually designed with architectural considerations. These types of units are particularly important for exterior wall panels and permit a wide variety of architectural treatments. After the particular project is finished, the elements developed may never be integrated into another building system. The second type of producer is the one that mainly casts standardized elements that have been developed by the industry or user. These elements include wall panels, roof and floor elements; flat slabs, hollow core planks, double tees, single tees, and also precast columns. See Figure 1 for examples of the standardized sections. These are two separate and distinct operations and are basically incompatible to produce from a single source.

CHAPTER 2 - BUILDING SYSTEMS

In order to standardize the building industry, precast manufacturers must use the modern methods of industrialization, volume production, line assembly, centralized plant, and quality control (2). Building systems, if they are to have broad application and usefulness, need to be innovative, elegantly conceived, simple and above all, practical. Standardized precast components are efficient and directly applicable to a variety of "erector set" building shell systems, that can be made readily available on a large scale. Many types of systems are available today, some using the standard precast units, others developed by producers to fit a variety of specific needs.

Systems Using Standardized Elements

Using the standardized elements listed above, many applications are sure to come to mind. Choosing the particular elements depends on the use of the building or the size, the length of spans, or even the owner preference. In discussing the general types of building systems, they will be classified into single and multistory buildings. Further division into specific types of structural systems will enhance the following discussion. All of the systems discussed here have been used in many applications throughout the United States, for economical solutions with pleasant aesthetics, and architectural features.

Single Story Construction

The use of precast concrete for bearing walls eliminates the need for a structural frame at the perimeter of the building. This application would be most apparent when clear space inside is not necessary. The walls can utilize either double tees, flat panels, or architectural

precast concrete. Typically, the roof is made up of double tees, which of course, rest on the bearing wall members. The connection between the roof member and the wall member can be handled by either bearing on a haunch cast directly into the wall member; or by bearing directly upon the wall panel.

Beam-column framing is also a common solution to a single story building. Selection of the type of beam to be used depends on engineering considerations, such as span length and live loads, and architectural considerations such as depth and ceiling construction, if any. Connections in general of the beam to column or column to base plate can be either bolted, welded or bearing on elastomeric bearing pads. Consideration of connections will be covered in detail later.

For long span single story buildings, precast-prestressed sections that will span 100 feet or more are available. Longer spans can be obtained by combining post-tensioning with precast concrete.

Multi-Story Construction

Bearing walls can again prove economical in multi-story structures. Precast bearing wall units can be cast in one to four-story high sections, utilizing either standard sections or specially developed sections (12). The bearing wall units can rest directly on the basement wall or foundation. The standard sections can be used or a variety of architectural shapes are available. Beam-column framing is sometimes used on the first floor to provide more open space, with the bearing walls starting on the second story. In high-rise structures, transverse load bearing wall panels can be used effectively to resist lateral forces and to support the floors and roofs. Window frames can be integrated between wall panels for the exterior facade. Load bearing walls can also be used with a cast-in-place

central core area to permit a column free interior. Any of the standard units then can be used for the floor and roof elements. Connections are made in the usual bearing wall fashion.

Precast-prestressed beams and columns can be used for either high or low rise multi-story construction. Architectural and engineering considerations dictate whether the beams are continuous over single story columns, or whether multi-story columns are used with single-span beams. In many cases, the framing becomes an architectural feature and aesthetic consideration is given to the connections between beams and columns.

Dyna-Frame Structural System

The Dyna-Frame Structural System is a method of building construction that incorporates fire-resistant precast concrete, Dyna-Frame structural frame elements, prestressed hollow core floor elements, and roof decks. The column and beam system is readily available in standard sizes. Because of Dyna-Frame's simplicity and uniformity throughout the structure, economies can result when applied to either single story or multi-story applications. The Dyna-Frame system is replacing structural steel and cast-in-place concrete construction in the design of many buildings because of specific economic advantages to the owner resulting from speed of erection and fire-rated members (3).

The key to the system is a unique precast concrete column and the method for effecting a column splice. The columns are always single story units, thus making it necessary to have a column splice at each floor. The columns are reinforced with a hot-rolled, seamless structural steel tube running longitudinally down the center of the column. The inside diameter of this tube is held constant at 4 inches; the wall thickness

is varied to suit load requirements. The percentage of steel is in the range of 1/2 of one percent to 6 or 8 percent of the gross cross-sectional area of the column. A wire coil is placed around the core and stretched out in the form to serve as spiral ties; contributing nothing to the capacity of the column, but improving the mode of failure at ultimate load. Three 3/8" 250K or 270K prestressing strands are placed within the spiral in the column and are stressed to 10 kips each. The small amount of prestress given to the columns by the three strands is helpful in resisting flexural stresses and controlling cracking in the column during shipping and erection. Figure 2b shows the column cross-section and reinforcing steel.

The floor and roof beams always sit on top of their supporting columns, never framing into them. This condition permits the designer a great deal of freedom in the design of beams. He can design using simple span beams as in design "A" of Figure 3, or multi-span beams as in design "B", or in a semi cantilevered configuration with beam splices at, or near, the natural inflection points as in design "C". The beams are conventionally prestressed with straight strands. In design "B" and "C", there is of course both positive and negative bending moments present in some beams, therefore, the straight strand has to be closer to the neutral axis than in design "A". This reduced eccentricity in the prestressing force eliminates camber and camber growth problems and at the same time, reduces live load deflections. In practice the beams are dead level (13).

There are two basic connections for the beams - one is at the column, and the other is at the beam splice. The beam-column connection consists of a 6 inch inside diameter round pipe which is cast into the beam to be concentric with the core of the supporting column. The space between the bottom of the beam and the top of the column is grouted, but not until all

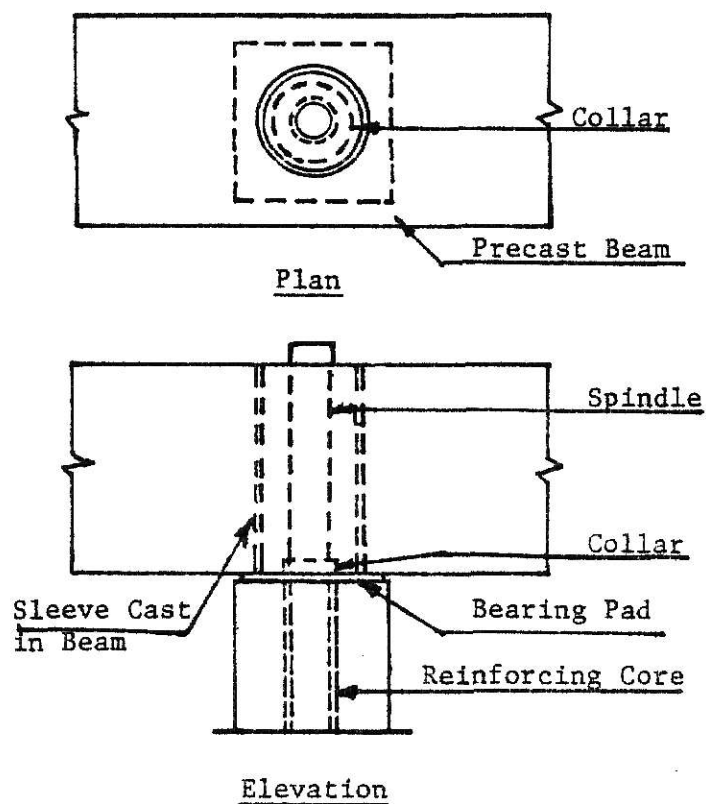


Figure 2(a): Column-Beam Connection

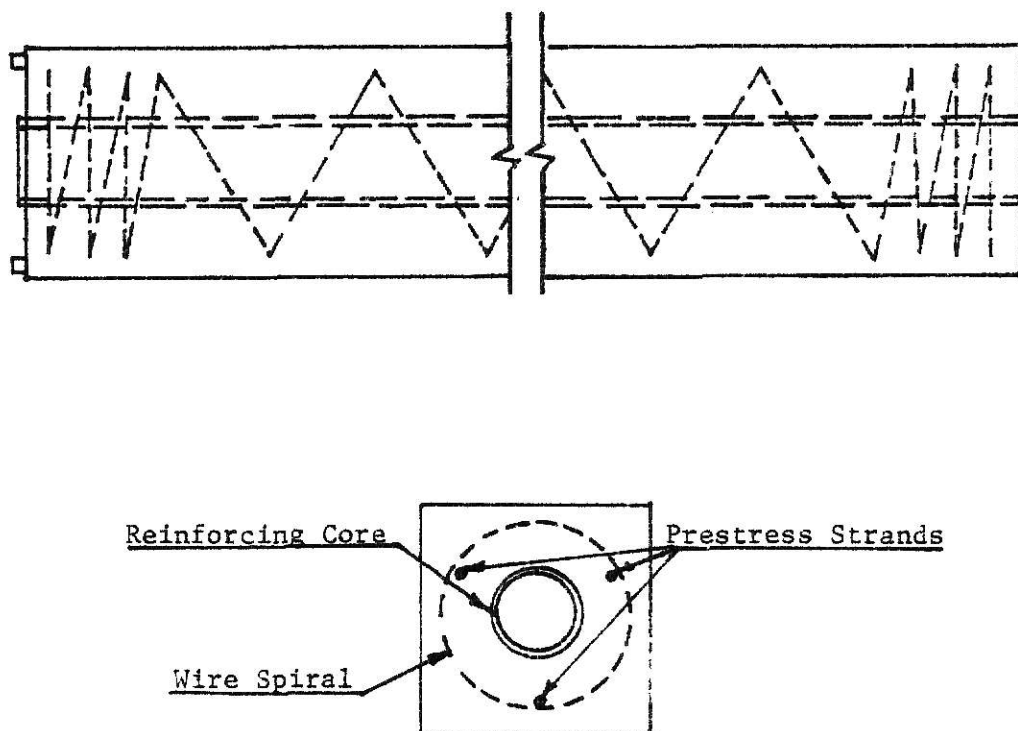
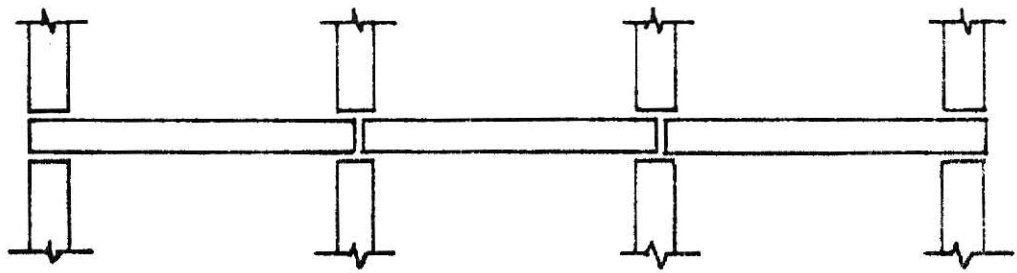
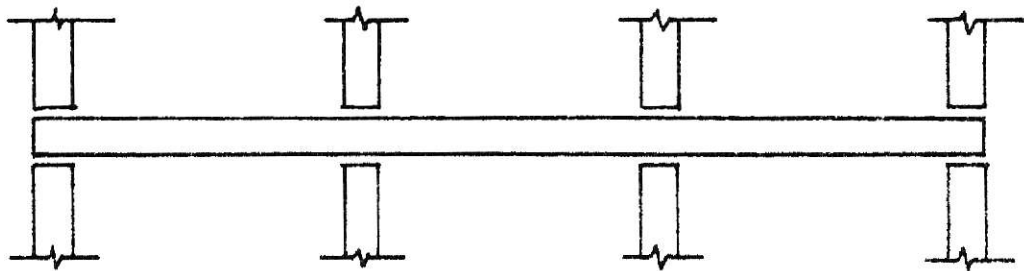


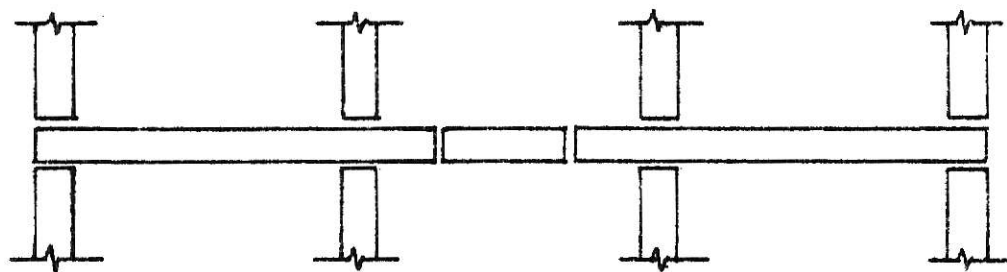
Figure (2b): Column Reinforcement



Design "A"



Design "B"



Design "C"

Figure 3: Dyna-Frame Beam Design
Source: Reference 13

the floor planks have been laid. Guide cones are set on top of the columns to bring the beams into proper position on the column. After the beam has been seated, the cone is pulled up through the sleeve in the beam, completing the erection connection. The beam splice is designed to take the entire reaction of the supported beam. The connection angles are located in the top of the beam between the ends of the floor planks. Except where a slip or expansion joint is desired, mild steel reinforcement is extended through the joint a sufficient distance to insure an adequate tie between beams. No attempt is made to transfer moment through the connections.

In the erection of the columns, a threaded pipe or spindle with a constant 4 inch outside diameter projects up out of the footing. A short collar is threaded over the spindle. Its outside diameter is the same as that of the core in the column above. The collar is adjusted for elevation and the column is slipped down over the spindle until the core bears on the collar. The bottom of the column is dry packed or grouted to complete the connection. Figure 2a shows the column connection.

Deck slabs are fully precast-prestressed concrete hollow core units as in Figure 1. After the deck units are placed on the beam soffit, a structural composite pour between the butt ends of the slabs monolithically ties the frame and deck into an integral unit. The deck units can be used with or without the 2 inch structural topping. The Dyna-Frame System is available in the midwest through Inland Concrete Company.

Dyna-Frame Systems can be used in a wide variety of building structures to meet many design problems. The largest project to date consists of a 320' x 900', one story, open structure. The bays are 30' x 30' and the roof is utilized as a parking deck. Multi-story structures

have also been built with Dyna-Frame and have proved economical. The tallest structure so far is a 14-story H.U.D. building. A hospital utilizing Dyna-Frame has been designed for 20 stories, but is presently only at 10 stories, the other 10 stories is to be constructed later (3).

Versa-Space

Versa-Space is another precast structural system. Versa-Space utilizes double tee wall units and double tee roof units. This particular system is an example of using standard elements and developing them into a specific system, able to be marketed as a product, i.e., a warehouse building. Refer to Figure 4 for a typical example of a versa-space application. The columns are square precast-prestressed units which support inverted tee beams. The double tee roof units are supported on the beams and the load-bearing wall units. Standard connections can be used, and since standard components are used, this system can be used nationwide by any of the producers that make standard elements.

The Versa-Space structural system is particularly applicable to warehouse and industrial type buildings where much open space is needed. The double tee wall units can extend to an inside height of 30 feet and the roof units can span 40 feet to 72 feet long. Future expansion is less costly than other systems because the concrete wall panels may be demounted and relocated.

Versa Space building system designs are completed for wind loads of 15, 20, and 25 psf. Lateral stability is provided through the diaphragm action of the roof which transfers lateral forces to the shear walls. The double tee roof and wall members are prestressed and are standard sections which can be designed from load tables in several sources (12). Versa-space is a building system developed and marketed by affiliates of Precast

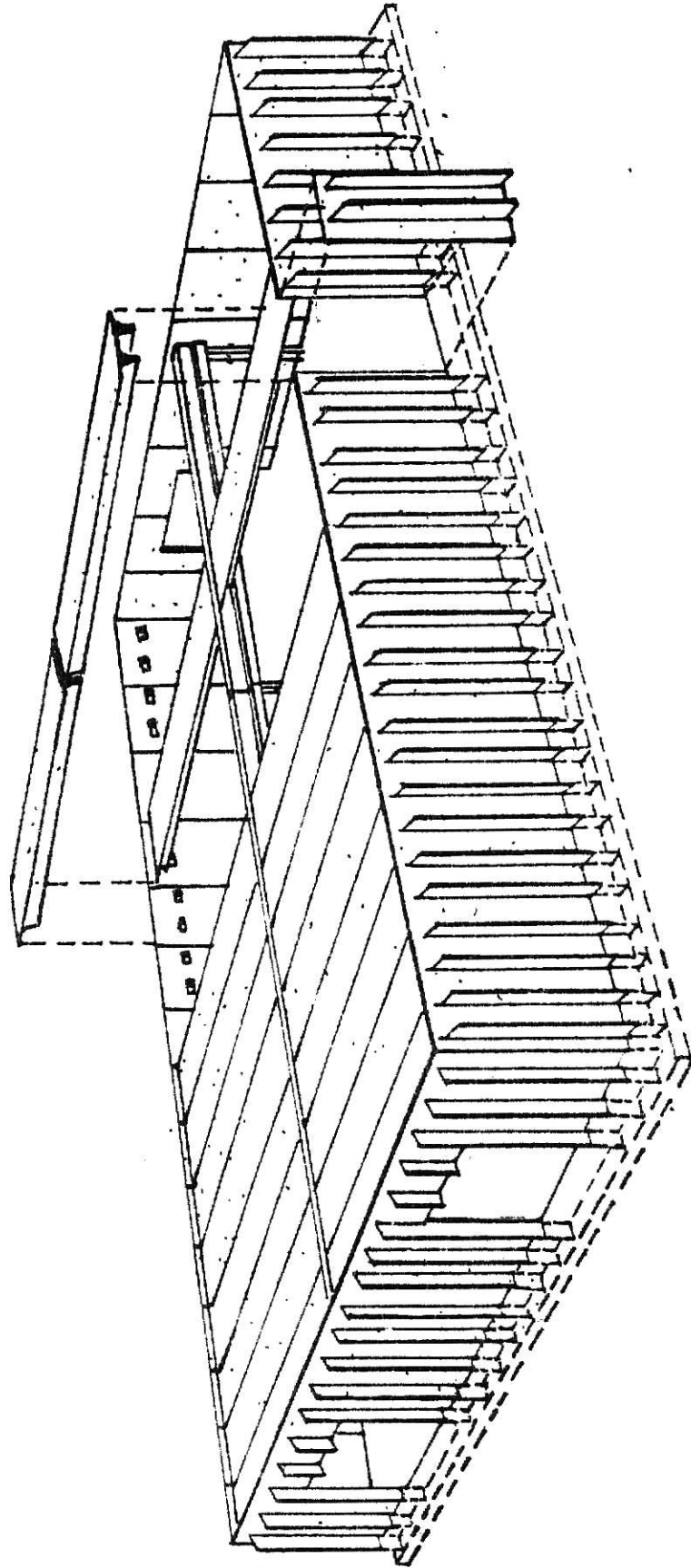


Figure 4: Versa-Space Building

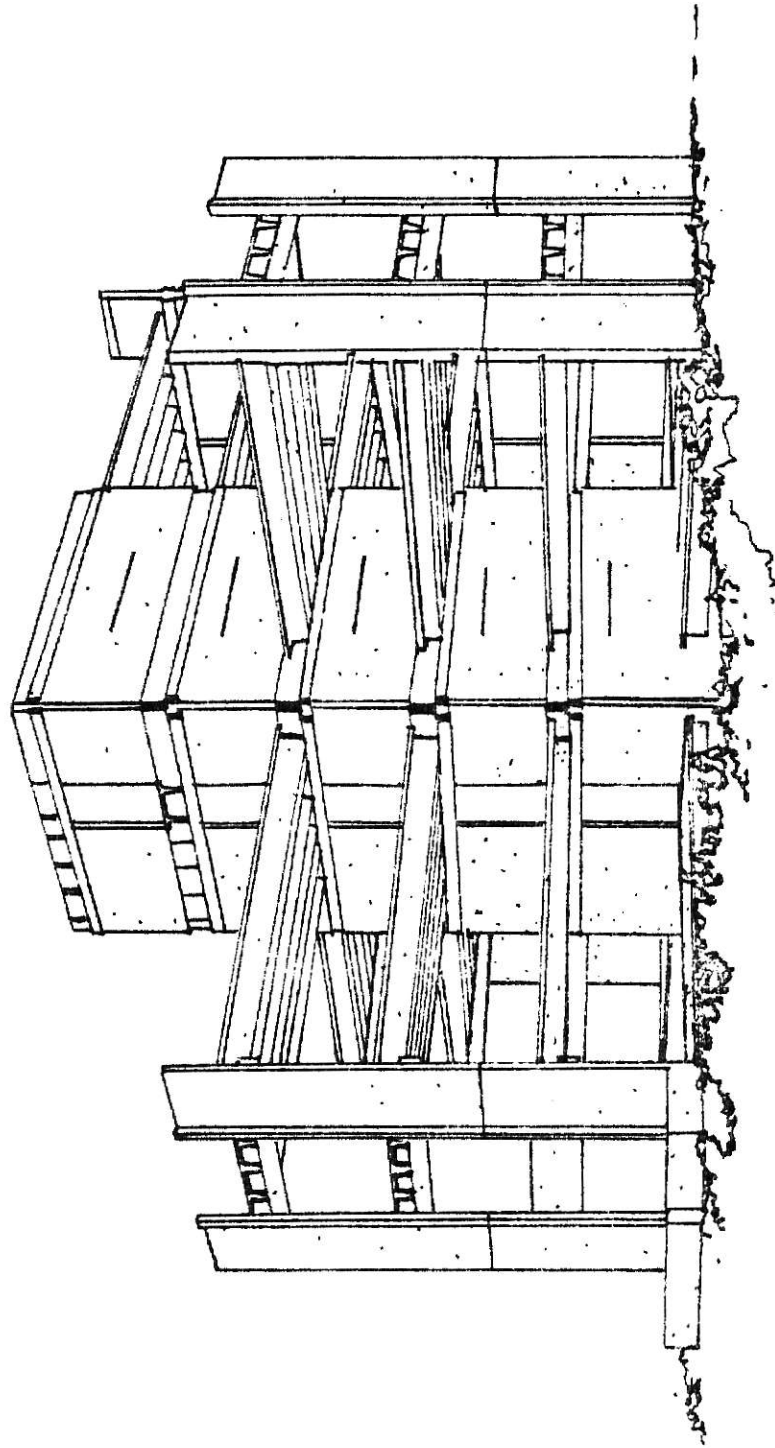


Figure 5: Vantage Space Building

Systems, Inc. They are available nationwide.

Vantage Space

Vantage Space is another building system developed by Precast Systems, Inc. Vantage space is mainly applicable to low-rise and high-rise multi-story buildings. It utilizes load bearing walls of architectural precast concrete which provide the exterior architectural character of the building. Vantage space utilizes a precast concrete service core which supports the floor deck units and transmits loads to the foundation while housing, supporting, and protecting most service functions. Double tee floor decks are used which radiate out from the service core to the load bearing walls. Walls can be made up of double tee sections or flat slab wall panels or the channel type member. Figure 5 depicts a vantage-space building.

Holes and inserts are prepositioned at the plant for installation of mechanical and electrical equipment. The core area is entirely precast concrete including walls, floor decks, and stair assemblies. Erection may be accomplished without the aid of temporary bracing or formwork.

Floor beams are required for each floor level as they span between the core assembly and the load-bearing wall panels. These beams are prestressed concrete "L" shaped beams and support the double tee deck units. A 3 inch concrete topping on the double tees completes the floor system.

The architectural wall panels may be double tees or other shapes, depending upon specific architectural requirements. The panel sections are manufactured in forms that equal the building height, thus allowing each full height building panel to be cast as a unit. After casting, panels are separated into convenient segments for hauling and erection.

This match casting technique simplifies the erection process and insures trouble free fit as erection proceeds. The match-casting process may also be employed for the service core panels. A ledge beam is cast into the wall panels and serves many purposes. It supports the double tee deck units and also allows for connection to the wall panel already connected to the floor; so temporary bracing is unnecessary.

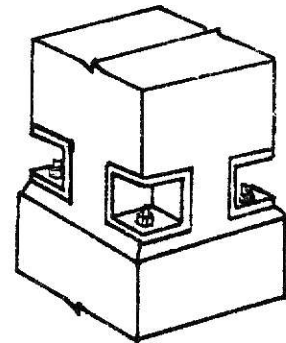
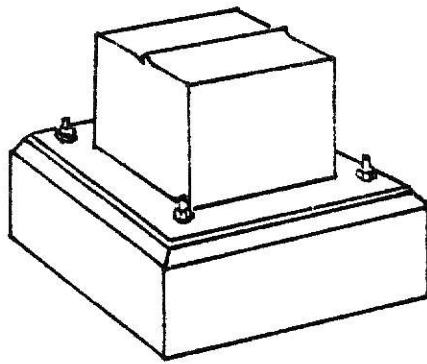
CHAPTER 3 - CONSIDERATIONS FOR CONNECTIONS

Precast structures resemble steel construction in that the final structure consists of large numbers of prefabricated elements which are connected on the site to form the finished structure.

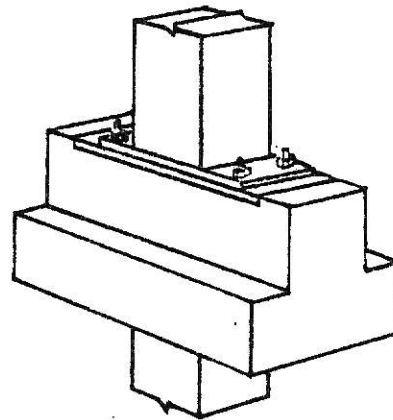
An over simplification is often made that only two types of connection systems, either "hard" or "soft", are available for resisting the various forces applied to a connection. A hard connection can be defined as one having steel plates or other shapes in the members being connected, with the connection made by welding or the use of high strength bolts. A soft connection could be defined as having two members simply resting one on top of the other with an elastomeric or other bearing pad material between them. Figure 6 shows some typical connections.

The fundamental difference between hard and soft connections should be whether or not limited rotations or movements are allowed to take place within the connection.

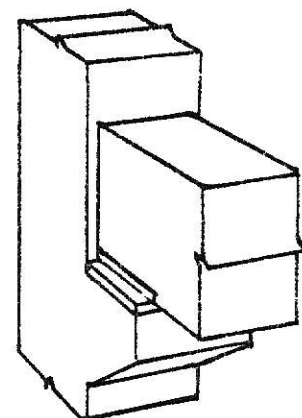
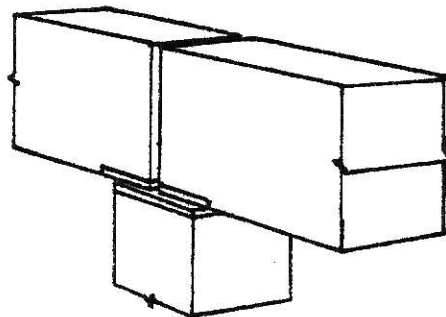
Successful design of connections cannot be achieved without fully considering production requirements. The understanding of precast concrete production produces economies in connections and also suggests ways in which the connection detail will work as intended. Standardization of connections is also an important aspect of connection design as it improves quality control and contributes to production economies. Standardization can apply to the actual elements in a connection; for example, if the majority of connection details required a $3/8$ in. plate, while in some cases a $5/16$ in. plate would be adequate, all the connections should be made with the $3/8$ in. plate (10). More generally, when the majority of the connections on a project are required to support an 80 kip load, whereas a few are subjected to 50 kips, all the connections should



Column To Foundation



Column To Column



Beam To Column

Figure 6: Standard Connections
Source: Reference 10

be designed for the 80 kip load. Little is gained in slight changes in dimensions, since the saving in materials may be more than offset by the extra labor involved in developing modifications.

A practical consideration in connection design is a limit on the size of reinforcing bars. Bars larger than No. 6 require embedment lengths for anchorage that may be impractical for the connection, difficult to bend, and can't be bent at right angles. When welding reinforcement, one must be careful not to weld in areas of a cold bend. This results in crystallization and unpredictable behavior of the reinforcing bar at the bend (10).

Connection details that require a large amount of added reinforcement in the ends of precast members can create production difficulties. Improper vibration of the concrete, with resulting honeycombing within the connection; or congestion of the bars may result in their being out of position.

When using embedded steel shapes in precast concrete; angles, plates and others, consideration needs to be given to the proper attachment to the forms. If the steel shapes cannot be held securely in the forms, they may become misaligned or skewed relative to their planned positions.

It is obviously important that voids and honeycombing in the ends of the precast members are undesirable. This often occurs when plates or angles are positioned so that concrete must be worked underneath them. For these situations, air release holes should be drilled in the horizontal portion of the steel shape, so that the entrapped air can escape thus reducing the tendency for honeycombing or voids.

Selection of tolerances for connections is as important as the structural analysis.

Tolerances must be compatible with the engineer's design to insure that, when used to the limits, the elements are not overstressed. All connections should provide the maximum tolerance that is structurally or architecturally feasible. Connection details should consider the possibility of the bearing surfaces being misaligned or warped from the desired plane. Adjustments along the order of $\pm 1 \frac{1}{2}$ " can be provided with the use of drypack concrete; or adjustments of $\pm \frac{1}{4}$ " with the use of elastomeric bearing pads.

In selecting tolerances, it must be remembered that different subcontractors may produce the members meeting at a connection. Particularly in the case of a bolted precast column to a cast-in-place foundation. The field concrete may be at an improper elevation, resulting in variances as much as $1 \frac{1}{2}$ ". Anchor bolts may be out of position by as much as 1 in., as well as being out of plumb. In this case where non-shrink grout may be used between the two surfaces, the planned dimension between the column and the foundation should not be less than $1 \frac{1}{2}$ " and a $2 \frac{1}{2}$ " dimension would be even more desirable.

During erection, loading conditions may occur that will control the connection design. These temporary conditions may result from eccentric loads, wind, construction loads, or impact which may place a far greater loading condition on the connection than after it is completed and live loads are imposed.

A review of the complete construction sequence may be necessary in order to satisfy the temporary conditions on a connection. Such a review may indicate that a particular column should be braced, instead of designing the base connection for withstanding the loading. Economics here are, of course, the deciding factor. If particular erection sequences

or procedures are desired, or if the designer is concerned with erection loads on the connections, then the engineer should require the erection sequence to be shown on the shop drawings.

Welded connections should be thoroughly investigated and should not be used indiscriminately by the designer. Additional forces due to restraint against volume changes may require increased strength for all connections. Where only a few field welded connections are used in a project, it is usually more economical to use an alternate connection rather than require an additional trade on the jobsite.

Connections requiring cast-in-place concrete for their completion pose a number of situations that must be considered. Where possible, the connection detail should be self-forming, for example a dowel and socket connection, where a column is cast with projecting reinforcing bars and some sort of conduit is cast into the foundation. After the column is in place, high strength non-shrink grout is placed around the reinforcing bars in the conduit.

With bolted connections, $3/4$ in. or 1 in. bolts are considered standard sizes in the precast industry and should be used. Occasionally, $1\ 1/4$ " bolts are needed, but always use $3/4$ " as the minimum regardless of the load.

All precast concrete connections must be designed to satisfactorily resist gravity loads resulting from dead and live loads, wind loads if frame action for wind is a design consideration, earthquake loads, or any other lateral loads that might be induced. These normal structural load considerations result in connections that resist tensile or compressive forces, shear, torsion or bending moments. The forces and stresses imposed on the connection by typical loads can be altered if special

loading resulting from restraint against volume changes on rotations, or prior overloading during erection occurs.

Neglect of loads due to volume changes may result in a connection that is underdesigned and potentially dangerous. Volume changes in precast prestressed members are caused by creep, shrinkage, and temperature change. When this potential movement is restrained sizeable forces may occur. Restraints can be developed in connections in a variety of ways. In flexural members, it is either by friction in the connections or by welding one or both ends of the members. Thorough grouting of dowels in the ends of beams or columns is another way of resisting axial movement in the members. When volume change restraints occur in flexural members, horizontal forces may develop in the connection that may be great enough to reduce significantly the assumed capacity of the connections.

To emphasize the importance of volume change loads, it is possible to completely destroy the effectiveness of a negative moment connection made with cast-in-place concrete closure (9). Forces due to restraint could be large enough to cause the negative moment reinforcement to yield and the ends of the beams to pull away from the concrete connection. With little or no negative moment capacity, the flexural members will be forced to support the total load as simple span beams, a condition that may exceed the positive moment capacity of the beam.

Every connection has to be designed either to resist fully the volume change loads that may develop due to restraint or to limit the magnitude of the forces or to reduce restraint build up. A connection that permits minor rotations and movements without placing the connection in distress has distinct advantages as long as satisfactory lateral resistance is obtained.

Rather than arbitrarily selecting either a hard or soft connection,

the designer should fully explore and analyze all forces on the connection, and develop his design to withstand these forces.

Load Factors For Connections

In selecting appropriate load factors for connections, it is recommended that they exceed those required for the individual members being connected (19). This recommendation is made because connections generally are subject to high stress concentrations whereas significant warning deformations and rotations of connected members occur under ultimate conditions. Slight variations of the as-built connection from the designed connection may cause possible changes in the magnitude, direction and position of loads on the connection. In view of the importance of connections, the PCI committee on connection details recommends an additional load factor of 4/3 for the ultimate design of connections (10).

Load factors of $1.4D + 1.7L$ are given in Section 9.3.1, ACI Building Code (ACI 318-71). When volume change effects are considered (Section 9.3.7 ACI Building Code), they are to be included with dead load in $0.75(1.4D + 1.7L)$. However, when considering volume change effects in brackets and corbels, the resulting tensile force should be included with the live load with a factor of 1.7 and no overall reduction (Section 11.14.2 ACI Building Code).

Bearing stresses on plain concrete are limited by the Code to $0.85\phi f'_c$, except when the supporting area is wider on all sides than the loaded area A_1 . In such a case, this value of the permissible bearing stress may be multiplied by $\sqrt{A_2/A_1}$ where A_2 is the maximum portion of the supporting surface, which is geometrically similar to and concentric with the loading area (19).

In heavily loaded members or those which resist large lateral forces, the limits for plain concrete bearing may be exceeded. Auxiliary

reinforcement should be provided at these locations. This reinforcement should be designed in accordance with the shear-friction theory as described in Section 11.15 of ACI 318-71. Shear friction applies when it is inappropriate to consider shear as a measure of diagonal tension, and provides a lower bound ultimate strength approach that can be used to evaluate many different connection types. A basic assumption in applying shear-friction theory is that the concrete within the connection area will crack in the most undesirable manner. Ductility is achieved by placing reinforcement across the ultimate failure plane where the force A_{vf} developed by the reinforcement is normal to the plane. This normal force in combination with friction at the crack interface provides for the shear resistance. The reinforcement required can be calculated by:

$$A_{vf} = \frac{V_u}{\phi(f_{yv})(\mu)}$$

where

$$\phi = 0.85$$

A_{vf} = area of shear-friction reinforcement

f_{yv} = yield strength of A_{vf}

μ = shear-friction coefficient

The Code specifies:

$\mu = 1.4$ for concrete cast monolithically

$\mu = 1.0$ for concrete placed against hardened concrete

$\mu = 0.7$ for concrete placed against a-rolled structural steel

If an axial force N_u is present, then A_{vf} should be calculated by:

$$A_{vf} = \frac{1}{\phi(f_{yv})} \left(\frac{V_u}{\mu} + N_u \right)$$

$$\phi = 0.85$$

N_u is determined by analysis but reference (12) recommends that a value not less than $N_u = 0.2 V_u$ be used. Reinforcement should be welded to confinement angles as shown in Figure 7.

Types of Connections

Ultimate strength design relationships for concrete bearing depend upon the type of loading, forces within the bearing area and magnitude of the bearing stress. The design concept must be modified when dealing with bearing pads which are designed by working stress design. Bearing pads are used at connections of precast members to provide uniform bearing. Some types of pads also help relieve stresses caused by restraint of rotation and volume change strains, either by slipping or by shear deformations within the bearing area. Commercial bearing pads used today are either elastomeric, laminated cotton duck, preformed pads composed of synthetic fibers and a rubber body, or tetrafluorethylene (TFE).

When the connection is recessed, or dapped, into the beam end, additional precautions should be taken. The shear-friction reinforcement and the stirrups should be designed as shown previously. However, the shear-friction reinforcement should have positive anchorage by welded cross bars or by welding to confinement angles as shown in Figure 8. Additional horizontal bars, in the form of closed ties should be placed as shown. The shear span ratio of a/d should not exceed 0.40, $\phi A_{vf}/bd(f_{yv})$ should not exceed 600 psi and h should not be less than one half the overall depth of the beam.

Corbels and brackets are widely used in precast construction for supporting beams at columns or walls. The structural performance of a corbel or bracket is fairly complex, however recent proposals have applied the laws of statics to corbels which resist a combination of vertical and

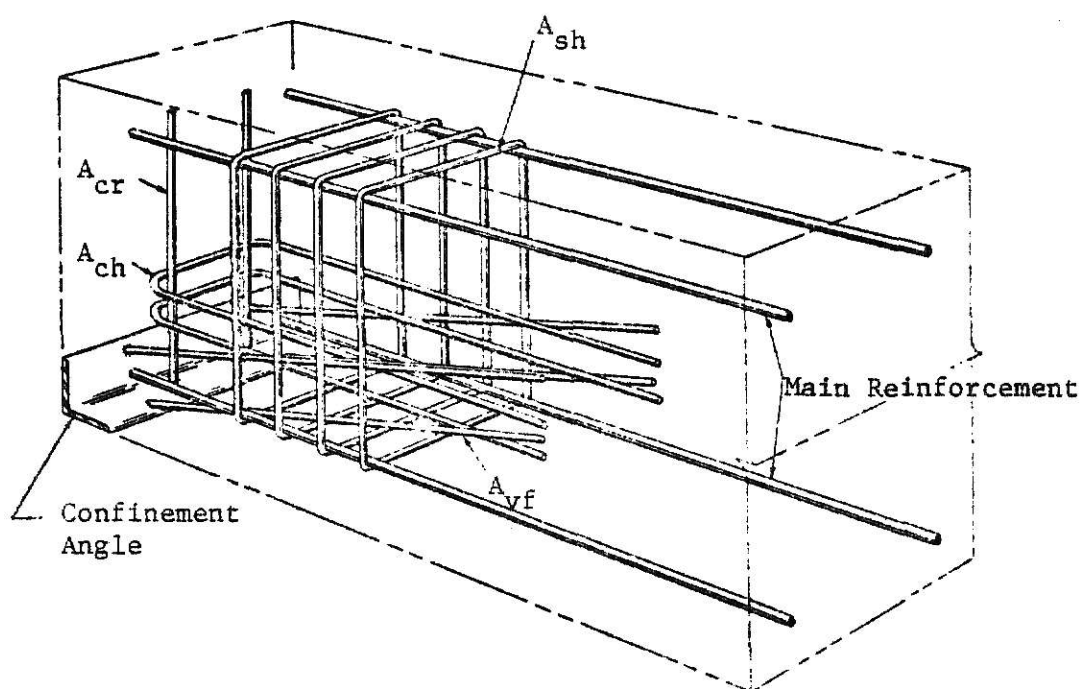


Figure 7: Beam Reinforcement
Source: Reference 11

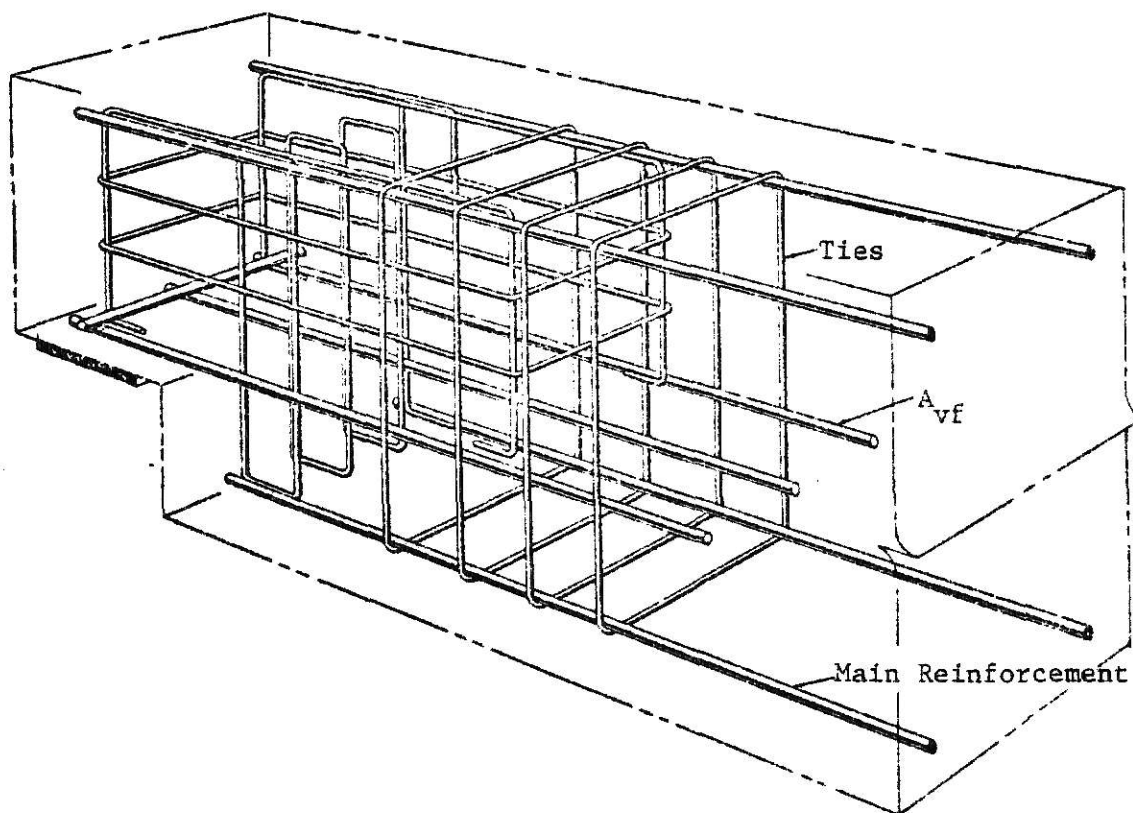


Figure 8: Dapped-Ended Beam
Source: Reference 12

horizontal loads (11). The corbel is considered as a "free body" cut from the column at the corbel-column interface as shown in Figure 9. The proposed design is applicable to corbels with shear-span-to-depth ratios of unity or less, or the ratio of a/d taken from Figure 9. If the corbel is subject to tension due to restrained creep and shrinkage, ACI 318-71 specifies that v_u shall not exceed:

$$v_u = \left[6.5 - 5.1 \sqrt{\frac{N_u}{V_u}} \right] \left[1 - 0.5 \frac{a}{d} \right] \times \left(1 + \left[64 + 160 \sqrt{\left(\frac{N_u}{V_u} \right)^3} \right] \rho \right) \sqrt{f'_c}$$

where $\rho = A_s/bd$ shall not exceed $0.13f'_c/f_y$ and shall not be less than 0.04 (f'_c/f_y). N_u/V_u shall not be taken less than 0.20 and the tensile force N_u shall be regarded as a live load. The area of reinforcement A_{vf} needed across the shear plane to carry shear, using Eq. (11 - 30) of ACI 318-71 is the same as before:

$$A_{vf} = \frac{V_u}{\phi(f_y)\mu}$$

where

$$\phi = 0.85$$

For corbels cast monolithically with the column:

$$\mu = 1.40 \text{ for normal weight concrete}$$

$$\mu = 1.4(0.85) = 1.19 \text{ for sanded lightweight concrete (unit weight not less than 106 lb. per cu. ft.)}$$

$$\mu = 1.4(0.75) = 1.05 \text{ for all lightweight concrete (unit weight not less than 92 lb. per cu. ft.)}$$

The design ultimate moment the corbel-column interface must resist is:

$$\text{Req. } M_u = V_u(a) + N_u(h - d)$$

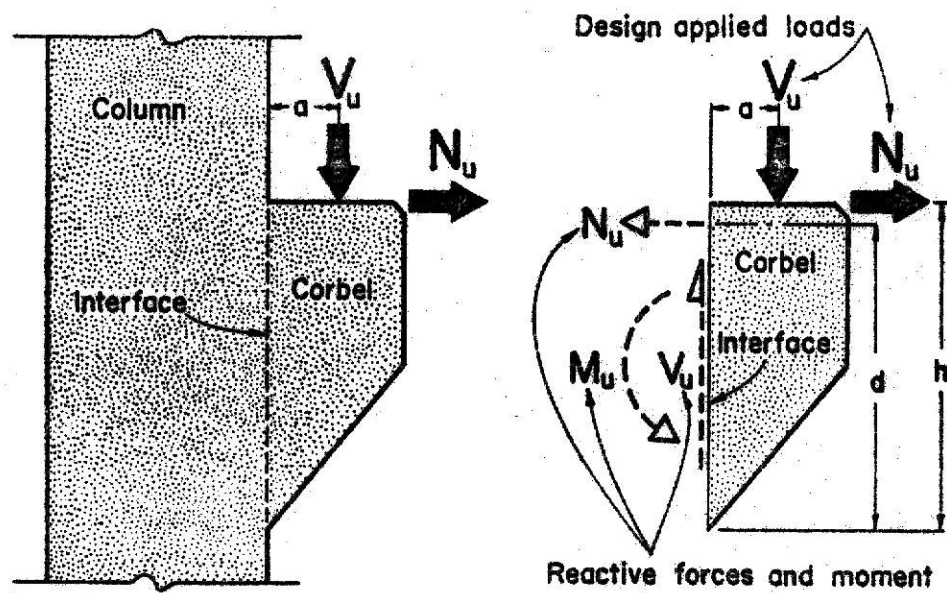


Figure 9: Typical Corbel, And The Corbel As A "Free Body"
Source: Reference 11

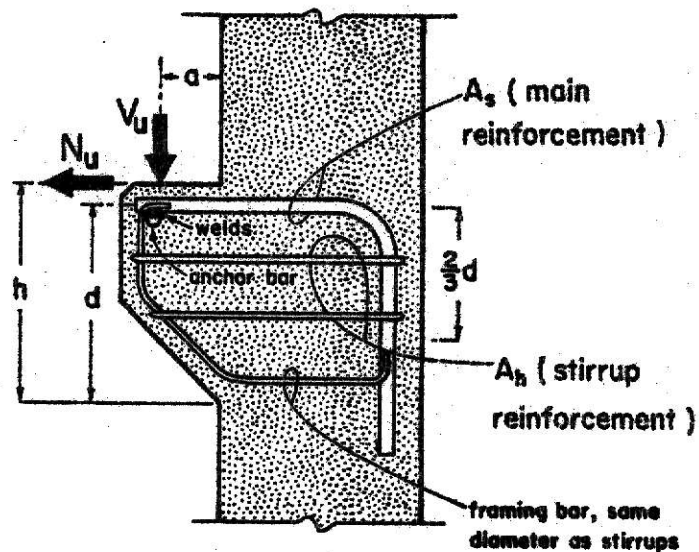


Figure 10: Typical Corbel Reinforcement
Source: Reference 11

The area of reinforcement required to resist M_u is taken from Sec. 10.2 of ACI 318-71:

$$A_f = \frac{M_u}{\phi f_y (d - a'/2)}$$

where

M_u = ultimate moment

f_y = yield strength of A_f

a' = depth of the rectangular stress block.

$\phi = 0.85$

Reference (11) proposes $\phi = 0.90$.

The area of reinforcement A_t necessary to resist the horizontal force N_u is:

$$A_t = \frac{N_u}{\phi f_y}$$

where

$\phi = 0.85$

For $A_f > 2A_{vf}/3$, the total area of main tension reinforcement A_s is calculated using:

$$A_s = A_f + A_t$$

If $A_f < 2A_{vf}/3$, calculate A_s using:

$$A_s = 2A_{vf}/3 + A_t$$

where $\rho = A_s/(bd)$ is not less than $0.04(f'_c/f_y)$. See Figure 10.

Embedded structural steel shapes, usually wide-flange sections, are also used in connections of precast concrete. The capacity of the connection is mainly dependent upon the size of the column, the compressive strength of the concrete, and the effective flange width of the steel section. The capacity of the connection can be increased by the addition

of angles or plates to increase the effective width, b , of the steel shape or by welding vertical reinforcement to the steel shape. In all cases, the flexural and shear capacity of the steel section must be checked. Structural shapes embedded into a precast member which has less than 36 in. of concrete above or below the embedded structural shape may require additional confinement or anchorage reinforcement to insure that the concrete in bearing can develop $\phi f'_c$ at ultimate strength.

Other connections are made with welded headed studs, deformed bar anchors, high tension bolts, connection angles, wedge inserts, and others. Design aids for connection design can be found in references (10) and (12).

Progressive Collapse of Load Bearing Walls

When a local failure is not confined to the area of initial distress, but spreads either horizontally or vertically through the structure, it is termed a progressive collapse (5). The progressive collapse is usually the result in the structure's inability to bridge over the local failure, that is, its lack of integrity. Large panel precast structures are certainly vulnerable in this respect.

Continuity is essential to develop the bridging capabilities needed for transmission and redistribution of loading. Ductility is necessary to sustain deformations associated with partial stability, and also to establish some measure of energy absorption under the dynamic effects of both normal and abnormal conditions.

To prevent progressive collapse, tensile capacity of connectors in each direction must be provided. With the large panel members tied together horizontally and vertically in most instances wall panels become "ineffective" instead of removed as a result of abnormal loadings. Abnormal loadings might be an explosion or some other loading which might totally

incapacitate the member. The panel may no longer function as a load-bearing member as originally intended but will at least remain in place. In some cases, an exterior panel was blown out, then the panels above failed, and the debris falling caused failure of the panels below, or a progressive collapse of the structure. With adequate design of the connections, the whole building would act like a very deep and very short cantilever beam. The connection must also be designed to withstand the effects of forces due to restrained creep, shrinkage, temperature change and differential settlement; further complicating the design of the connection.

Performance of Precast Structures in Earthquakes

Typically, in the past, precast structures have been very vulnerable to earthquakes. In the Alaskan earthquake of March 27, 1964, many precast-prestressed structures failed (8). However, in most cases it was not the actual members that failed, but the connections. In fact, prestressed concrete structures hold a position between steel structures and conventional reinforced concrete structures when elastic behavior, a property of special importance in determining earthquake response, is considered (17).

Connections should be designed for, or at least checked for, every stress and strain which may possibly occur during an earthquake. They should be ductile and free from brittle failure (14). In the case of post-tensioned precast structures, the tendons should be grouted to avoid unexpected failures of the elements.

In Rumania, a 7.2 Richter earthquake struck on March 4, 1977. High rise buildings built after seismic provisions were incorporated into their code (which are not as stringent as the U.S. Uniform Building Code) performed satisfactorily with minimum distress, and many suffered no noticeable damage (14).

As long as connections for precast concrete structures are designed for the increased lateral loads induced by earthquakes, they will resist earthquakes to a great degree (15).

CHAPTER 4 - SYSTEMS BUILDING

Systems building is a multi-faceted goal toward which we continue to strive but never quite expect to reach. To approach this broad subject in specific terms, is somewhat difficult. Some of the major elements of systems building include; pre-planning, industrialization, functional integration, the modular concept, and organization.

Pre-planning involves extensive planning and engineering without consideration of specific projects. To do this, you are in a sense, investing time and productive effort in the hope of recovering dividends through duplication on many individual projects. An example of this might be the extra effort put forth initially in establishing standard detail sheets, so that only slight modifications are needed to suit a particular situation. This is but a small example; the total concept involves not only the physical building, but the entire process of development, design, production, and construction.

Industrialization, like pre-planning, is an investment. Industrialization is an investment in plant and equipment based on the expectation of providing sufficient volume to justify the expenditure. It is the lack of a reasonable guarantee of continuing volume that inhibits the industrialization of concrete building components (7). Of particular concern to the precast-prestressed producer, as systems building and industrialization approaches, is the continued value of his principal production tool, the long-line prestressing bed. As shapes for systems construction get more complex, considerable ingenuity and development of the long-line bed will be a necessity.

Segmental construction, that is, the post-tensioning of precast segmented units is another important industrialization consideration. The obvious versatility of the form of construction is very attractive.

As highly complex integrated structural systems are developed, the usefulness of this technique will increase.

Functional integration can best be described as the opportunity to design dual or multiple functions into systems' components; utilizing one basic function to perform in the capacity of others. For example, a double tee floor unit can support its design load while providing a finished reflective ceiling and also may serve as part of the air distribution system. The ultimate goal would be to achieve a composition of the structural, electro-mechanical and esthetics. The capability of concrete to be molded into any shape makes it the ideal material for this concept.

One of the keys to successful functional integration lies in the field of modular coordination. If systems building is to become successful, a modular framework must be developed to enable the bringing together of systems, subsystems and all parts of a building in an orderly, predictable manner. Without a modular concept, systems building could become just a series of different individual projects with little chance for efficiencies and potential advantages to occur.

In considering systems building, the building occupancy and use is important. While considering particular types of buildings, the relative importance of the major elements which comprise systems buildings vary, but the overall goal of simplifying the total building process is always in sight.

For instance, industrial buildings are mainly a structural system; the needs of the industrial corporation are low maintenance shelters which compliment the operations of a manufacturing process. Industrialization helps but is not essential; architectural design also is secondary. In contrast, school buildings rely heavily on functional integration and

architectural character. A school system is probably the most difficult since in addition to the complications of building design, educational requirements must be considered.

Professional design firms are developing systems and offering completed buildings on a turn key basis. Corporate engineering departments are becoming more common and are, in a similar way, designing standard buildings which meet their particular functional needs. As discussed before, individual companies or groups of prestressers are developing basic structural systems from either special forms or the standard elements. The next step is for the producers to complete the building. Then they would have a definite product, buildings, to market. Obviously, the producers need a nationwide organization, or at least be able to cover several regions to be able to compete with other types of building systems.

CHAPTER 5 - BRIDGE SYSTEMS

Bridge systems of precast concrete have been used in the United States since the early 1950's. Precast-prestressed concrete bridges went through the usual development of the precasting industry although maybe not as fast as the building industry. In Europe and other foreign countries, great advances in the use of precast-prestressed concrete have taken place. Spans of 500 feet and greater have been obtained using segmental type construction (6).

Precast bridges prove economical also in the short range span. Many secondary road bridges today utilize precast concrete elements. A typical secondary bridge may consist of precast, prestressed piling, with cast-in-place concrete pier and abutment beams. Then double tees for short spans or single tees for longer spans are placed between piers. Cast-in-place diaphragms join the tee elements. A cast in place deck topping may or may not be specified. Another example of secondary road bridges may utilize precast deck components supported by standard AASHTO Girders. Continuity is again achieved by utilizing cast-in-place concrete with mild steel for the negative moments over supports.

AASHTO girders are also utilized in the same fashion as steel girders on primary bridges. Precast-prestressed concrete piling may be used with cast-in-place bent caps, which support the precast-prestressed girders. Falsework supported from the girders supports the cast-in-place deck. These types of bridges prove economical in the 50 to 100 foot span range.

For longer spans of precast concrete, segmental construction is more typical. It is essential, however, that the design of the bridge be integrated with the construction and erection methods. Precasting in bridge construction offers the same advantages as buildings of high quality concrete,

economy of repetitive use of precast segments manufactured at a single location, and the problem of deflections during construction can be overcome. Segmental construction employs precast segments cast of high quality concrete in sizes which can be transported and erected. The segments generally range from 5 to 50 ton and 6 to 100 feet in length. These segments are usually reinforced with mild steel and are designed to be connected by post-tensioning after erection. This may consist of temporary post-tensioning to aid during transportation and erection, or the segments may be prestressed at the plant in such a manner that the prestressing is also used in the final position. Segments may be prestressed at the plant in the transverse direction, and later post-tensioned longitudinally after erection.

Joints are of the utmost concern as are the connections of precast buildings. Stress across these joints may be very high. Many types of joints have been employed and proven useful. Poured concrete joints have been widely used in widths from 3" to 24". Some joints have used scalloped edges for transmitting high shears.

Dry joints, where the segments butt directly against each other have many advantages. The general method is to cast each unit against the preceeding segment. This generally requires a second handling of each segment in the plant. Segments are then match-marked for erection. Dry joints enable speedy and economical assembly in the field and assure a perfect joint.

All concreted joints introduce problems of providing duct continuity across the joint, except where external tendons are used. In grouting the post-tensioning tendons, grout may escape at the joint and seal adjoining ducts. Therefore, all the tendons should be stressed and then grouted all at one time.

External tendons eliminate many problems, the greatest one being that of making the webs thick enough for the post-tensioning ducts. Usually the thickness of the web is determined by the necessity of providing room for the ducts. By placing the tendons external to the web, friction may be minimized. The shear from the tendons is transferred by stirrups extending from the web. Encasement after tensioning is provided by pouring concrete, or epoxy castings or steel boxes. It is essential to insure that corrosion cannot attack these tendons.

Almost every standard bridge system has been utilized in constructing prestressed concrete bridges or precast segments; including simple spans, cantilever suspended span, continuous girder, truss, tied arch, double cantilever and arch-cantilever. In specific examples, a combination of these systems have been employed. Some precast segmental bridges have employed this concept throughout the entire structure. Pile caps utilizing precast concrete box sections, then the piers formed of precast hollow box-sections post-tensioned vertically. Bridge box beams utilizing precast-segmented construction then may cantilever out from these piers, making the entire bridge a segment structure.

Precast segments, of very high quality and close tolerances, are readily available from existing precasting plants. They can be transported by barge or truck, erected by derricks, cranes, or any other usual method. The standard elements, also readily available, make construction time short and are becoming more economical by the day.

CHAPTER 6 - HANDBOOK DESIGN

Design aids for the standard structural elements can be found in the P.C.I. Design Handbook or from the producers, themselves. The handbook serves as a guide to the designing engineer. One chapter is devoted entirely to product information and capability. However, the design engineer should consult local precast plants for the exact dimensions and prestressing strand patterns of the particular section he is interested in. The shapes that are included in the handbook are double and single tees, hollow-core slabs, beams, girders, columns, piles, and wall panels. Twenty-eight day cylinder strength for concrete in the prestressed units is assumed to be 5000 psi. Concrete strength at time of strand tension release is usually 3500 psi, however, in some cases release strengths higher than 3500 psi are required. In order to keep the one day casting cycle, the engineer should try to design with the 3500 psi strength.

Load tables show the maximum allowable superimposed load, the approximate anticipated camber under the products dead load, and the calculated deflection due to the allowable superimposed load. Separate load tables are presented for double and single tee units with and without a 2" structural concrete topping. In addition, separate tables are presented for some units for both normal weight concrete (150 pcf) and lightweight (115 pcf) concretes. Where the 2 inch concrete topping is used, the topping concrete is assumed to be normal weight concrete with a cylinder strength of 3,000 psi.

Prestressing strands used in the load tables are either 7/16 inch or 1/2 inch diameter strands with an ultimate strength of 270,000 psi. Different strand patterns are presented with various eccentricities. Strands presented are either straight strands, or depressed strands at one or two points.

Losses assumed in computing the required cylinder strength at time of strand release are 10%. Total losses are assumed to be 22% for normal weight and 25% for lightweight concrete. The load tables provide data by which approximate camber and deflection may be calculated. The deflection values given are for deflection under a 100 pounds per lineal foot live load and based on the modulus of elasticity for 5000 psi concrete.

Two types of load tables are presented in the handbook. Type "A" load tables provide complete engineering data on a particular section. Type "A" tables present the section properties of the unit, the safe superimposed live loads for different common strand patterns, the stresses in the top and bottom fibers and the deflection at the midpoint of the beams under a simply supported condition. The eccentricity at the ends and the midpoint is also given. If type "A" load tables are used, no additional design work is usually needed.

Type "B" load tables provide only a guide to the designer on the feasibility of a section. The tables show the approximate maximum loads with the usual maximum number of strands. In some cases, it will be practical to use more strands, but generally a particular design can be satisfied with fewer strands than shown and must be specifically designed for the case at hand. The type "B" load tables are split into particular section widths with tables for different depths of that width. Each depth beam of a particular width section could be covered by a type "A" load table that could be developed. All the loads shown in the tables are based on the provision of the American Concrete Institute Building Code (ACI 318-71).

Load-span curves are shown for various depths of hollow-core slabs available in all areas of the United States. The curves indicate a safe load capacity for a given span and may be used to tentatively select the

depth of the hollow-core unit. At this stage of the design the local hollow-core slab manufacturers should be consulted because camber and deflection may limit the use of a particular depth unit even though the load carrying capacity is adequate. Both topped and untopped sections are presented.

Interaction diagrams are shown for typical double tees, flat panels, and mullions (as used in window panels). While relatively few sections are shown, interpolation can be made between the various diagrams. Variation of reinforcement or prestress force has a relatively small effect on the load carrying capacity.

The diagrams shown are for loads on panels in place. Stresses induced during stripping and handling will often be the governing design considerations. These stresses can be calculated by conventional structural analysis, considering the number and placement of pick-up points, section properties of the member in any position possible during handling, impact (usually assumed as 50% of the member weight), and the strength of the concrete at the time of handling. An example will further describe the "steel like" design procedure.

CHAPTER 7 - DESIGN EXAMPLE

A short example will suffice in demonstrating the use of the design handbook load tables to pick sections. Checks should be made on stresses, ultimate moments, deflections and camber, and the shear along the beam. The stresses should be investigated at the periods when different loading is on the member; at release and at the service loading. It can be shown that the governing stress at the service loading will be at about 0.4ℓ (12). The ultimate moment is also checked at 0.4ℓ . The design handbook load tables are set up for simple span conditions. Further modifications could be made for indeterminate structures.

Problem

A warehouse is needed with a great amount of clear space. Therefore, double tee roof sections were chosen, with inverted tee beams for the center support. Load bearing double tee wall panels will also be used. Dimensions are as shown in Figure 11.

I. Design Roof Members

Span = 70'

Live Load = 25 psf

Superimposed D.L. = 10 psf

Select Section - 8' wide units selected because of roadway width requirements

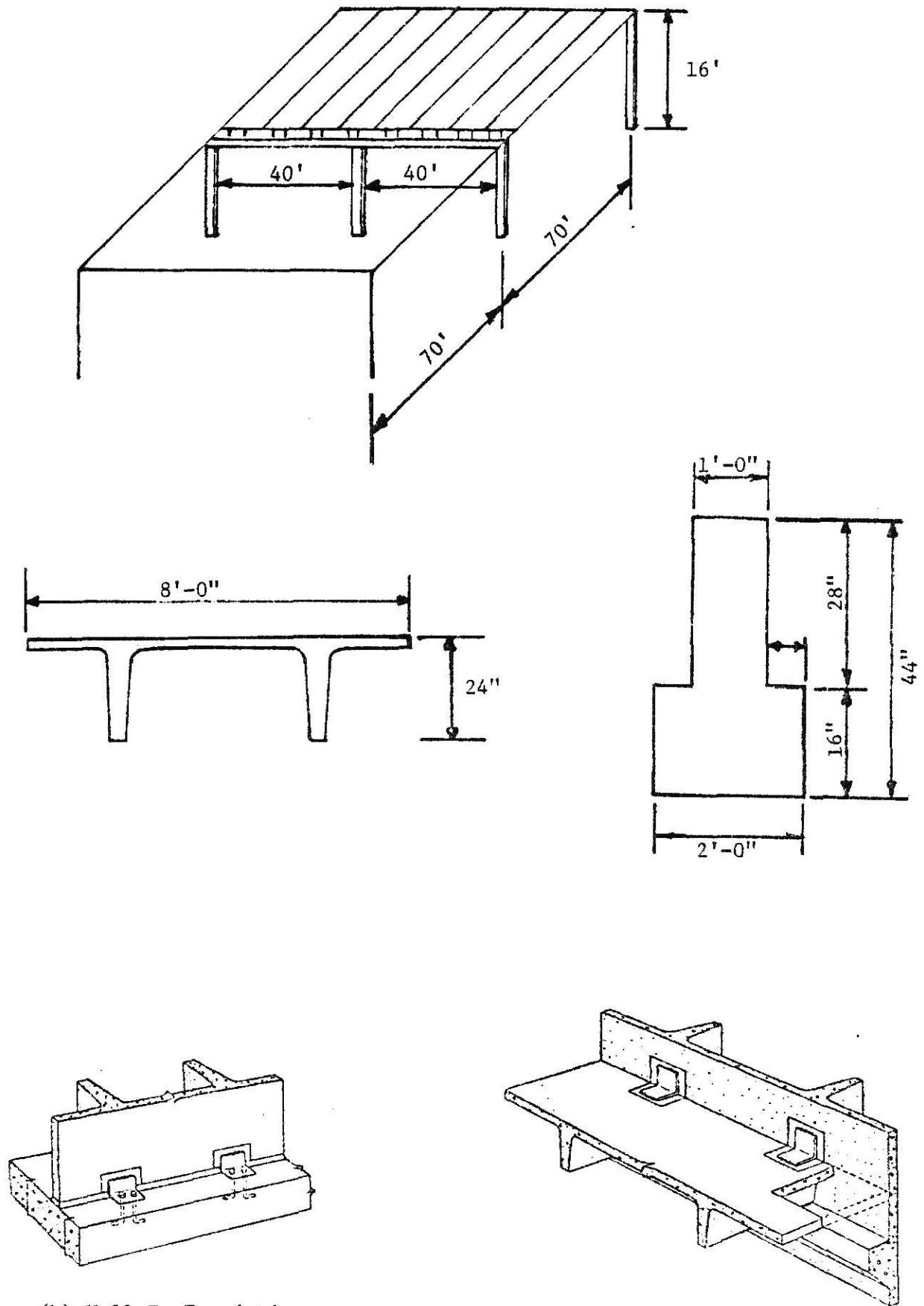
From Type "A" load tables, select 8DT24 (8' wide, 24" depth, double tee)

Section Properties

$$A = 401 \text{ in}^2$$

$$I = 20,985 \text{ in}^4$$

$$Y_b = 17.15 \text{ in (bottom)}$$



(b) Wall To Foundation

(a) Roof To Wall Member

Figure 11: Design Example

$$Y_t = 6.85 \text{ in (top)}$$

$$Z_b = 1224 \text{ in}^3$$

$$Z_t = 3064 \text{ in}^3$$

$$wt = 418 \text{ plf.}$$

Normal Weight Concrete

$$f'_c = 5000 \text{ psi} \quad E_c = 4300 \text{ ksi}$$

$$f'_{ci} = 3500 \text{ psi} \quad E_{ci} = 3600 \text{ ksi (At release)}$$

Prestressing Steel Properties

Seven wire strand, 1/2" Diam.

$$f_{pu} = 270 \text{ ksi (Ultimate)}$$

$$f_{si} = .7(270) = 189 \text{ ksi (Jacking Stress)}$$

Assume 20% Total Losses

$$f_{se} = .80(189) = 151.2 \text{ ksi (Effective stress in prestressing steel)}$$

Select Strand Pattern and Eccentricities

Try 14 strand (7 each stem)

$$A_{ps} = 14(0.153) = 2.142 \text{ sq. in.}$$

$$P_f = (2.142)(189) = 405 \text{ kips}$$

$$P_o = (\text{Assume 10\% initial loss}) = 0.90(405) = 364 \text{ kips}$$

$$P = 2.142(151) = 323 \text{ kips}$$

Eccentricities, Single Point Depression (From Tables)

$$e_e = 3.72''$$

$$e_c = 13.65''$$

$$e @ .4 \ell = .8(13.65 - 3.72) + 3.72 = 11.66''$$

$$e' = 13.65 - 3.72 = 9.93''$$

Investigate Stresses

<u>M @ Midspan</u>	<u>Midspan</u>	<u>M @ .4 ℓ (= .96 M @ Midspan)</u>
$M_{dl} = 0.418(70)^2/8 = 256 \text{ k'}$		246 k'
$M_{sdl} = 0.08(70)^2/8 = 49 \text{ k'}$		47 k'
$M_{ll} = 0.200(70)^2/8 = 122.5 \text{ k'}$		117.6 k'

LOAD	Support @ Release $P = P_o$		Midspan & Release $P = P_o$		0.4 ℓ-Service Load $P = P$	
	f_b	f_t	f_b	f_t	f_b	f_t
P/A	$\frac{P_o}{A} = +908$	+908	+ 908	+ 908	+ 805	+ 805
P_e/z	$\frac{P_{oe}}{Z_b} = +1107$	-442	+4061	-1621	+3078	-1229
M_{dl}/z			-2511	+1003	-2410	+ 963
M_{sdl}/z					- 461	+ 184
M_{ll}/z					-1153	+ 461
Stresses	+2015	+466	+2458	+ 290	- 141	+1184
Allowable Stresses (Psi)	$0.6 f'_{ci}$ +2100 OK	$3\sqrt{f'_{ci}}$ -177 OK	$0.60 f'_{ci}$ +2100 HIGH	$3\sqrt{f'_{ci}}$ - 177 OK	$6\sqrt{f'_c}$ - 429 OK	$.45f'_c$ +2250 OK

Check Ultimate Capacity @ .4 ℓ

$$\begin{aligned} \text{Actual } M_u &= 1.4(M_{dl} + M_{sdl}) + 1.7(M_{ll}) \\ &= 1.4(246 + 47) + 1.7(117.6) = 610 \text{ k'} \end{aligned}$$

$$\begin{aligned} d @ 0.4 \ell &= e @ .4 \ell = e @ .4 \ell + y \text{ from top} \\ &= 11.66" + 6.85" = 18.51" \end{aligned}$$

Using Table 5.2.1 of the PCI Design Handbook which is a design aid for determining the ultimate moment capacity for bonded prestress steel.

$$\omega_p = \frac{A_{ps}}{bd} \frac{f_{pu}}{f'_c} = \frac{2.142(270)}{(96)(18.51)(5)} = .0651$$

Interpolation of table gives $K_u = 272$.

$$M_u = \frac{K_u b d^2}{12000} = \frac{272(96)(18.51)^2}{12000} = 746 \text{ ft-kips}$$

$$746 > 610$$

Check Shear

$$\begin{aligned} v_u &= \frac{\ell}{2} [1.4(W_{dl} + W_{sdl}) + 1.7(W_{ll})] \\ &= \frac{70}{2} [1.4(418 + 80) + 1.7(200)] = 36.3 \text{ kips} \end{aligned}$$

$$v_u = \frac{V_u}{\phi b_w d} = \frac{36300}{.85(9.5)(08)(24)} = 234 \text{ psi}$$

For v_u at support less than 250 psi, minimum web reinforcement is adequate.

$$234 < 250 \quad \text{O.K.}$$

Camber and deflection should be investigated for this section. A second trial modifying the eccentricity to allow a lower f_{ci} could be undertaken. Or since the stresses and the ultimate moments are low, fewer strands could be tried.

II. Design An Inverted Tee Beam To Support The Double Tee Roof Members

$$\text{Span} = 40', \text{ Live Load } (25 \text{ psf on roof})(70) = 1.750 \text{ k/ft.}$$

$$\text{Dead Load, Double tee beams} = \frac{(418 \text{ lb/ft.})(70)}{8'} = 3.658 \text{ k/ft.}$$

$$\text{D. L. on roof} = (10 \text{ psf})(70') = \underline{.70 \text{ k/ft.}}$$

$$\text{Total load} = 6.108 \text{ k/ft.}$$

Select Section

From P.C.I. Design Handbook a 24IT52 (24" wide, 52" high), can handle a safe superimposed load of 6.796 f/ft. This is controlled by ultimate strength with 800 psi top tension allowed, therefore mild reinforcement would be required in the top.

III. Design of Double Tee Wall Panels

$$l_u = 16' - 0''$$

Braced against sidesway, $k = 1.0$

$$f'_c = 5000 \text{ psi}$$

$$\text{Live Load} = (418 + 80 + 200) \left(\frac{70}{2}\right) \left(\frac{1}{8}\right) = 3.054 \text{ k/ft.}$$

Superimposed Dead Load = 1.0 k/ft.

Total Load = 4.054 kips/ft.

Select Section

From Rocky Mountain Prestress load tables for $l_u = 16'0''$ an 8DT10+2 has an allowable safe working load of 5.25 kips/lin. ft.

A flexural analysis of the wall panel should be made using a wind loading applicable to the geographic location (80 mph for Kansas), analyzed as a simply supported beam from the roof members to the foundation. The information from Rocky Mountain Prestress indicates that wind load moments are rarely critical but should be investigated.

The prestressing steel would be designed for the wind load moment or the stresses occurred during handling. The columns' ultimate strength should be investigated, as should the bearing stresses on the corbels.

Corbels should be designed in accordance with the method shown in the connection chapter.

IV. Design of the Interior Columns

$$l_u = 16' - 0''$$

Braced against sidesway $\therefore k = 1.0$

$$f'_c = 5000 \text{ psi}$$

Try a 14" x 14" column

$$\text{Live Load} = 1.75 \times 40 = 70 \text{ kips}$$

$$\text{Dead Load} = 4.358 \times 40 = 174 \text{ kips}$$

$$P_u = 1.7(70) + 1.4(174) = 363 \text{ kips}$$

$$e = .1 \times 14 = 1.4 \text{ in.}$$

$$M_u = 1.4(363) = 42 \text{ ft. - kips.}$$

$$\frac{k l_u}{r} = \frac{16 \times 12}{.3 \times 14} = 46 > 22 \therefore \text{slenderness must be considered}$$

$$E_c = 5700\sqrt{5000} = 4.03 \times 10^6 \text{ in}^4$$

$$I_g = 14^4/12 = 3201 \text{ in}^4$$

$$EI = (4.03 \times 10^6)(3201)/2.5 = 5.16 \times 10^9 \quad (\beta_d = 1.0)$$

$$P_{cr} = \frac{\pi^2(5.16 \times 10^9)}{(16 \times 12)^2} = 1376 \text{ kips}$$

Using $C_m = 1.0$

$$\delta = \frac{1.0}{1.363/.7(1376)} = 1.6$$

$$M_u = 1.6(42) = 67 \text{ Ft. - Kips.}$$

From the column load tables of the PCI Design Handbook;

14" x 14", 4 - 7/16" dia. strands ($f_{pu} = 250,000 \text{ psi}$)

$$P_u = 380 \text{ kips}; M_u = 95 \text{ kips} \therefore \text{OK}$$

Use 14" x 14" column, 4 7/16" 250 k strands

V. Framing Considerations

The lateral loads in this warehouse building are to be resisted by the double tee wall members. After the roof members and wall members are in place, a composite slab is poured,

making the roof members a diaphragm. This diaphragm takes the loading from the lateral forces and transmits the loadings to the transverse walls. Typical connections details of the roof to wall members and the wall members to the foundation are shown in Figures 11a and 11b, respectively.

CHAPTER 8 - CONCLUSION

I have tried to give a broad view of precast concrete construction, why it came about, advantages, disadvantages, and engineering problems. Typically in concrete construction, the structural engineer designed the size of the member, the amount of reinforcing steel, and the mix of the concrete itself. In precast concrete construction, the structural engineer only analyzes the loading on the individual members, and the precast producer must design the beam to resist the particular loading. In other words, the integrity of the actual member is now in the hands of the producer, as in steel construction.

Precast concrete eventually will reach the handbook design stage. The P.C.I. Design Handbook has pioneered this effort with examples of load tables and the use of them. Complete standardization of the elements will have to be achieved before the precast handbook can approach the steel handbook stage. As of now, minor variations of sections throughout the country occur which limit the handbook and make its use only a guide to the structural engineer. The specialty shops will continue production in large amounts. Segmental construction has just started in the United States and will continue to grow and be used in the long span bridges. Much can be learned about segmental construction from Europeans and Australians, who are far ahead of American engineers in segmental bridges.

True building systems, as such, are actually few and far between. The Dyna-Frame Structural System is one which is innovative and completely standardized. The Versa-space or Vantage-space type systems are really just a marketing idea for the standard elements. Any precast facility actually does this same thing, or could do this if given freedom from the architect, engineer, and owner.

Precast concrete is here to stay and will continue to grow as labor prices increase. The number of plants operating today make the precast elements accessible to every part of the country. Since the overhead is high, this growth must be well planned and interaction between all the disciplines involved, structural engineers, producers, and architects, must take place for this growth to be beneficial to all.

APPENDIX - NOTATION

- a = Shear span, distance between concentrated load and face of support
 a' = Depth of equivalent rectangular stress block
 A = Cross sectional area
 A_1 = Area loaded by bearing stress
 A_2 = Supporting surface area
 A_f = Area of non-prestressed reinforcement to resist flexure
 A_{ps} = Area of prestressed reinforcement.
 A_s = Total area of tension reinforcement
 A_t = Area of reinforcement necessary to resist the horizontal force, N_u
 A_{vf} = Area of shear friction reinforcement
 b = width of compression face of member
 b_w = Web width
 c = Coefficient from design aids
 C_m = A factor relating the actual moment diagram
 D = Dead load
 d = Distance from extreme compression fiber to centroid of tension reinforcement
 E_c = Modulus of elasticity of concrete
 E_{ci} = Modulus of elasticity of concrete at time of initial prestress
 e = Eccentricity of prestress force parallel to axis measured from the centroid of the section
 e' = Distance between c.g. of strand at end and c.g. of strand at lowest point, $e_c - e_e$
 e_c = Eccentricity of prestress force from the centroid of the section at the center of the span

- e_e = Eccentricity of prestress force from the centroid of the section at the end of the span
 f_b = Stress in the bottom fiber of the cross section
 f'_c = Specified compressive strength of concrete
 f'_{ci} = Compressive strength of concrete at time of initial prestress
 f_{pu} = Ultimate strength of prestressing steel
 f_{se} = Effective stress in prestressing steel after losses
 f_{si} = Initial or tensioning stress in prestressing steel
 f_t = Stress in the top fiber of the cross section
 f_y = Specified yield strength of non-prestressed reinforcement
 f_{yv} = Specified yield strength of shear reinforcement
 f_t = Stress in the top fiber of the cross section
 h = Height of cut out in dapped-ended beam
 I = Moment of inertia
 k = Effective length factor for compression members
 K_u = Design aid coefficient
 ℓ = Length of span
 ℓ_u = Unsupported length of compression member
 L = Live load
 M = Service load moment
 M_{dl} = Moment due to service dead load
 M_{ll} = Moment due to service live load
 M_{sdl} = Moment due to superimposed dead load
 M_u = Ultimate moment
 N_u = Ultimate tensile force

P = Prestress force after losses

P_i = Initial prestressing force

P_o = Prestress force at transfer

r = Radius of gyration of the cross section of a compression member

v_u = Ultimate shear stress

V_u = Ultimate shear force at section

Y_b = Distance from bottom fiber to center of gravity of section

Y_t = Distance from top fiber to center of gravity of section

Z_b = Section modulus with respect to the bottom fiber of a cross section

Z_t = Section modulus with respect to the top fiber of a cross section

β_b = Ratio of maximum design dead load moment to maximum design total load moment

δ = Moment magnification factor for compression members

μ = Shear friction coefficient

ρ = Ratio of non-prestressed tension reinforcement

ρ_p = Ratio of prestressed reinforcement

ϕ = Capacity reduction factor

$\bar{\omega}_p$ = Design aid coefficient, $\rho_p f_{pu}/f'_c$

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PRECAST-PRESTRESSED BUILDING
SYSTEMS AND ELEMENTS

by

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ABSTRACT

A brief overview of precast concrete building systems and elements is presented. The advent of precast concrete construction was caused by increased labor expenses and the demand for lower tolerances on concrete work. Particular systems have been developed and standard elements are also used in building systems. Several commercially available systems are presented and reviewed.

Connection design is of the utmost importance and considerations for connection design should be given to production, field erection, and the force interactions on the connections. Design methods commonly used for connections are presented.

An example of a single-story warehouse building, using standard elements, is included. The example utilizes published literature from producers and other sources for handbook design procedures.