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# ATLSS Precast Concrete Systems: Review of Existing Systems and New Conceptual Developments

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ADVANCED TECHNOLOGY FOR LARGE STRUCTURAL SYSTEMS

Lehigh University

# ATLSS PRECAST CONCRETE SYSTEMS:

# REVIEW OF EXISTING SYSTEMS AND NEW CONCEPTUAL DEVELOPMENTS

by


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Any opinions, findings, and conclusions expressed in this report are solely those of the authors. They do not necessarily reflect the view of the sponsors or those acknowledged above.

# ABSTRACT

This report presents initial results from research on ATLSS Precast Concrete (APC) systems, conducted at the Center for Advanced Technology for Large Structural Systems (ATLSS), at Lehigh University. The APC systems are primarily for, but not limited to, low- to mid-rise buildings in moderate seismic zones.

Existing and emerging precast concrete systems are reviewed, particularly ones capable of providing seismic resistance and of allowing construction features and performance that are favorable compared to CIP concrete systems. New APC systems are then conceptually developed with these same criteria. The proposed systems contain two innovative beam-column connection schemes, namely, the ATLSS beam-to-column and the ATLSS beam-to-beam connections. The beam-to-column connections are separated into two types: the Modified ATLSS Passage-Type (MAPT) and the ATLSS Nodal-Type (ANT) connections. The beam-to-beam connections are optional and are utilized when transportation constraints prevent casting the precast beams to the required length. Both two- and three-dimensional connection systems are proposed, and the construction sequences demonstrated.

This report describes Part I of the ATLSS research on APC systems. Design methodologies and experimental evaluations for the systems and their seismic resistance are being described in subsequent reports.

# 1. INTRODUCTION

## 1.1 Background

Precast concrete members with simple (shear) connections have long provided many advantages over cast-in-place (CIP) concrete members. Structures incorporating precast concrete members with simple connections generally have a much faster erection time while maintaining a performance comparable to CIP concrete structures. Furthermore, they have often proved to be more economical than CIP structures. The economy of precast structures results from the higher degree of quality assurance, a reduction in on-site formwork and labor, and the use of precast systems that have a "built-in" self-supporting ability. The latter eliminates the need for most of the temporary supports.

Precast concrete members with moment connections, however, do not seem to have clear advantages over CIP concrete as do the precast members with simple connections. Structures incorporating precast members with moment connections must be designed to ensure proper moment and shear transfer between members. The situation is compounded when structures are required to resist lateral loads, especially seismic loads. The nature of such loads requires that the structures be capable of sustaining load reversals, as well as possess both the required strength and ductility to survive extreme seismic events. The latter demands are tied directly to the strength and ductility of the critical sections which are often located at the member interfaces.

Many precast concrete moment connections constructed in the past have, unfortunately, been unable to show such desired properties. Consequently, many seismic resistant precast concrete systems opt for the use of CIP construction to establish the moment connections between their precast components. While such cast-in-place construction makes continuity of the reinforcement possible at the connections, which is necessary if a connection is to behave in a ductile manner; it slows down the erection process of the structures and requires temporary supports for the connecting precast components. Structures incorporating precast concrete with moment connections in this situation are not much more competitive than their CIP counterparts. There is therefore a need to develop new precast connections that do not rely on CIP construction to render the joints monolithic for moment resistance.

## 1.2 Review of Seismic Resistant Precast Concrete Systems

In general, seismic resistant precast concrete systems can be separated into two categories, depending on the design philosophy adopted for the connections of the precast components. The first category contains precast connections called *ductile* connections, whereas the second uses *strong* connections. Ductile connections are detailed to be weaker than the connecting precast members. They are intended to be the locations of ductile inelastic deformation. If the precast members are designed with adequate strength margin over that of the connections, the precast members will remain elastic throughout the entire seismic response.

Strong precast connections, on the other hand, are detailed to be effectively rigid and stronger than the designated locations of inelastic action (plastic hinges). In this case, if the connection detail is successful, the precast structures can be designed according to existing design codes, and they can be expected to behave in a manner similar to CIP structures. Unless indicated otherwise, the seismic resistance systems and sub-systems described herein have strong connections.

Parry [1-1] presented a concept of an innovative precast concrete moment connection (Fig. 1-1), which will be referred to hereafter as the ATLSS Passage-Type connection. The connection has a potential to be used in an automated construction environment with an advanced crane system such as the Stewart platform (Fig. 1-2). The Stewart platform [1-2] is a future crane system capable of providing stable and accurate six-degree-of-freedom motion control over crane movement.

The ATLSS Passage-Type connection was intended for use in moderate seismic zones. Pilot experiments involving three beam specimens were conducted to study the strength and ductility of the beam part under cyclic loading. The truss modeling concept and the conventional ACI 318 code [1-3] were used in the design of the specimens. The experimental results indicated a successful connection design method. The specimens displayed the strength and ductility required to resist cyclic loads, and they were loaded well into inelastic regions without significant degradation in the load-carrying capacity.

One possible disadvantage of the ATLSS Passage-Type connection is that the strength and stiffness of the column, in the plane of loading, may be inferior to the traditional CIP concrete columns. This is because the column's longitudinal rebars must be discontinued at the passage area for the precast beam to pass through. Furthermore, without an advanced crane system like the Stewart platform, the connection cannot be easily assembled in the field.

Seckin and Fu [1-4] presented a precast interior beam-column connection (Fig. 1-3), which involved the use of steel plates attached to the rebars of the beams and the column. The continuity of the reinforcement was achieved by welding the steel plates together at the interfaces between the beams and the column. An experimental program involving three specimens with this configuration and one monolithic beam-column connection was conducted to study their behavior under cyclic loads. It was reported that two of the precast specimens failed prematurely due to fracture of the weld at the connections. The other specimen, however, achieved a performance comparable to the monolithic specimen and failed by rupture of the reinforcing bars at the weld details between the bars and the steel plates at the beam ends.

Seckin and Fu's connection exhibited potential strength and ductility as required for use in seismic zones. The erection of the connection involves the use of field welding to establish the proper force transfer between the precast beams and column at every connection, and special weldable rebars must be used in the precast beams and columns.

Mast [1-5] proposed a precast concrete system for mid-rise structures such as office buildings and parking garages in severe seismic zones. The system relies on perimeter precast columns and spandrel beams to resist wind and seismic loads (Fig. 1-4), and on the interior frames to resist gravity loads. The perimeter columns are designed using the same requirements as for the columns in a CIP ductile frame. The spandrel beams, on the other hand, contain diagonal mild steel flat bars which are intentionally debonded from concrete so that they may yield, either in tension or compression, throughout their length. The arrangement of the mild steel flat bars provides the spandrel beams with the required strength and toughness comparable to that of the beams of a CIP ductile frame. The concrete of the spandrel beams does not carry any loads except its self-weight, and it serves only as the lateral restraint against buckling for the flat bars. The connections between the spandrel beams and the precast columns are proprietary (see Fig. 1-5) and are designed to be sufficiently strong to force all the yielding into the flat bars of the spandrel beams. The flat bars of the spandrel beams are designated as the sole energy dissipation source for the structural system during an earthquake. A concept similar to Mast's was also adapted in the development of a bracing panel system in steel frames by Chen and Lu [1-5a].

With the use of the proprietary column-spandrel beam connections in conjunction with a fast paced gravity system, erection times of systems such as Mast's may be reduced significantly. However, the perimeter frames are not self-supporting and the spandrel beams are manually connected to the precast columns at the time of installation. Also, many precast structures with interior-gravity and perimeter-lateral load resisting systems did not perform well [1-6] during the January 1994 Northridge earthquake in California. Large deformations imposed on the structures initiated failures in the gravity systems which had limited ductility. To upgrade the performance of these systems, the overall stiffness of the systems should be increased. This can be achieved either by stiffening the perimeter-lateral systems and/or by including shear walls in the structures. In addition, the inelastic deformation capabilities of the interior-gravity systems and the diaphragms should be increased.

French et al. [1-7 to 1-9] tested six exterior and one interior precast beam-column connections under cyclic loading at the University of Minnesota. The connection details varied from (1) a post-tensioning connection using two post-tensioning bars, (2) a connection with four threaded rebars, (3) a composite connection using CIP topping and a precast beam connected with a post-tensioning bar, (4) a welded connection, (5) a bolted connection, (6) a connection with four rebars threaded into column-anchored couplers, and (7) an interior connection similar to (6) but with the use of tapered-threaded splices. These connections are identified in Fig. 1-6 as connections BMA to BMG.

Based on the connection design philosophy discussed earlier, the BMA to BMD connections can be classified as strong connections with their plastic hinges relocated to a distance 35 in. away from the column faces. The hinging location of the BMC specimen was at 24 in. away from column face when subjected to positive bending (due to upward shear force at the beam end), and at 35 in. when subjected to negative bending. The BME to BMG connections, on the other hand, were ductile connections with plastic hinges forming at the connection area.

All the connections were designed for severe seismic regions (UBC Zones 3 and 4). The precast beams were partially prestressed. The test results indicated that the specimens with ductile connections exhibited better energy dissipation characteristics than the specimens with strong connections. Furthermore, specimens with BMA to BMD connections achieved interstory drifts of at least 3.3%, whereas specimens with BME to BMG connections achieved interstory drifts greater than 4%. It was concluded that the composite connection (BMC) and the threaded rebar connection with the tapered splices (BMG) appeared to be good candidates for use in seismically active zones.

The work at the University of Minnesota provided precast simple connections with moment resistant capacities. This approach could result in precast concrete systems with improved erection time together with the necessary strength and ductility for use in seismically active regions. These connections, however, require tight tolerances and precise alignment controls which may have an adverse impact on the erection process.

At the time of this review, researchers at the National Institute of Standards and Technology (NIST) have tested four monolithic, and twenty precast post-tensioned beam-column connections [1-10 to 1-14]. The precast post-tensioned connections were of the ductile type. The NIST study is to develop guidelines for the design of an economic precast moment-resisting beam-column connection which utilizes post-tensioning reinforcement, suitable for use in regions of high seismic risk. Figure 1-7 shows a beam-column configuration determined by NIST as suitable for further investigation. The connection utilizes mild steel to transfer moment and post-tensioning steel to transfer shear.

The performance of this NIST precast connection was superior to that of the monolithic connections in many ways. The precast connection suffered lesser strength degradation under cyclic loading and had larger interstory drift capacity. With regard to the latter, the connection could undergo 6% drift while maintaining 55% of its maximum strength. In addition, the precast connection dissipated more energy per cycle than the monolithic connection, for drifts of up to 1.5%. Thereafter, the connection dissipated energy at a level of approximately 75% of the energy dissipation of the monolithic connection [1-14].

NIST concluded that its proposed precast post-tensioned connection is a viable solution for precast concrete structures in high seismic regions. However, it was noted by Stone et al. in Ref. [1-14] that due to the stringent requirements of the UBC code [1-15], the connection may not comply with the UBC requirements for reinforced concrete *special moment-resisting frames* in several areas. Specifically, (1) the connection uses reinforcement with a yield strength, 78 ksi, higher than that permitted by UBC Section 2625c-6; (2) the beams include bar splices located within two beam depths from the column face; and (3) the transverse reinforcement needed in the beams and column joint may be less than specified. The proposed connection, therefore, must be used only as a part of structures under the undefined structural system category. According to the UBC code, an undefined structural system must demonstrate a lateral force resistance and energy absorption capacity comparable to that of conventional systems. However, as mentioned earlier, at interstory drifts greater than 1.5%, the proposed connection dissipated only about 75% of the energy dissipated by the monolithic connection. This issue is to be addressed in future research.

Dolan et al. [1-16 and 1-17] at the University of Washington tested eight moment and eight simple precast connections selected from the PCI connection manual [1-18]. The study was sponsored by PCI, and the objective was to identify economical and competitive methods in designing precast concrete connections. Of the total of sixteen tests, only the results of the tests on the moment connections were applicable for seismic design.

Figure 1-8 shows the configurations of the eight moment connections tested at the University of Washington. These connections were (1) a welded connection (shown in the figure as connection BC15), (2) a combination of CIP topping and a welded connection (BC16A), (3) a bolted beam-column connection (BC25), (4) a bolted column-to-column connection (CC1), (5) a precast beam constructed into a CIP column (BC26), (6) a post-tensioned connection (BC27), (7) a connection grouted or partially grouted to dowels (BC28 and BC29), and (8) a composite connection consisting of a precast U-beam shell filled with CIP concrete using threaded bars as a means of attachment (BC99).

Experimental results indicated that the most promising connection was the BC26 connection, and the dowel connections (BC28 and B29) should not be classified as moment connections. Except for the BC26 connections, the rest of the connections exhibited low energy dissipation capacity. No definite conclusions were made since only one specimen of each type was studied.

Park et al. [1-19 to 1-21] summarized the state-of-the-art precast concrete systems in New Zealand. It was stated in the references that the majority of floors in New Zealand buildings are now constructed of precast concrete units spanning one way between beams or walls. In addition, most precast concrete systems in New Zealand aim at emulating monolithic behavior, often referred to as the emulation design approach, and they normally employ CIP concrete to establish continuity between precast members.

Figure 1-9 shows three precast arrangements commonly found in New Zealand. System 1 involves the use of "semi-precast" beam units and CIP concrete columns. The precast beams are seated on the ledges, provided by the concrete cover of the CIP columns. Then precast floor units are placed on the top of the precast beams and reinforcement is placed longitudinally on the beams, in the beam-column joint core, and on the precast floor units. Lastly, CIP topping is placed to complete a CIP-emulated floor.

System 2 makes more extensive use of precast concrete and avoids placement of concrete in the congested beam-column joint regions. The precast concrete beams span between the mid-lengths of two adjacent spans, and thus include in them the complex joint-core reinforcement details. The precast beams are placed on the concrete columns which can be either CIP or precast, and the longitudinal column bars pass through and protrude above the beams via pre-embedded corrugated metal ducts. The diameter of the metal ducts is usually two or three times larger than the diameter of the column reinforcement, in order to minimize potential tolerance and alignment problems.

To establish proper bonding between the beams and the columns, the metal ducts are grouted. Then the precast beams are joined at the mid-span with CIP connections. Precast floor units are seated on the beams, and later reinforcement is placed on the beams and floor units. Cast-in-place topping completes the floor. Alternatively, the total depth of the beams can be precast, and the floor units are supported on ledges provided on the sides of the beams. Figure 1-10 shows a 22-story building constructed using System 2.

System 3 also utilizes precast concrete extensively and minimizes on-site reinforcement fabrication, just like System 2. The system employs precast tree-shaped units which bundle columns and beams together. The connection between each "tree" occurs at the beam's mid-span and either at the column base or at the column's mid-height. Commonly CIP concrete is used to establish the beam connections, and mechanical splices are utilized for the column connections. It was noted by Park in Ref. [1-20] that due to their relatively large dimensions, the precast components of System 3 can be very heavy and difficult to transport. Special attention, therefore, should be given to crane capacity and the transportation method when considering using System 3. Figure 1-11 shows a building constructed using System 3 with tree units spanning a two-story height.

The New Zealand systems use precast concrete quite extensively and yet are still able to emulate CIP behavior by using CIP concrete to establish continuity between precast members. Furthermore, except at the beam-to-column connections of System 1 and at the beam-to-beam connections of Systems 2 and 3, the use of CIP concrete does not interfere with the erection process. This permits the New Zealand systems to be constructed at a pace faster than most CIP concrete systems

Yoshizaki et al. [1-22] summarized the state-of-the-art practice of precast seismic structural systems in Japan. The primary goals of precast concrete systems in Japan are to emulate the behavior of the conventional CIP concrete structures, reduce on-site labor, and shorten erection time. The Japanese precast construction utilizes a significant amount of CIP concrete, but does not require extensive formwork. Sometimes it is used in combination with conventional methods, partly due to the local legal requirement [1-22, 23].

Typical precast frame systems in Japan can be divided into two categories: "column-tree" systems and single-member systems. Column-tree systems normally use two dimensional precast members such as those denoted as Types A and B in Fig. 1-12. In these systems, the column-tree components are joined together at the mid-span and at the mid-story height where stresses are small. Due to manufacturing and transportation difficulties, however, these column-tree systems are not widely used.

In single-member systems, all the precast components are made separately and are connected together or to CIP components using CIP concrete. Beam components are normally precast, whereas column components are either precast or CIP. In some cases, beam and column shells (see Fig. 1-13) are used. These shells have dual functions: one, they form parts of the structural components and two, they also serve as forms for CIP concrete. The latter, therefore, eliminates the need for on-site formwork. Typical configurations of precast members in these systems are shown as Types C to F in Fig. 1-12.

For both column-tree and single member systems, semi-precast composite slabs are used in conjunction with CIP concrete to construct the floor units. This is done to eliminate the need for formwork and temporary supports for floor construction. Shear walls, if required in the building systems, are mostly made of CIP concrete to avoid transportation and connection difficulties associated with large precast components.

Regarding the potential seismic performance of these systems, Japanese construction law requires that precast concrete systems demonstrate acceptable levels of strength and ductility before being used in place of CIP systems. A procedure for qualifying the precast systems based on unit and member testing methods was developed and outlined by the Building Center of Japan. As a result of this qualifying method, most precast systems being used in Japan should perform as well as their CIP counterparts. In order to satisfy this legal requirement, the use of large amounts of CIP concrete is common in the Japanese precast systems, as it is the most promising method of providing comparable CIP performance. It was stated by Kurose et al. [1-23] that because of this legal requirement, much of the research on precast concrete systems in Japan is funded by individual construction companies. The details of the Japanese precast systems, thus, vary according to the standard practice of each organization. This situation makes it difficult for Japanese officials to establish building codes that provide uniform treatment to precast concrete structures.

This survey of existing and emerging precast concrete systems concludes with a discussion of the ongoing PRESSS research program. PRESSS, Precast Seismic Structural Systems, is a very large-scale coordinated research program being conducted in the US, in parallel with a similar effort in Japan. The overall goals of the US-PRESSS program are to develop precast concrete systems suitable for use in seismic environments and to prepare seismic design recommendations necessary for incorporation into model building codes [1-24 and 1-25]. The emphases of the US-PRESSS program are on systems containing ductile connections, and the development of the necessary amendment to the current building codes. The latter is due to the likelihood that the US-PRESSS ductile connections would not conform with the current building code requirements.

Currently, the US-PRESSS program is pursuing research into three types of precast systems; namely, ductile frame systems, shear wall/bearing wall systems, and cladding systems [1-26]. Figure 1-14 shows a typical floor plan for the ductile frame systems. The frame systems utilize a ductile steel corbel in conjunction with a stressed bar system in forming a beam-column connection (Fig. 1-15). The ductile steel corbel resists both shear load and the tension at the beam bottom caused by seismic moment. The stressed bar system involves the use of a "dog-bone" type beam which has a deeper section at each end. The beam-column connections are intended to be ductile connections.

Figure 1-16 depicts a typical floor plan considered for shear wall/bearing wall systems. A hollow core floor system without topping is to resist both vertical and diaphragm shear loads. Both friction and welded horizontal shear transfer mechanisms are considered in the systems. Flexural loads in the systems are resisted by stressed bars, coupled walls and welded connections. Both flexural and shear mechanisms are considered for energy dissipation.

A typical elevation view of a structure using the cladding systems is shown in Fig. 1-17. These systems consist of perimeter non-load bearing walls and can be used with any interior gravity load resisting system. The cladding panels of the systems are to be designed to behave elastically under earthquakes, and the energy dissipation is provided by the panel-to-panel connectors.

Studies on the components of these systems are being carried out at the time of this review, [1-27 and 1-28]. Several alternatives are currently being considered, and the finalized versions of these systems are not yet available.

## 1.3 Objectives of Study

There are many existing and emerging precast concrete systems and sub-systems that have the potential to perform well in seismic active zones. Some of these systems and sub-systems incorporate certain construction features that are similar to precast concrete systems with simple connections. These features could permit such precast seismic systems to be built with shorter construction periods compared to CIP concrete systems. Moreover, the construction periods of these precast systems can possibly be reduced even further if temporary shoring can be minimized and certain features that would allow for automated construction can be added.

The earlier review indicates that much of the new research and development in precast seismic resistance areas is focused on systems with ductile connections. This is because many researchers feel that strong connections do not require further research attention [1-10, 1-24, and 1-25]. The authors, however, contend that the strong connection approach should not be ignored and more research is still needed, especially in the area of connection design methodologies. Currently, most available design methodologies for strong connections are empirical and applicable to only a few *typical* cases. Research focusing on rational and universal design approaches for strong connections is needed.

This report, the initial part of an overall study, describes the conceptual development of new lateral load-resistant precast concrete systems that incorporate some beneficial features of the existing precast concrete systems and have the potential to be adapted into an automated construction environment. The new systems utilize strong connections and have the strength and ductility required for seismic resistance. They are intended primarily for, but are not limited to, low- to mid-rise buildings in moderate seismic zones, since approximately 99% of new buildings constructed in the United States during the first half of the 1990s fall into this category, Table 1-1.

**Table 1-1 New Buildings Constructed in the United States between 1992 and 1995 [1-29]**

| Year | 1 Story | 2 to 4 Stories | 5 to 19 Stories | 20 Stories plus |
|---|---|---|---|---|
| **1992** | 60.7% | 27.8% | 9.9% | 1.6% |
| **1993** | 62.0% | 28.5% | 9.0% | 0.5% |
| **1994** | 64.3% | 27.0% | 7.8% | 1.0% |
| **1995 (1st Half)** | 65.2% | 26.3% | 7.4% | 1.2% |

**Remarks:** 1 to 4 stories = low-rise buildings
5 to 19 stories = mid-rise buildings
20 stories and higher = high-rise buildings

The objectives in later parts of the overall study are to develop and experimentally verify rational design methodologies for primary precast components and their connections, and to analytically assess the structural performance of the new systems subjected to earthquake ground excitation. Additional reports will describe these studies.

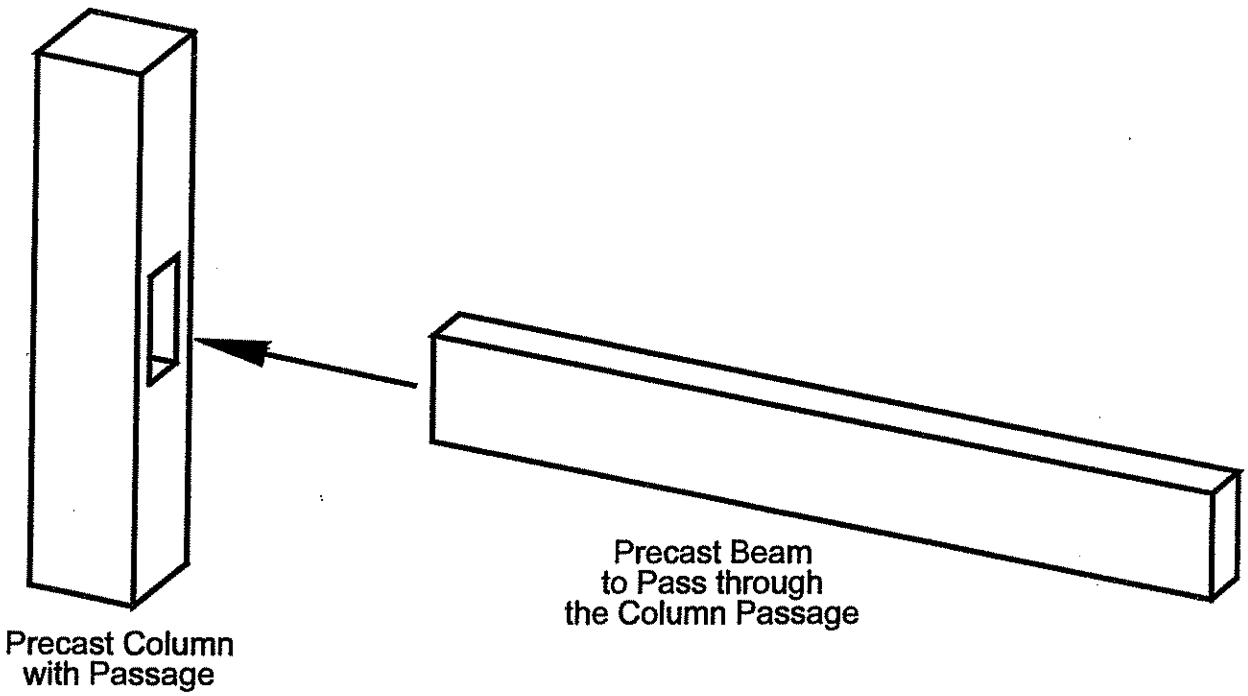



**Figure 1-1 ATLSS Passage-Type Connection**

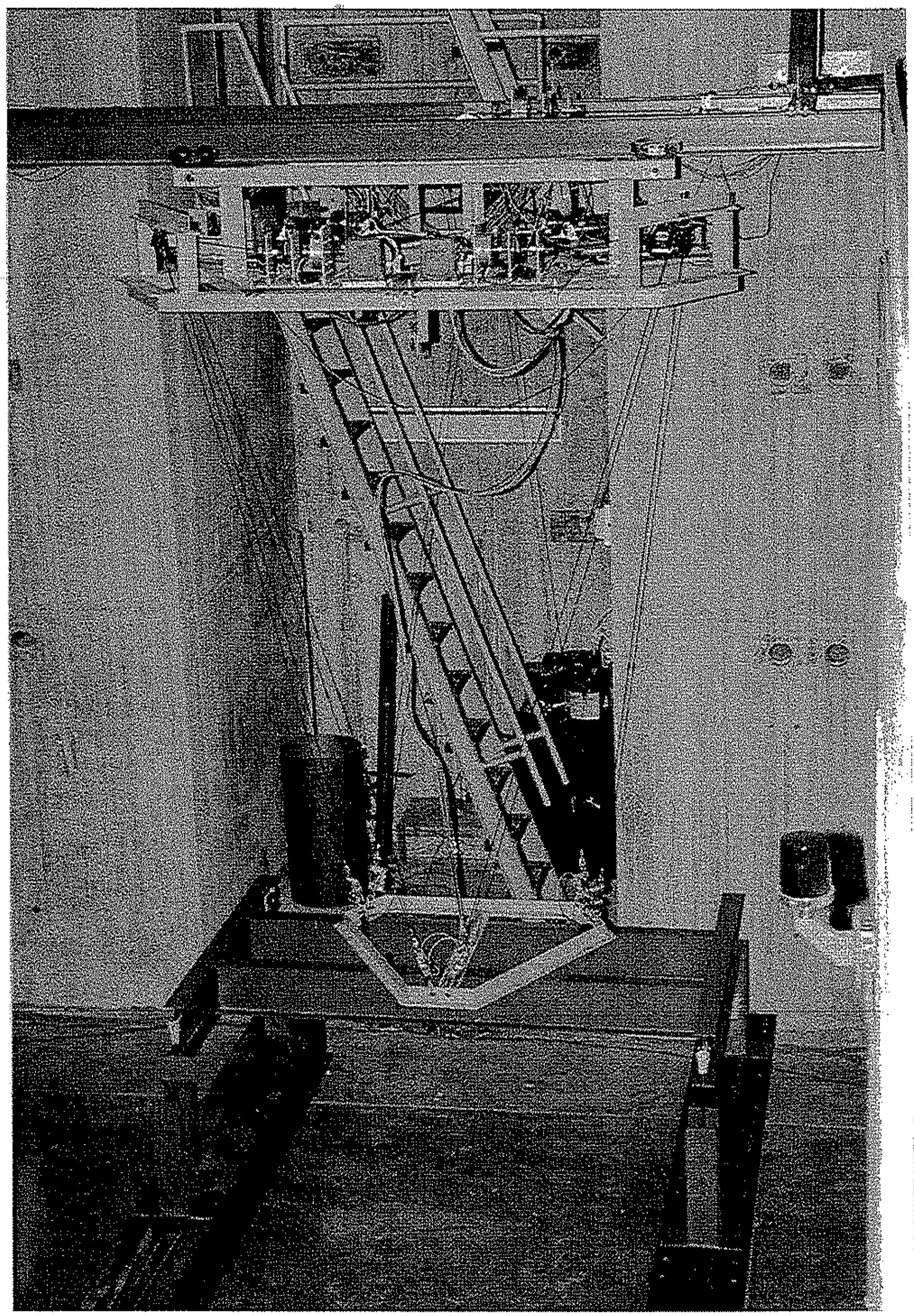
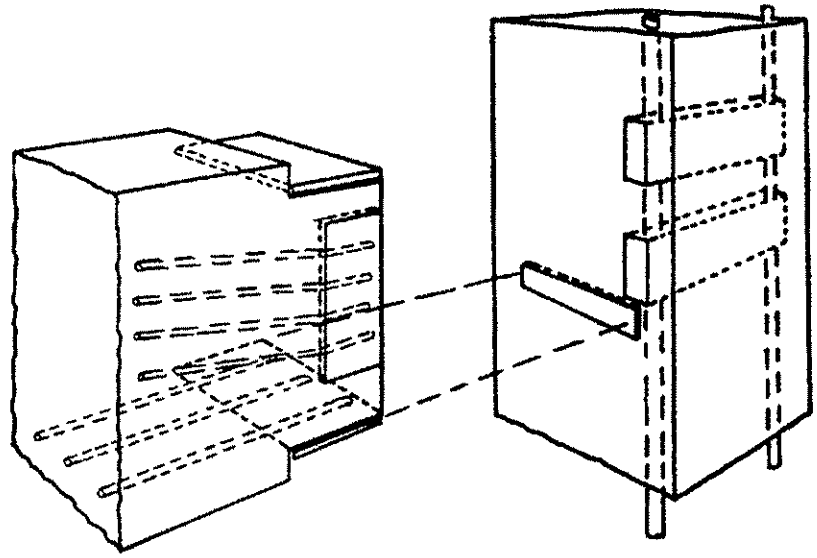
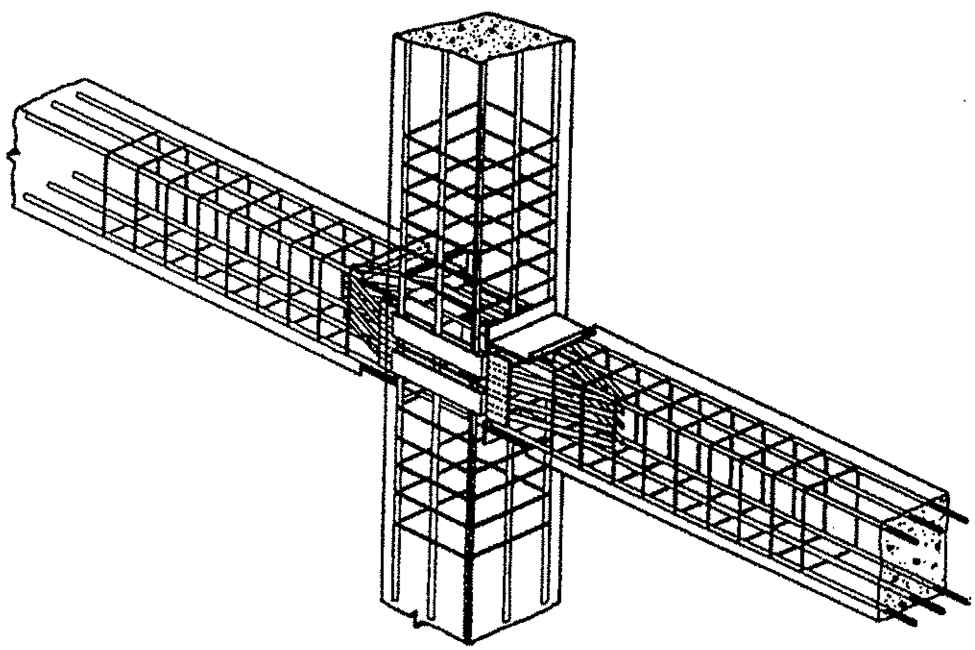


**Figure 1-2 Stewart Platform (Prototype) Shown Lowering Steel Panel into Place**



(a) Connection Details



(b) Precast Beam-Column Connection

Figure 1-3 Seckin and Fu's Connection (Taken from Ref. [1-4])

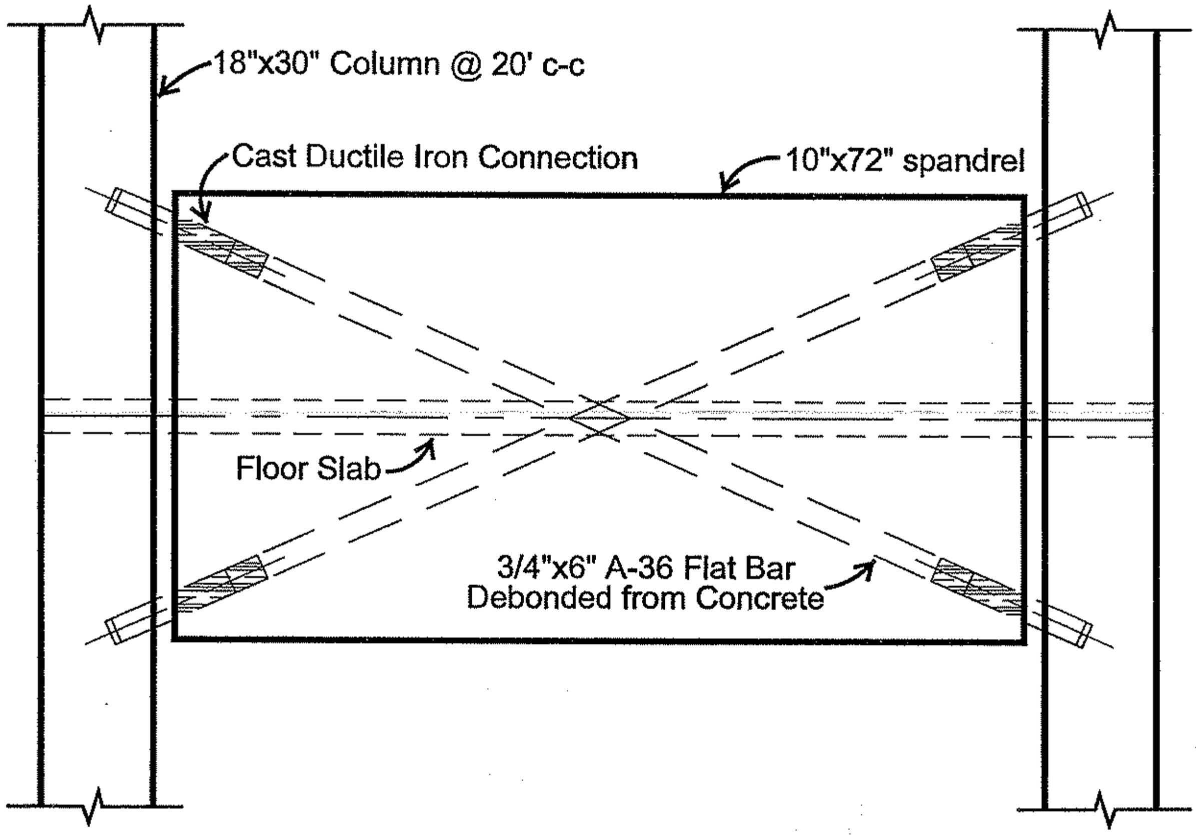
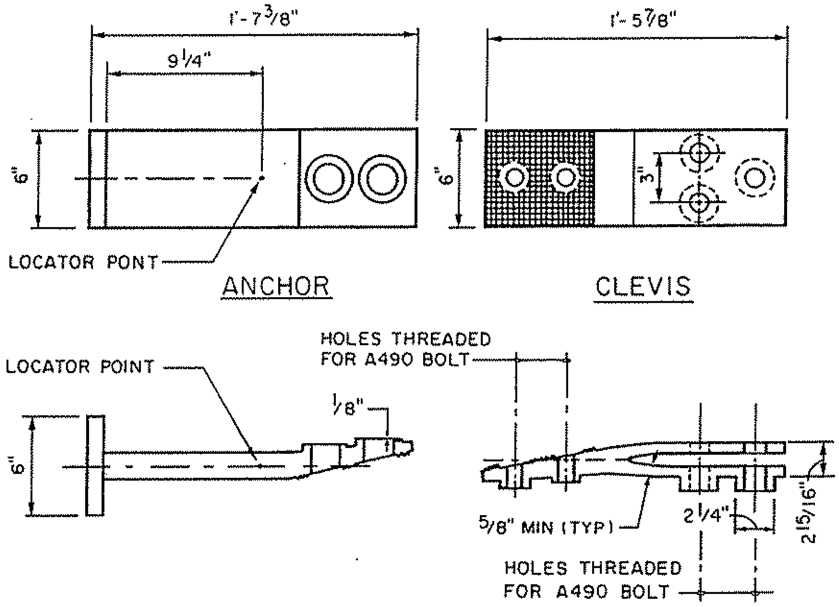



**Figure 1-4 Mast's Precast Seismic System (Adapted from Ref. [1-5])**



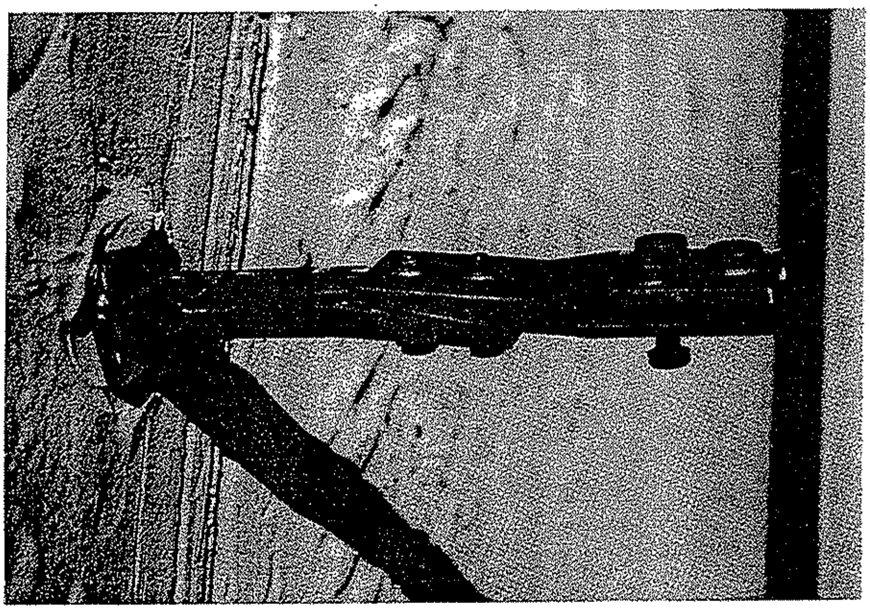



**Figure 1-5 Proprietary Spandrel-Column Connector Used in Mast's System**

**(Taken from Ref. [1-5])**

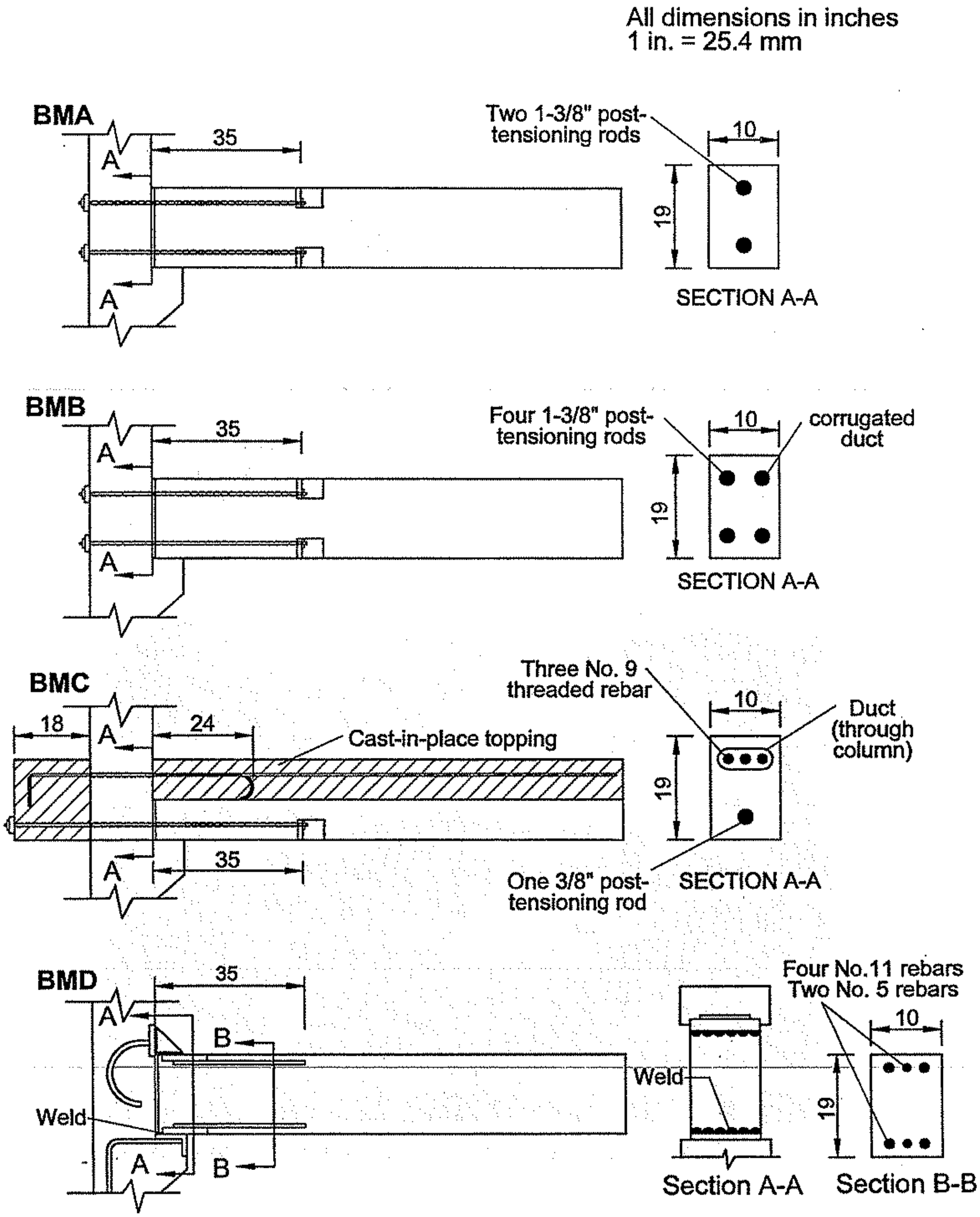



**Figure 1-6 Precast Connections Tested at University of Minnesota [1-7 and 1-8]**

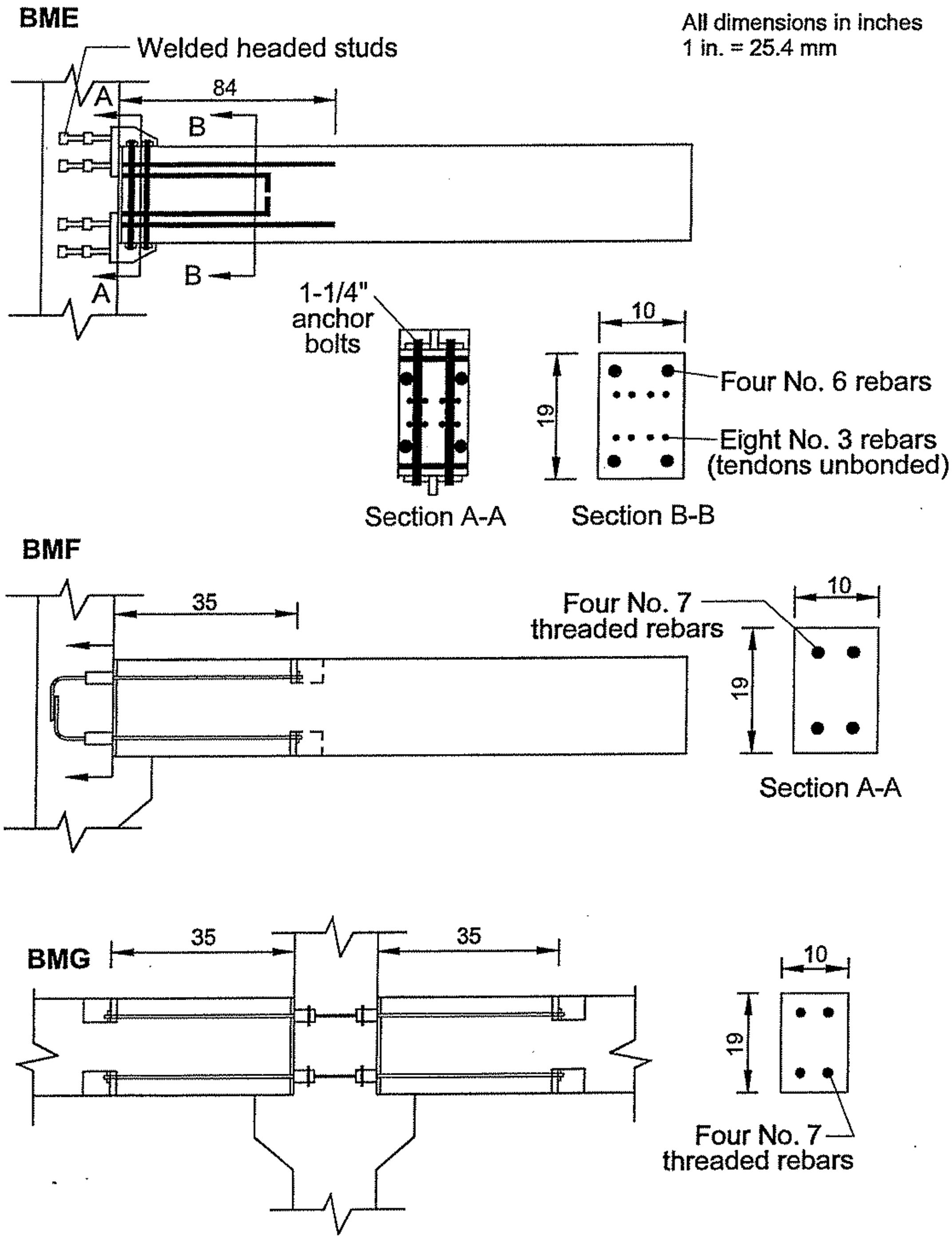



**Figure 1-6 (Continued)**

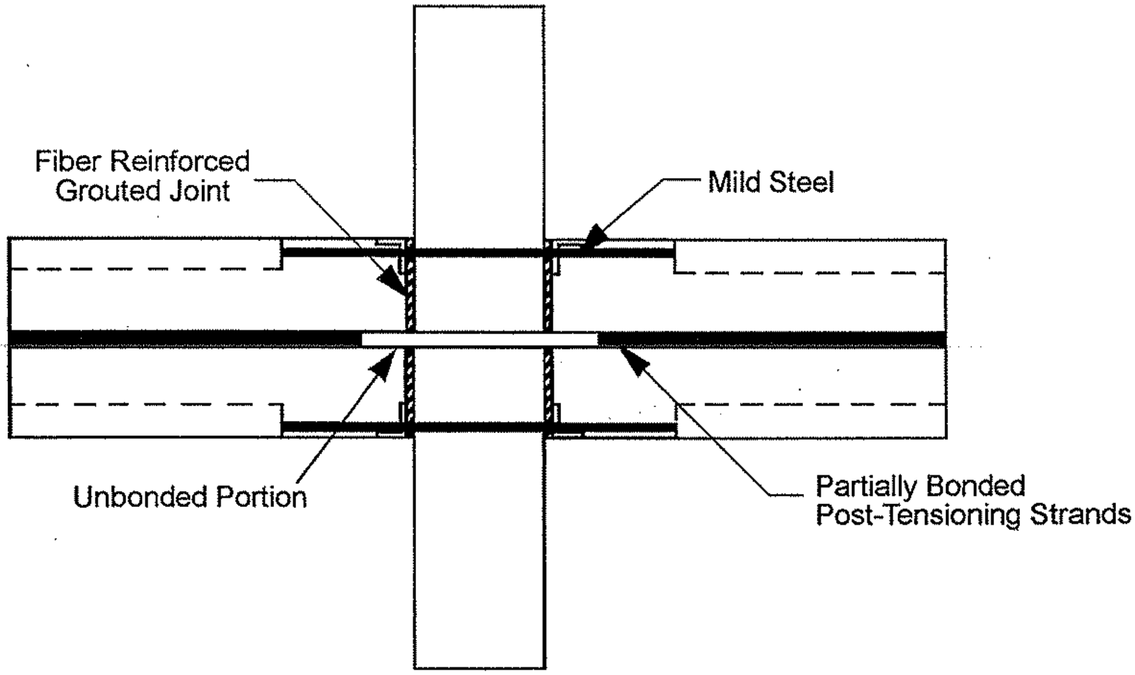



**Figure 1-7 NIST Precast Post-Tensioned Beam-Column Connection**

**(Adapted from Ref. [1-14])**

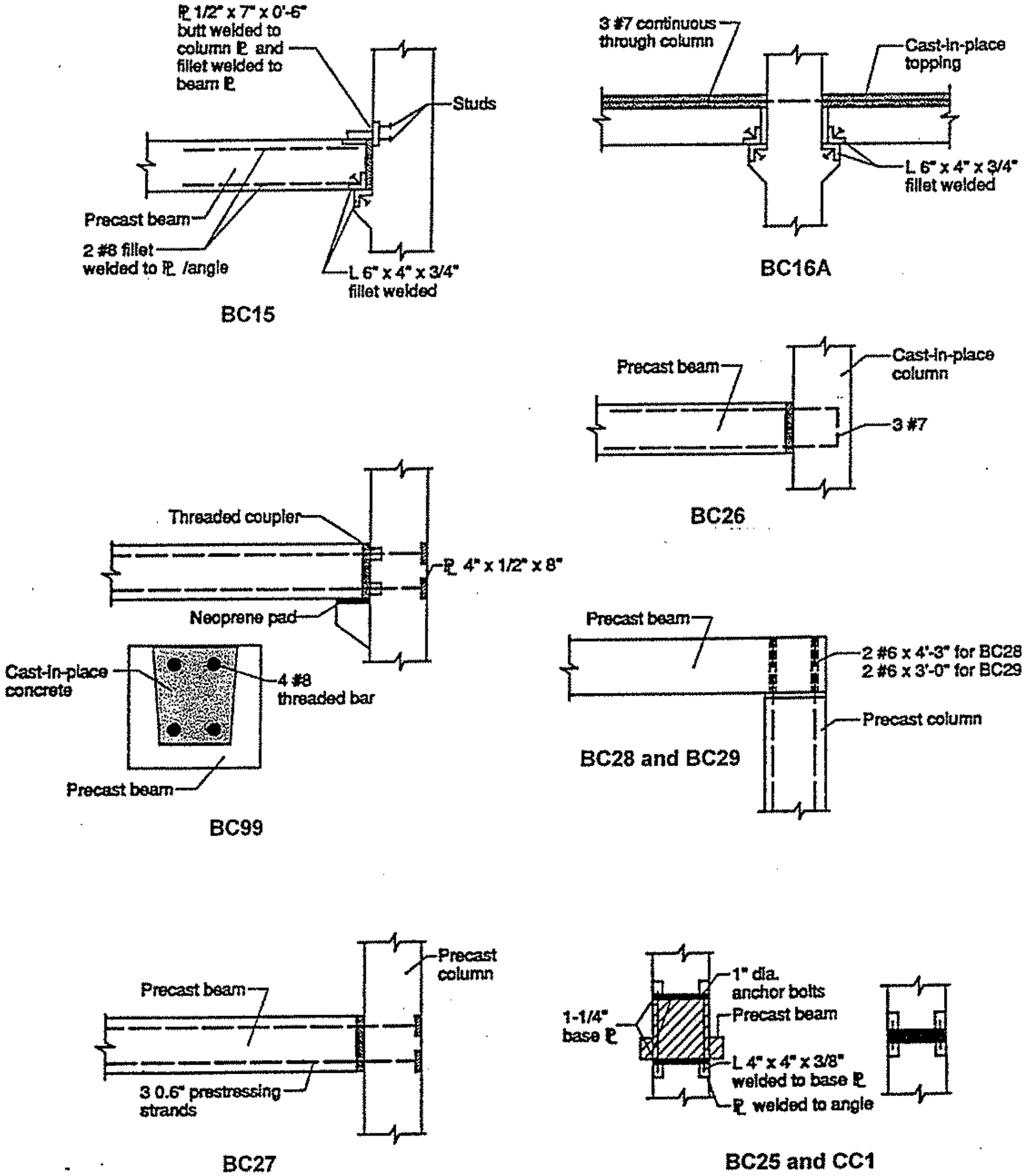
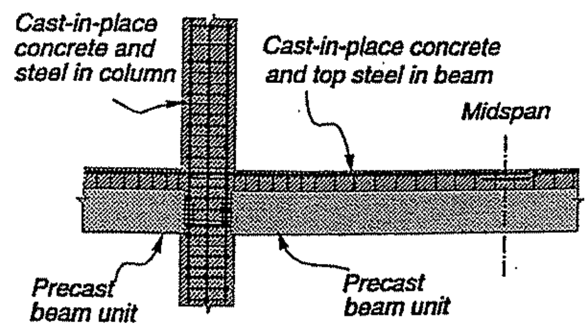
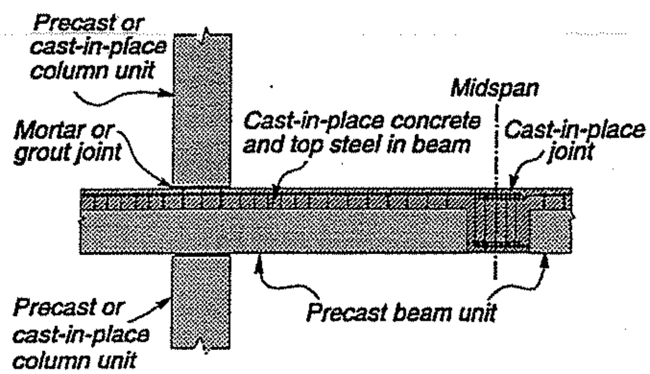


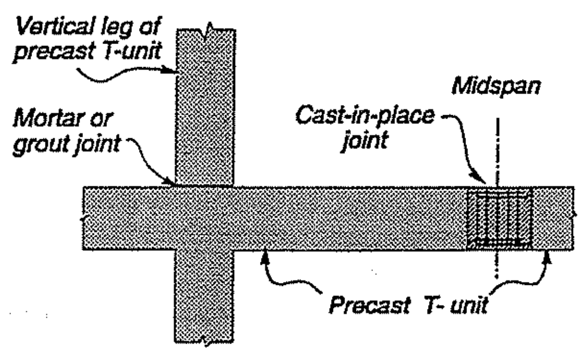

**Figure 1-8 PCI Moment Connections Tested at University of Washington [1-16]**




**(a) System 1: Precast Beam Units Between Columns**




**(b) System 2: Precast Beam Units Through Columns**

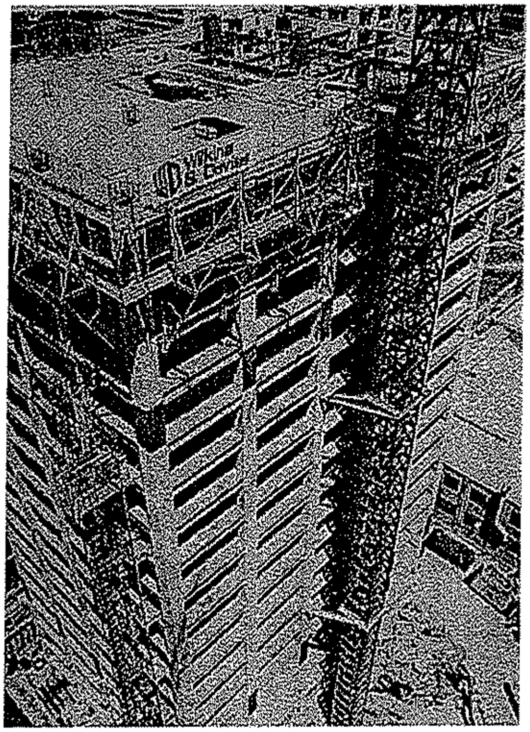



**(c) System 3: Precast T-Units**

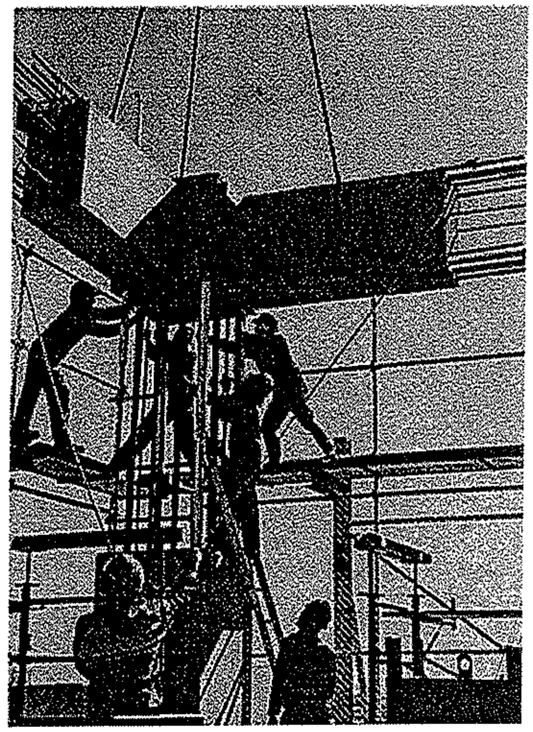
*Notes:* *Precast Concrete* *Cast-in-place concrete*

*Reinforcement in precast concrete not shown*

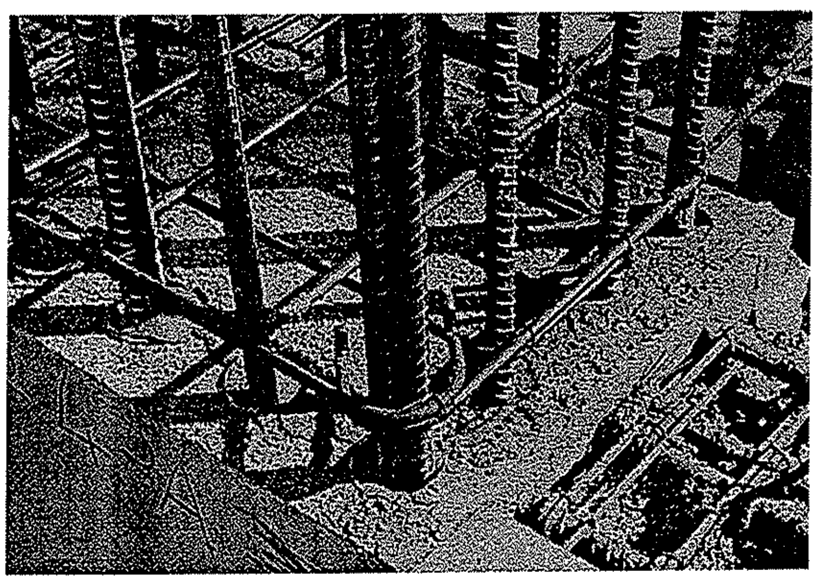
**Figure 1-9 Three Typical Precast Configurations Used in New Zealand [1-20]**



(a) Reinforced Concrete Structure with Perimeter Frame

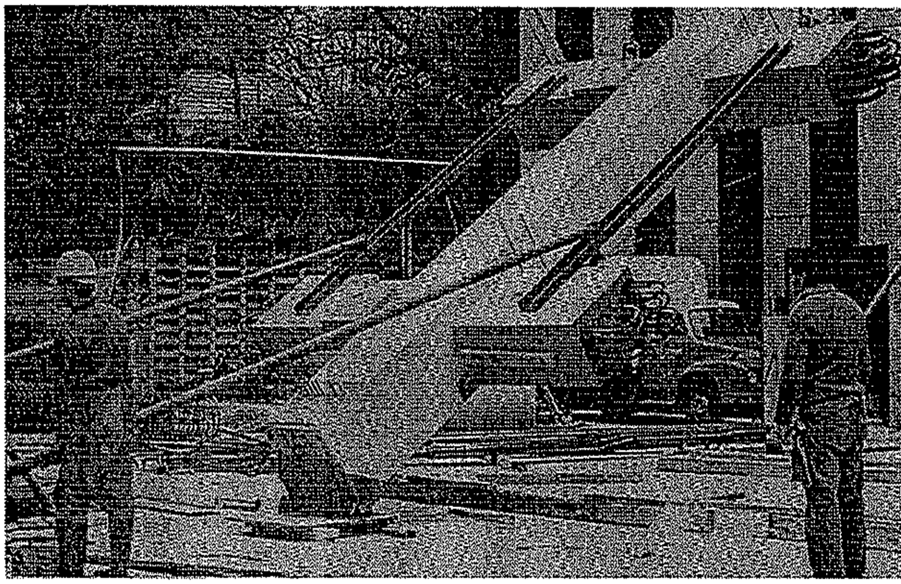


(b) Precast Concrete Beam Corner Unit Being Lowered into Place

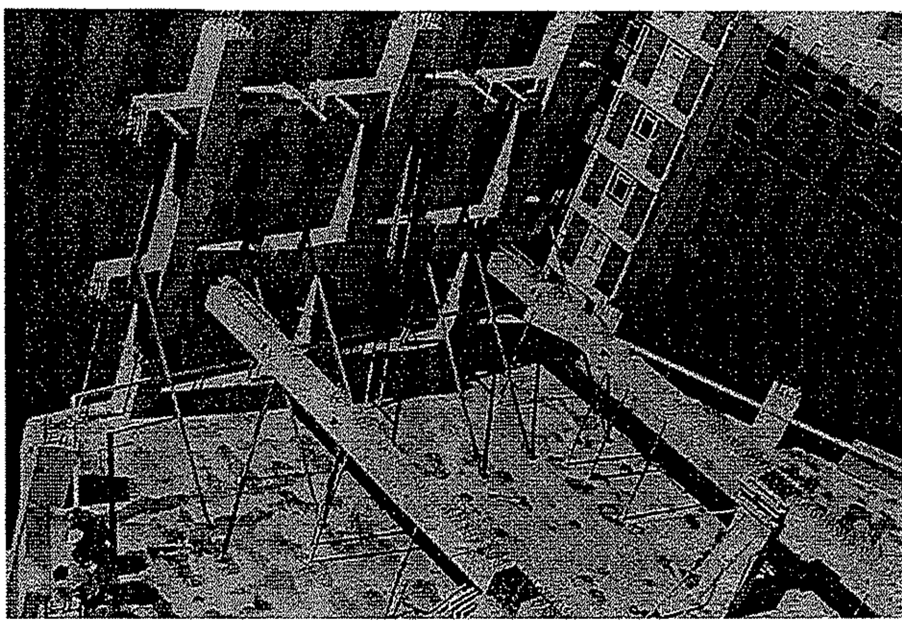


(c) Column Bars after Being Grouted in the Joint Core of a Beam Unit

**Figure 1-10 Construction of a 22-Story Building Using System 2 in New Zealand [1-20]**



(a) Double Cruciform-Shaped Precast Concrete Frame Unit Being Lifted



(b) Construction of Perimeter Frame

**Figure 1-11 Construction of a 13-Story Building Using New Zealand System 3 [1-20]**

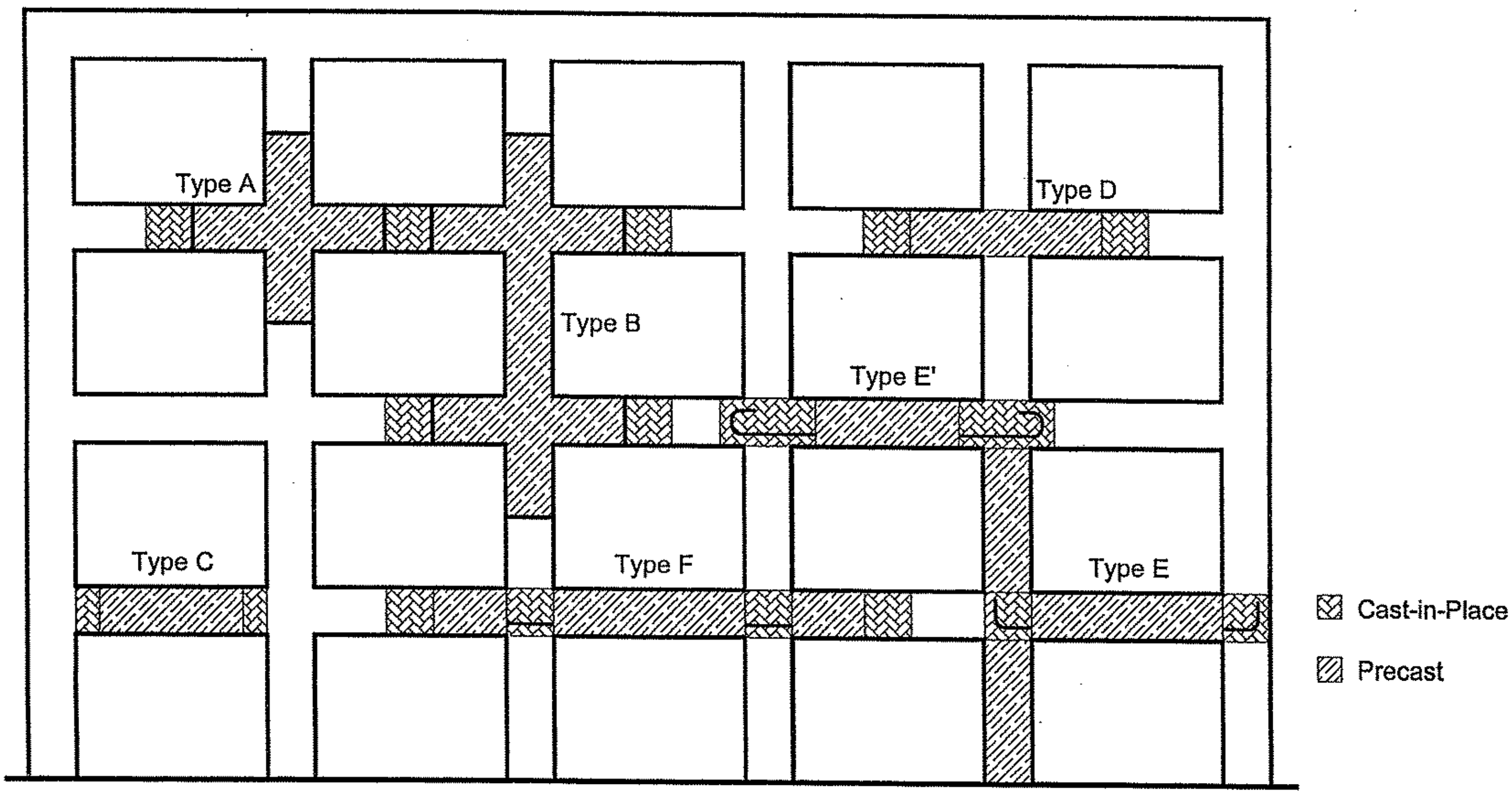



Figure 1-12 Various Precast Units in Japanes e Frame Systems (Adapted from Ref. [1-22])

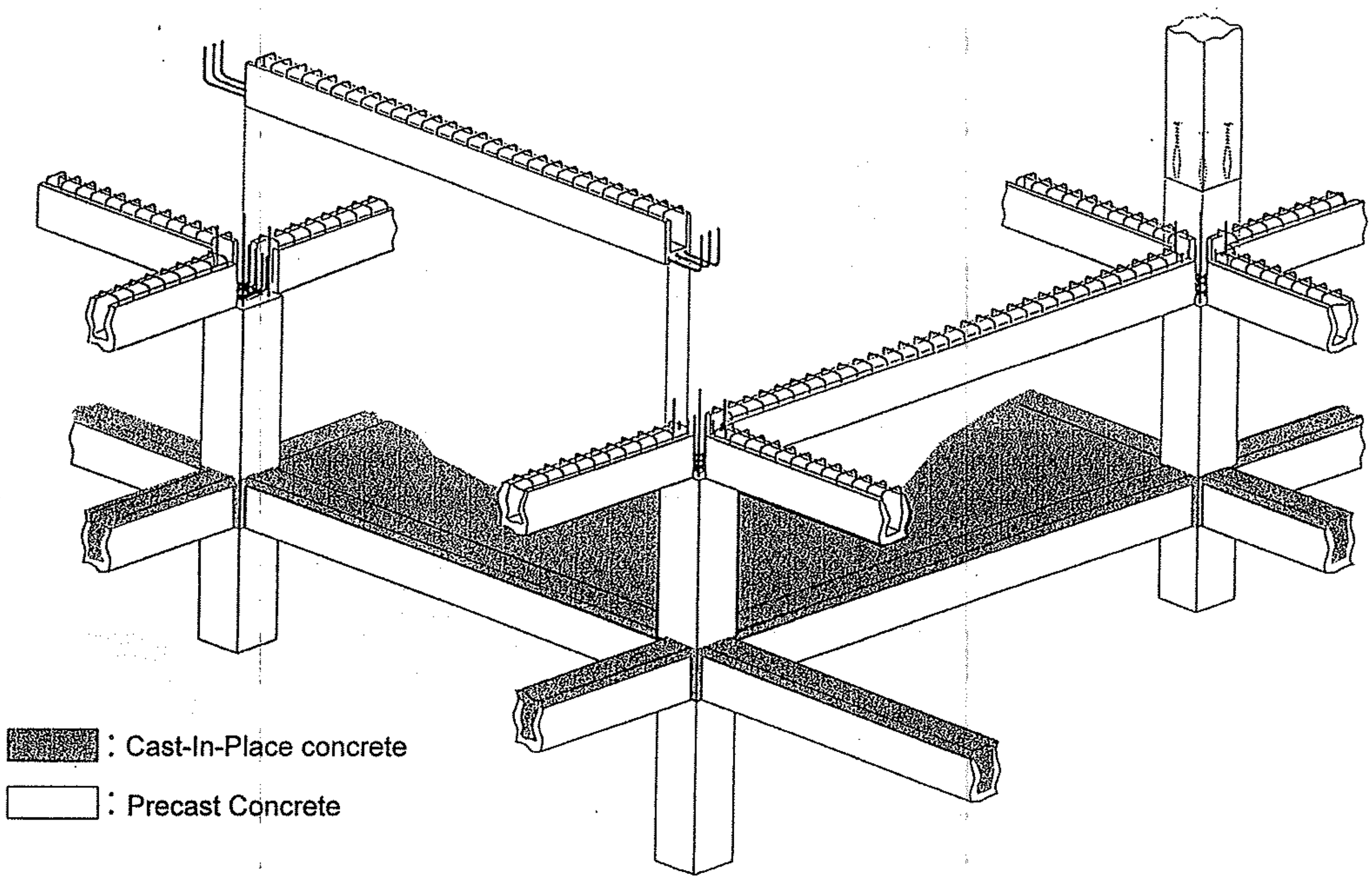



**Figure 1-13 Precast Concrete System Utilizing Beam and Column Shells (Adapted from Ref. [1-22])**

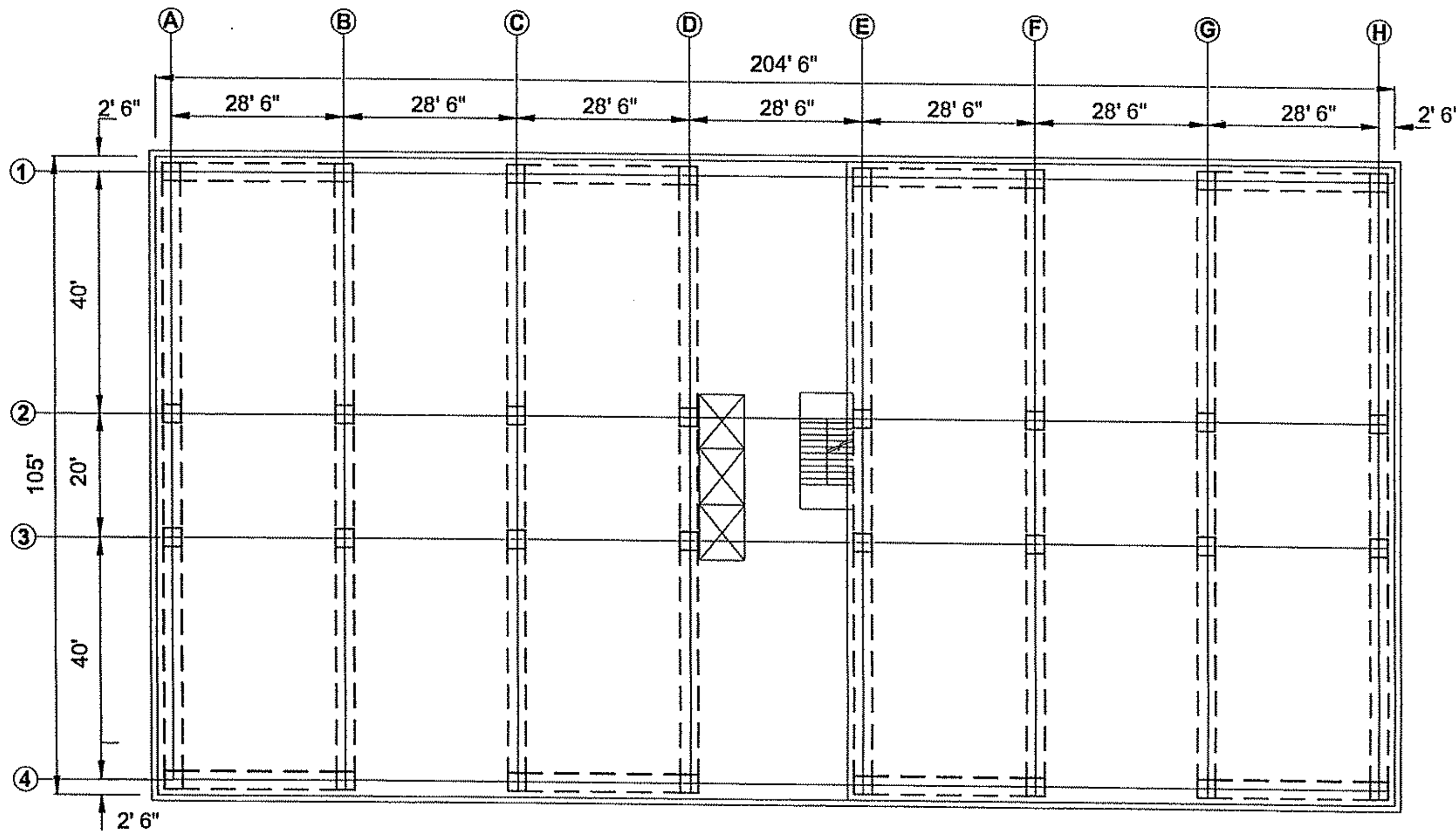



**Figure 1-14 Typical Floor Plan of PRESSS Ductile Frame Systems (Taken from Ref. [1-26])**

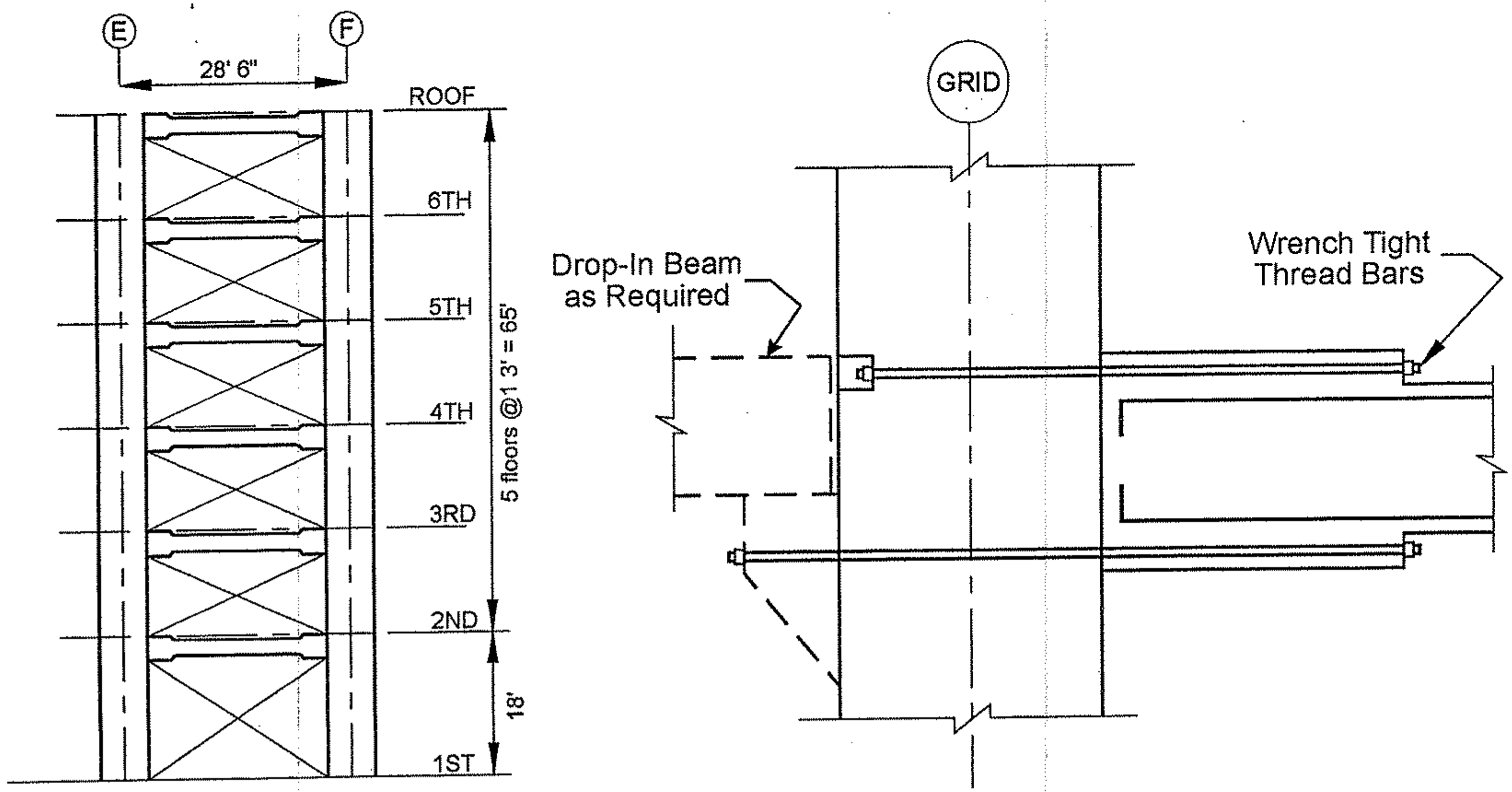



**Figure 1-15 Elevation of PRESSS Ductile Frame Systems [1-26]**

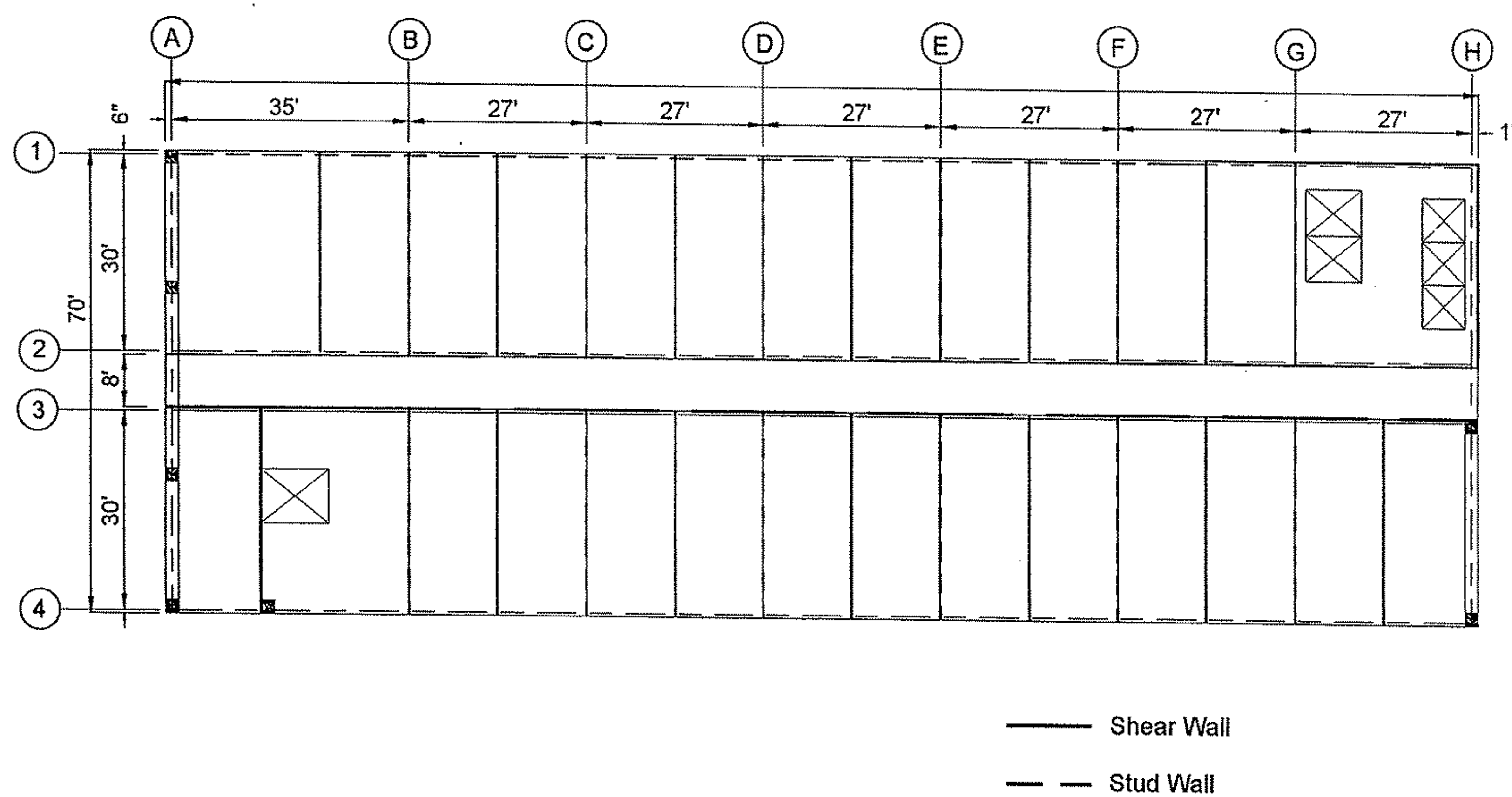



Figure 1-16 Typical Floor Plan of PRESSS Shear Wall/Bearing Wall Systems [1-26]

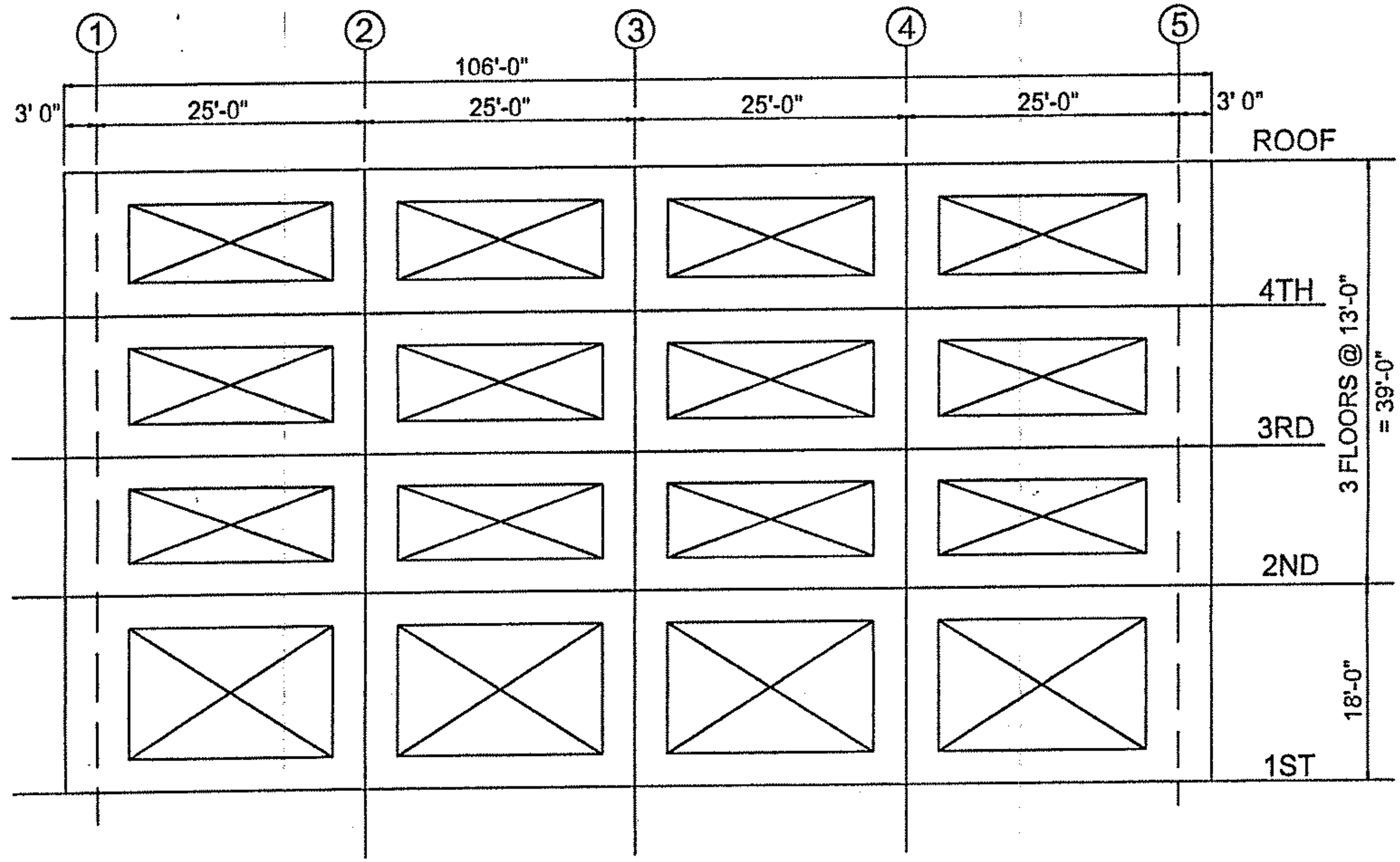



Figure 1-17  Typical Elevation of PRESSS Cladding Systems [1-26]

# 2. ATLSS PRECAST CONCRETE (APC) SYSTEMS

Existing precast concrete systems were reviewed in the last chapter. For precast concrete systems to be competitive with CIP systems, the use of CIP construction, especially at the connections between precast members, must be minimized. The earlier review has shown that most of the systems surveyed have this objective in mind, to a greater or lesser degree. In accordance with the objective to reduce CIP concrete, new ATLSS Precast Concrete (APC) systems were developed at the ATLSS Center. These new APC systems have components such that CIP construction at the joints are eliminated. Section 2.1 describes the APC systems in general, and Sections 2.2 and 2.3 deal with beam-to-column and beam-to-beam connections, respectively. Feasible construction and erection sequences for these APC systems are demonstrated in Section 2.4.

## 2.1 Description of APC Systems

The proposed APC systems are innovative precast concrete systems for, but not limited to, low- to mid-rise buildings in moderate seismic zones. They are composed primarily of linear elements, i.e., beams and columns. The systems are compatible with potential future automated construction environments and are composed of two feasible alternatives: two-dimensional and three-dimensional systems. The two-dimensional APC system involves coupling a planar framing system and a wall system. As an option, any commercially available gravity framing system can be added to the two-dimensional APC system. All the systems (the planar and the gravity framing systems, and the wall system) systems contribute in carrying gravity loads. However, only the planar framing system and the wall system resist lateral loads; the planar framing system resists the lateral loads in its planar direction, while the wall system resists the lateral loads in the direction perpendicular to the planar system. The planar framing system utilizes two connection schemes called the ATLSS beam-to-column and the ATLSS beam-to-beam connections. Figure 2-1 shows an example of a typical building plan utilizing the two-dimensional APC system with precast floor slabs. Note that precast floor slabs are used in the two-dimensional APC system to minimize construction periods. The orientation of the precast slabs is at the discretion of the design engineer.

The three-dimensional APC system includes an interior and a perimeter framing system. The interior framing system is for gravity loading and can be any of the commercially available gravity loading systems. The perimeter system, on the other hand, is mainly for lateral loading. It utilizes the ATLSS beam-to-column and ATLSS beam-to-beam connections. Similar to the two-dimensional system, the floor system of the three-dimensional APC system is composed of precast slabs spanning in an appropriate direction.

A wall system can be introduced into the three-dimensional framing system, and is used to carry parts of the gravity and the lateral loads. This will reduce the burden on the perimeter framing system. Figures 2-2 and 2-3 display plan views of an example building structure utilizing the three-dimensional framing system without and with a wall system, respectively.

## 2.2 ATLSS Beam-to-Column Connections

Two types of ATLSS beam-to-column connections have been developed for the APC systems: the modified ATLSS passage-type (MAPT) and the ATLSS nodal-type (ANT) connections. As the name implies, the MAPT connection is a modification of the previous ATLSS Passage-Type (APT) connection [1-1]. The ANT, on the other hand, is a *new* connection. These connections are described as follows.

### 2.2.1 Modified ATLSS Passage-Type (MAPT) Connection

The MAPT connection is a two-dimensional precast beam-to-column moment connection. It has two variations; namely, regular and exterior connections. Consequently, it can only be used in the two-dimensional APC system, or as regular connections in the three-dimensional system. The regular MAPT connection is utilized where beam continuity across a column is required, whereas the exterior connection is used where a beam terminates at a column. Figure 2-4 shows both types of the MAPT connection.

Unlike an APT connection which has a precast column with a passage in the middle for a precast beam to pass through, a MAPT connection is composed of three components: the lower column, the drop-in beam, and the upper column. The columns have a U-shape opening (passage or slot) with protruding rebars at the top and have devices called splice sleeves embedded at the bottom. The splice sleeves are used for joining the column longitudinal rebars. The joining process is achieved by means of grouting the sleeves after the column rebars have been inserted in them. The drop-in beam has metal ducts embedded at its center. Metal ducts provide a passageway for the longitudinal rebars to extend from the lower to the upper columns. The rebars passing through the precast beam provide extra stiffness to the MAPT connection, as compared to the APT connection. Furthermore, these rebars make the MAPT connection more monolithic than the APT connection.

Figure 2-5 demonstrates the construction sequence for a regular MAPT connection. First, a lower column is erected at its location. Then a precast beam is lowered onto the lower column, while the column rebars are guided through the metal ducts. The beam-column interface and the metal ducts are grouted. After the grout has set, an upper column is framed in on top of the beam-and-lower-column subassembly. At the same time, the rebars of the lower column are directed into the splice sleeves at the bottom of the upper column. The splice sleeves and the interface between the upper column and the lower assembly are then grouted to form a complete MAPT connection.

### 2.2.2 ATLSS Nodal-Type (ANT) Connection

The ANT connection is also a moment connection. It has three possible configurations: regular, exterior, and the corner (see Fig. 2-6). Similar to the MAPT connections, the regular connection is used where beam continuity across a column is required. The exterior and the corner connections, on the other hand, are utilized where a beam ends at a column. The difference between the exterior and the corner connection is that the exterior connection is two-dimensional, whereas the corner connection is three-dimensional. ANT connections can be used in both two and three-dimensional systems.

An ANT connection is composed of three elements; namely, the upper column, lower columns and the nodal beam. The columns rebars project out at the top of the column, while splice sleeves are embedded at the bottom. The nodal beam has an enlarged section called the node through which metal ducts have been embedded, as passages for the column rebars. The construction sequence of an ANT connection is similar to that of a MAPT connection and is shown in Fig. 2-7.

## 2.3 ATLSS Beam-to-Beam Connection

The need for a beam-to-beam connection arises when transportation constraints prevent casting beams to the required length. The proposed ATLSS beam-to-beam connection is illustrated in Figs. 2-8a and 2-8b. It utilizes a set of connectors called the APC connectors to connect a pair of precast beams. A set of APC connectors contains two connectors. The first one is called the mortise, and the second the tenon. The mortise is embedded at the end of a connecting beam, whereas the tenon is partially embedded and partially protrudes from the end of the other beam.

Normally, each beam carries either two mortises or two tenons, although an exception occurs when a beam is to connect with an exterior column to form an exterior beam-to-column connection. Such a beam carries only one mortise. The beams with the mortises are erected first. Then the beams carrying the tenons are placed in such a way that the tenons are fully inserted into the mortises. Each mortise-tenon connection is then grouted to complete the beam-to-beam connection.

Use of the APC connectors results in a shear connection. To develop a modest amount of bending resistance, the connectors can be used in conjunction with two sets of top and bottom plates (see Fig. 2-9). The first set of the top and bottom plates is anchored to the beam reinforcement before casting. The second set of plates is field bolted to the first set after the mortise and the tenon are engaged and grouted. An alternative is to bolt one of the second set of plates to the bottom of the beam containing the mortise on the ground before lifting. This would reduce the effort required in field installation of the second set of plates.

## 2.4 Description of Construction and Erection Procedures

### 2.4.1 Two-dimensional APC System

A typical floor plan and an elevation of a demonstration structure is depicted in Figs. 2-10 and 2-11. The example structure is assumed to be located in New England region (seismic zone 2) and is primarily for office use.

The design for the structure calls for the use of the two-dimensional APC system with the ANT beam-to-column connection scheme and with a gravity framing system. The system is composed of four planar and five gravity two-dimensional frames. The frames designated for the planar system are located on the vertical grid lines B, D, F, and H, whereas those for the gravity system are located on the vertical grid lines A, C, E, G, and I. The wall system includes six shear walls locating on horizontal grid lines 2 and 3.

Figure 2-12 shows the typical configuration for each type of precast member. Each story of the structure is composed of two types of precast columns (C21 and C22), six types of precast beams (B21 to B26), one type of precast wall (W21), one type of precast double-tee slab (S21) and CIP topping. Column C21 and beams B21 and B22 are components of the planar frames, and column C22 and beams B23 to B26 are parts of the gravity frames. The precast wall W21 belongs to the precast wall system. The double-tee slab S21 is for transferring gravity loads onto the frames and walls, whereas the CIP topping is to provide the diaphragm action under lateral loads and also to provide the required finishing surface.

It should be noted that for this structure, each B21 beam carries a mortise at its inner end, and a B22 beam carries two tenons. Furthermore, if the selected mode of transportation can accommodate precast members of length up to roughly 120 ft, the precast beams B21 and B22 can be replaced by a single beam to eliminate the need for the beam-to-beam connections. In addition, the precast columns C22 can be replaced by a single column spanning from the ground level to the roof level, thus eliminating the need of joining the columns at every story level. These would reduce the number of crane picks and the time spent on grouting, resulting in a lower erection cost.

A proposed construction sequence for a typical story of the demonstration structure follows. This sequence assumes that the transportation mode utilized cannot accommodate longer member length. Thus, the precast beams B21 and B22 cannot be cast integrally, and the column C22 can span only one story at a time.

1. Erect the precast walls W21 and the precast columns C21 and C22 of the first floor. Plumb all wall and column pieces and temporarily brace them for later grouting.
2. Seat and drop in the precast beams B21 to B26 of the second floor atop the precast columns of the first floor. Then place the precast double-tee slab S21 on the beam ledges.
3. Check the plumbness of the precast walls and columns, and adjust the alignment if necessary.
4. Grout (not necessarily in the following order) all the splice sleeves at the wall and column bases, the metal ducts at the beam-to-column connections, and the coupled mortises at the beam-to-beam connections.
5. Remove all the temporary bracing.
6. Repeat steps 1 to 5 for each floor.
7. Place the necessary reinforcement for the CIP topping on top of the precast slabs S21 of every floor, and pour concrete.

Installation of non-structural wall panels and cladding, and mechanical and electrical systems for each completed story can be done after the erection of the floor above is finished. Alternatively, the installation can be carried out after the entire structural skeleton is erected.

### 2.4.2 Three-dimensional APC System

The demonstration structure discussed above is also used for the three-dimensional APC system. The system utilizes a perimeter frame and eight shear walls to resist all lateral loads, and uses three interior frames to assist in carrying gravity loads (see Fig. 2-13). In addition, the perimeter frame utilizes the ANT beam-to-column connection scheme.

Figure 2-14 shows the typical shape of the precast members. Each story of the structure is comprised of two types of precast columns (C31 and C32), ten types of precast beams (B31 to B310), two types of precast shear walls (W31 and W32), one type of precast double-tee slab (S31) and CIP topping. Column C31 and beams B31 to B36 are parts of the perimeter frame, whereas column C32 and beams B37 to B310 make up the interior frames. Walls W31 and W32 pertain to the shear wall system. The precast slab S31 is for distributing gravity loads onto the frame-wall systems. The CIP topping provides the diaphragm action to transmit lateral loads.

The proposed construction sequence for a typical story of the structure is as follows, using the same assumptions as before.

1. Erect the precast walls W31 and W32, and the precast columns C31 and C32 of the first floor. Plumb and temporarily brace them for later grouting.
2. Seat the precast beams B31 to B34 and B37 to B310 of the second floor atop the columns and walls of the first floor, and then drop in the precast beams B35 and B36. Furthermore, place the precast double-tee slab S31 onto the beam ledges.
3. Verify the plumbness and leveling of all the precast members, and make adjustments as necessary.
4. Grout (not necessarily in the following order) all the splice sleeves at the wall and column bases, the metal ducts at the beam nodes, and the coupled APC connectors at the beam-to-beam connections.
5. Remove all the temporary bracing.
6. Repeat steps 1 to 5 for each floor.
7. Place the necessary reinforcement for the CIP topping on top of the precast slabs S31 of all floors, and pour concrete.

Installation of the non-structural components of each completed story can be carried out either after the floor above it has been finished, or after the entire structural skeleton has been erected.

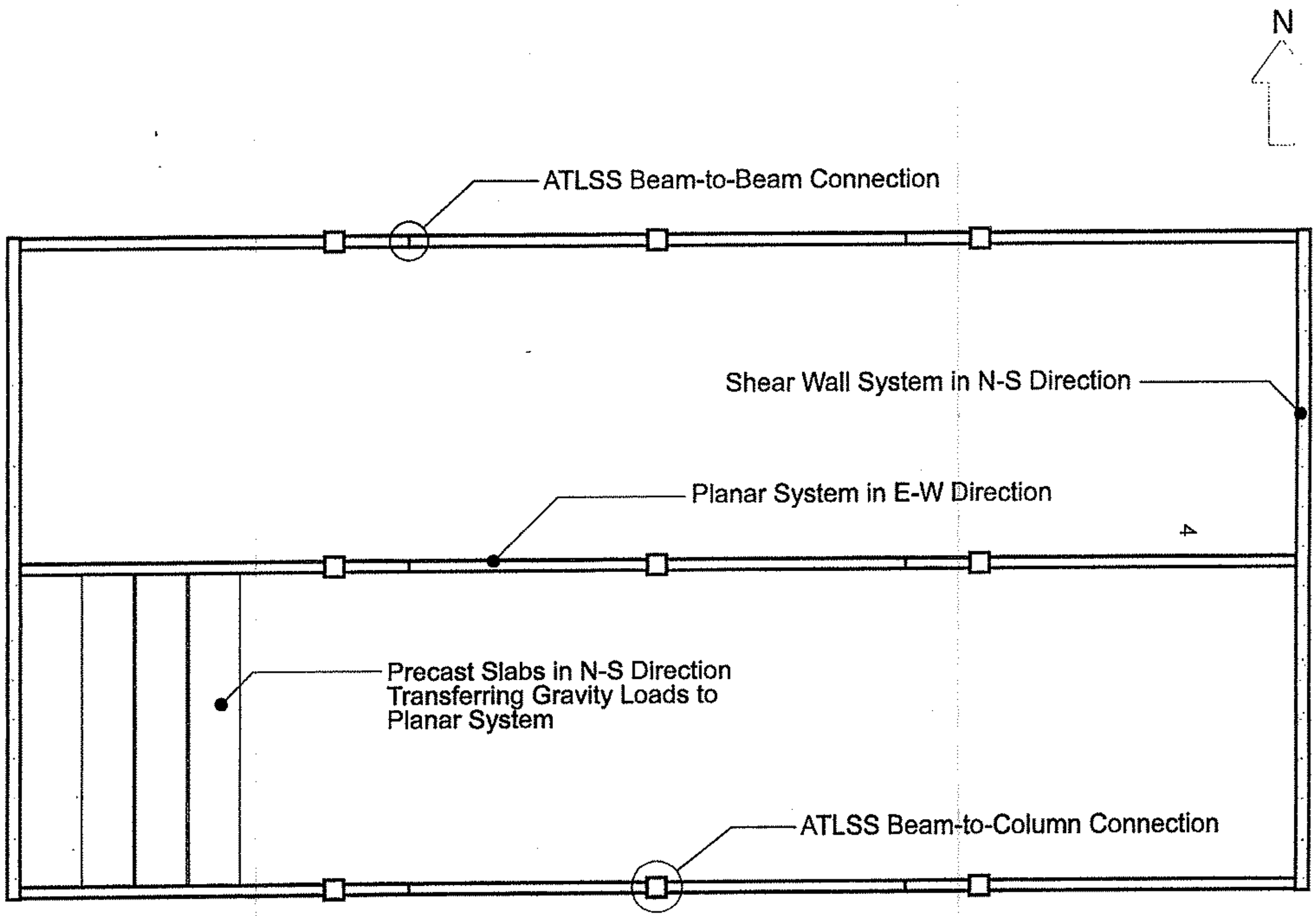



**Figure 2-1 Typical Floor Plan of a Structure Using Two Dimensional APC System**

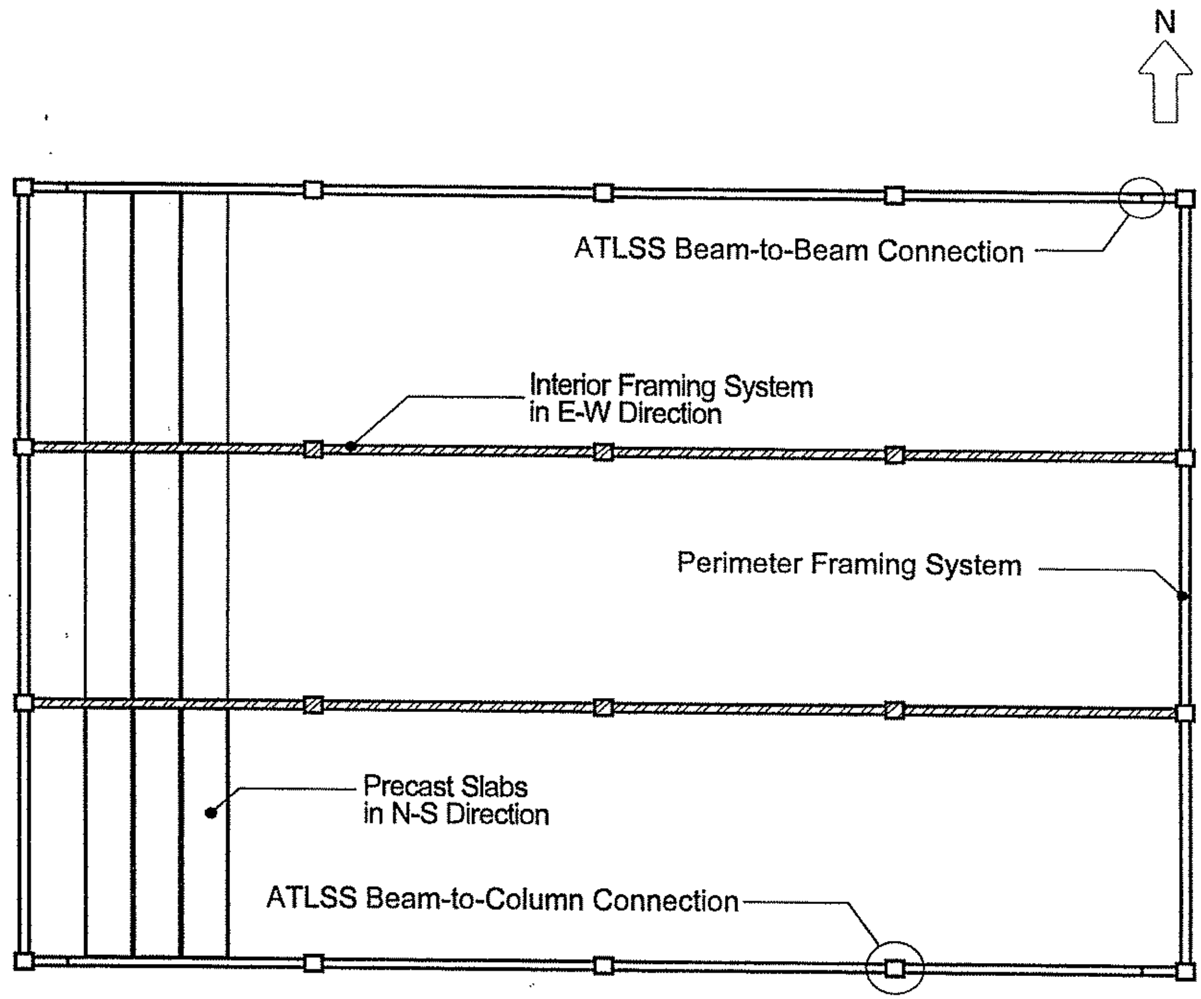



**Figure 2-2 Typical Floor Plan of a Structure Using Three Dimensional APC System (without Wall System)**

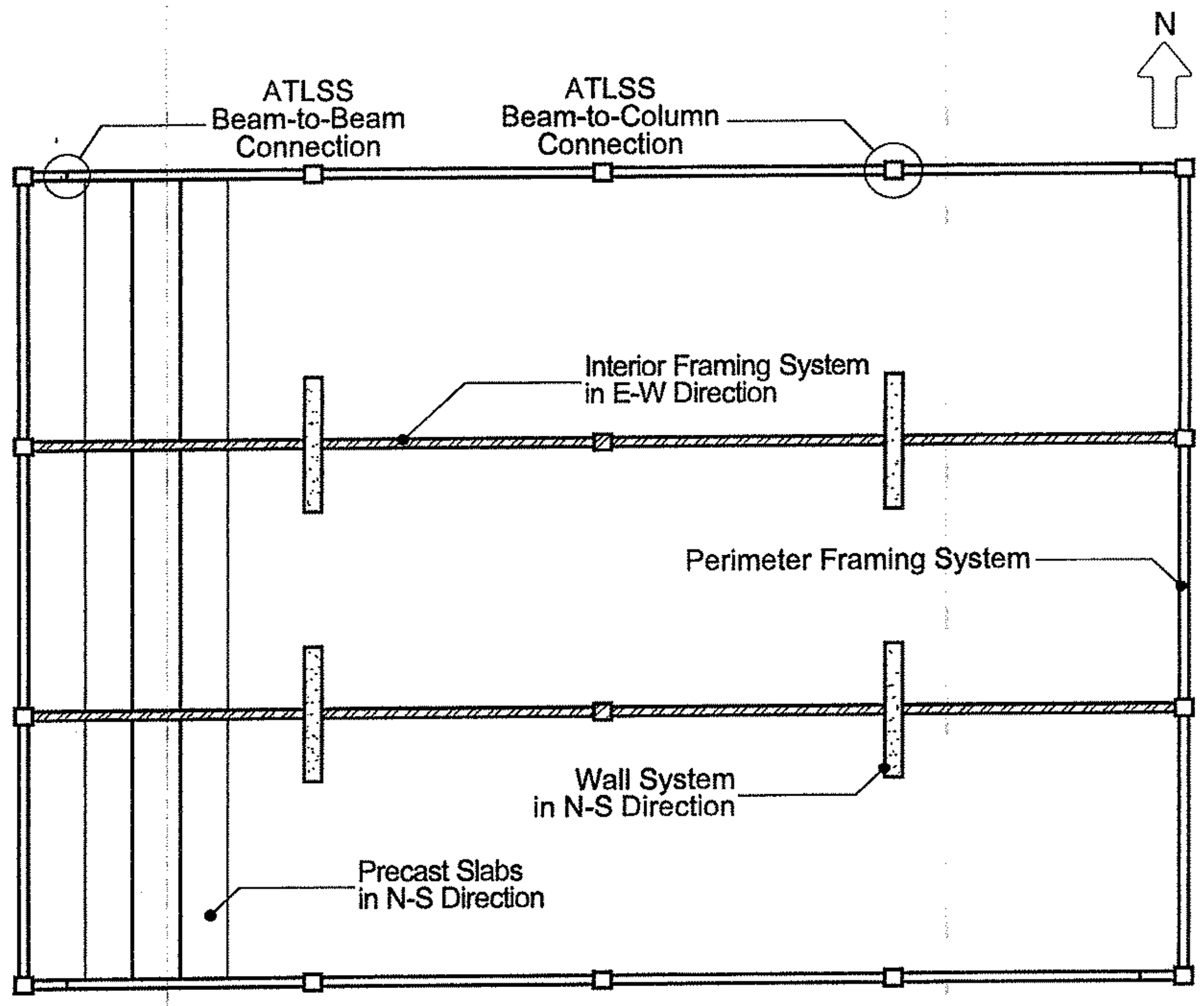
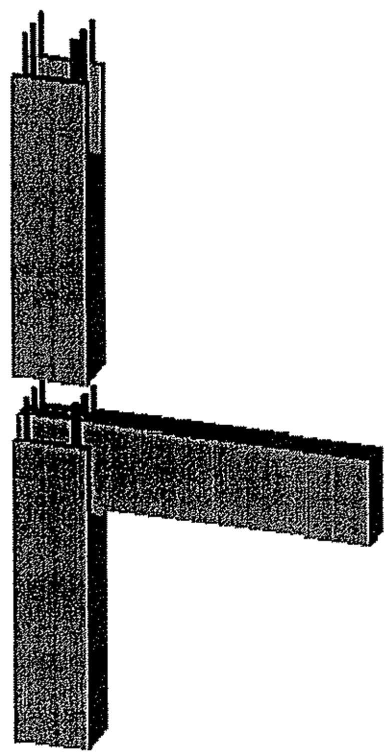
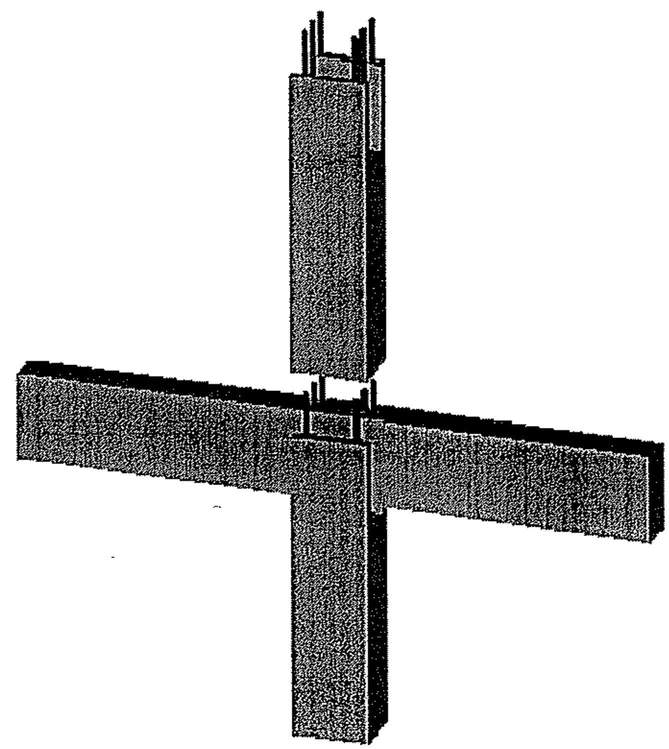



**Figure 2-3 Typical Floor Plan for a Structure Using Three Dimensional APC System (with Wall System)**

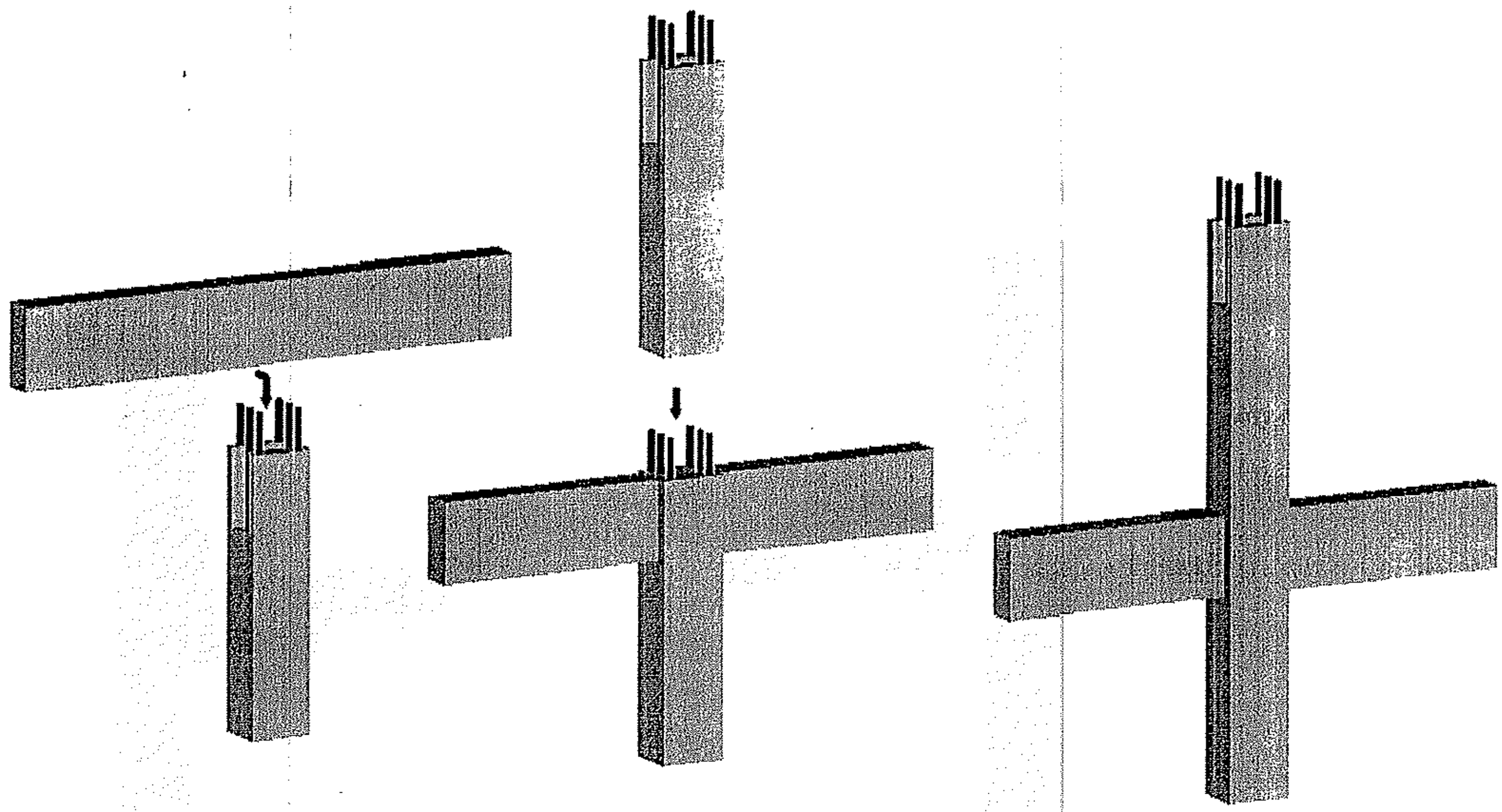


(a) Exterior MAPT Connection



(b) Regular MAPT Connection

Figure 2-4 Two Variations of MAPT Connections

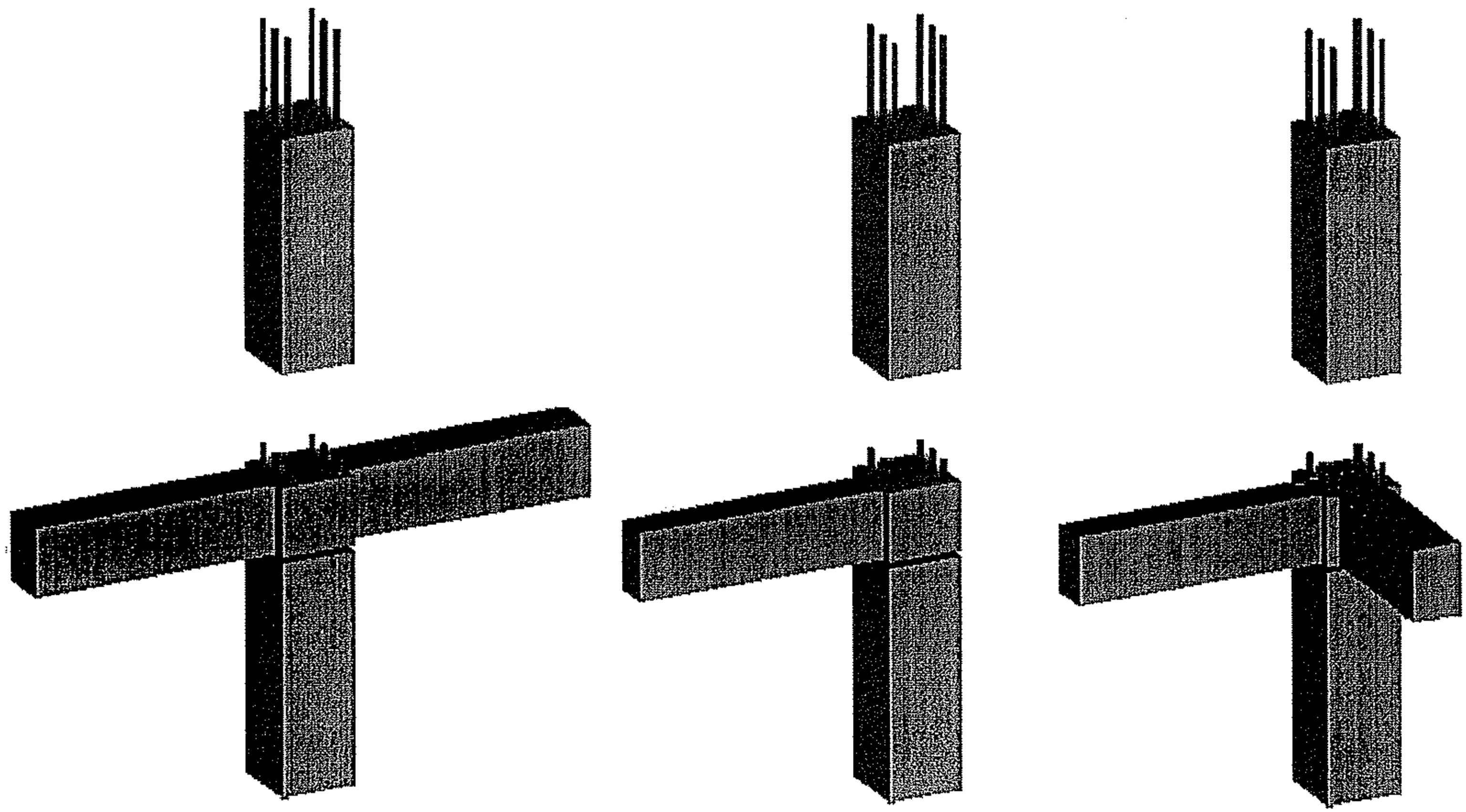


(a) Placing of a Precast Beam onto a Precast Column

(b) Framing in an Upper Column

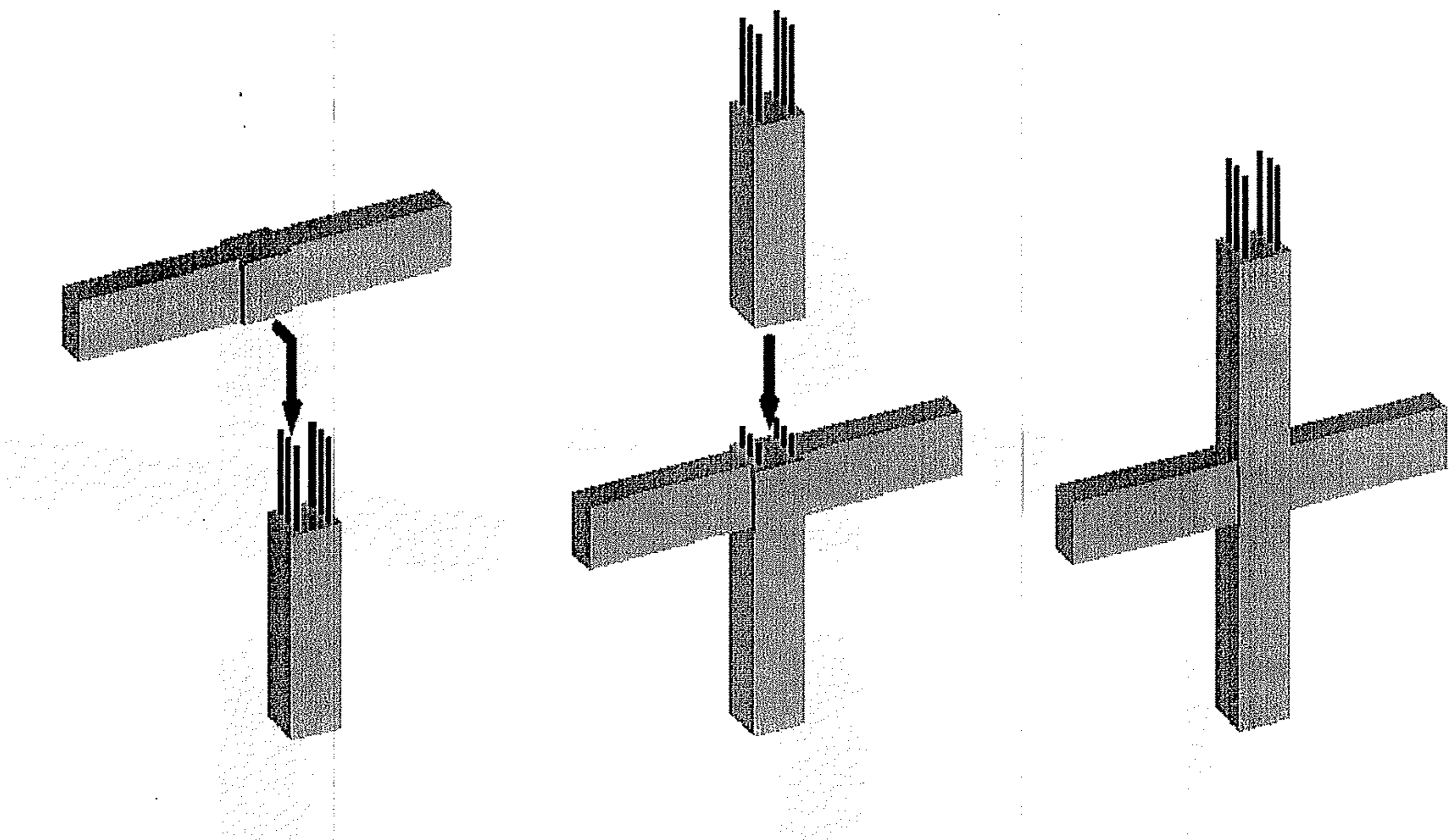
(c) A Completed MAPT Connection

Figure 2-5 Construction Sequence of a MAPT Connection



(a) Regular ANT Connection (b) Exterior ANT Connection (c) Corner ANT Connection

Figure 2-6 Three Variations of ANT Connections



(a) Placing of a Precast Beam onto a Precast Column

(b) Framing in an Upper Column

(c) A Completed ANT Connection

Figure 2-7 Construction Sequence of an ANT Connection

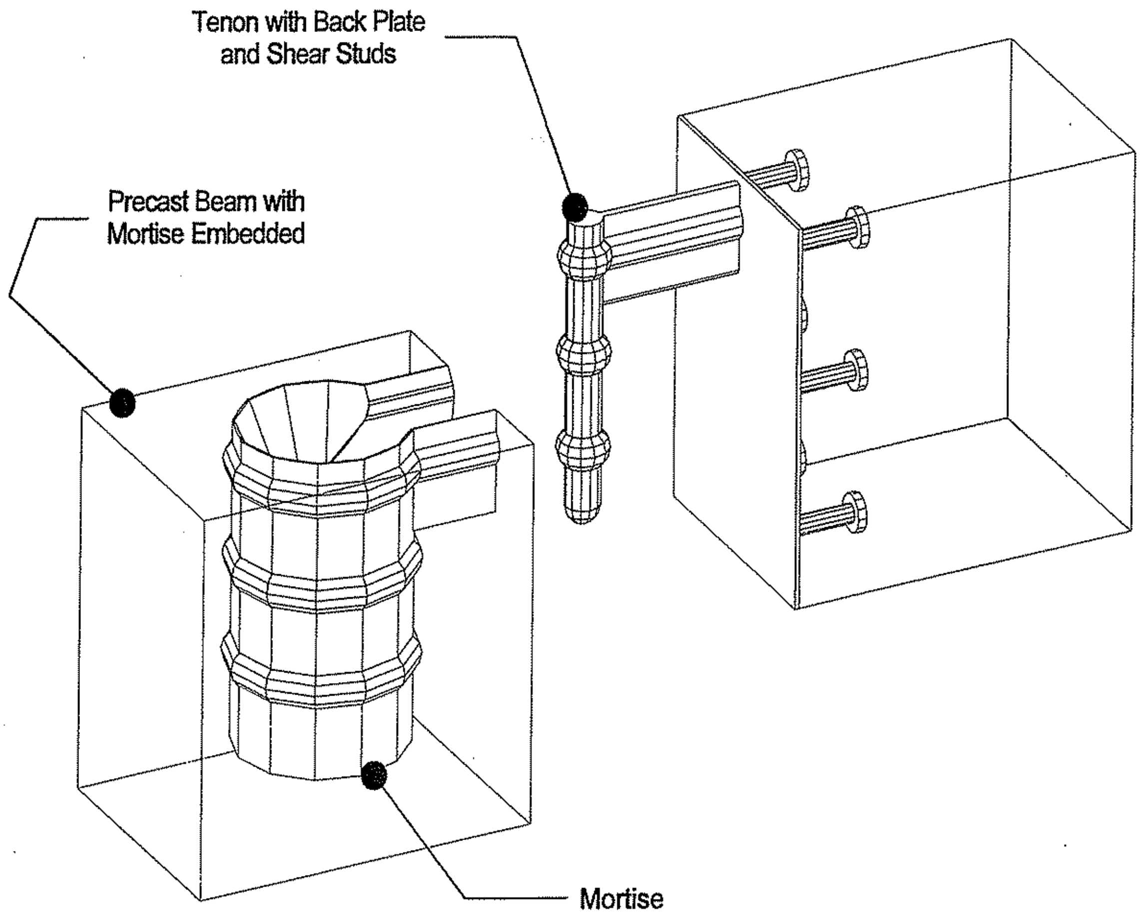



**Figure 2-8a Proposed ATLSS Beam-to-Beam Connection**

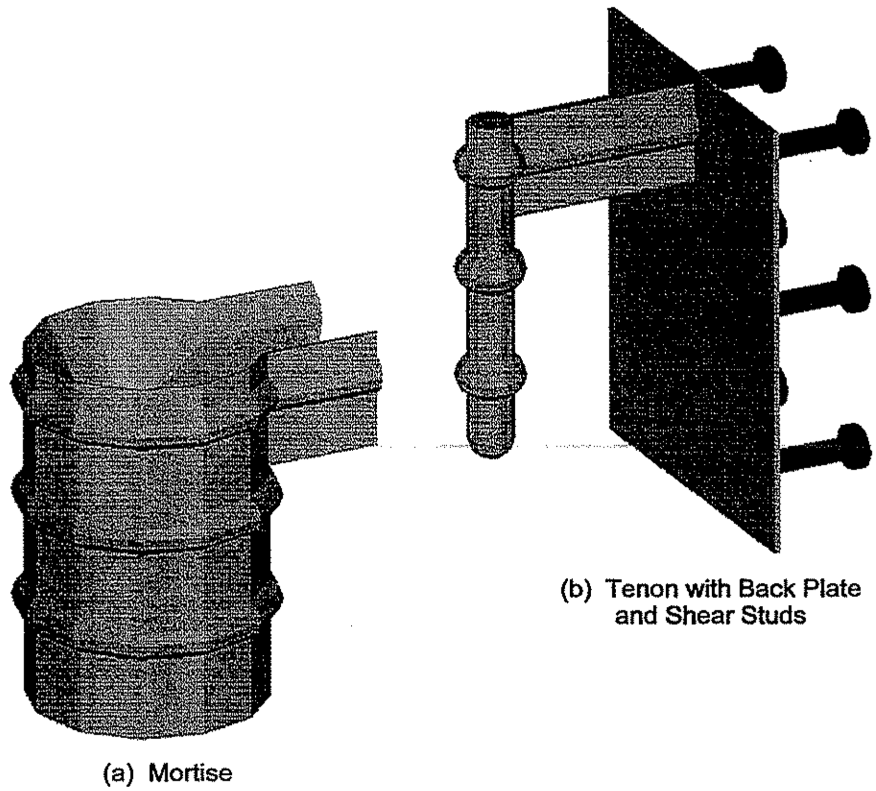



**Figure 2- 8b APC Connectors**

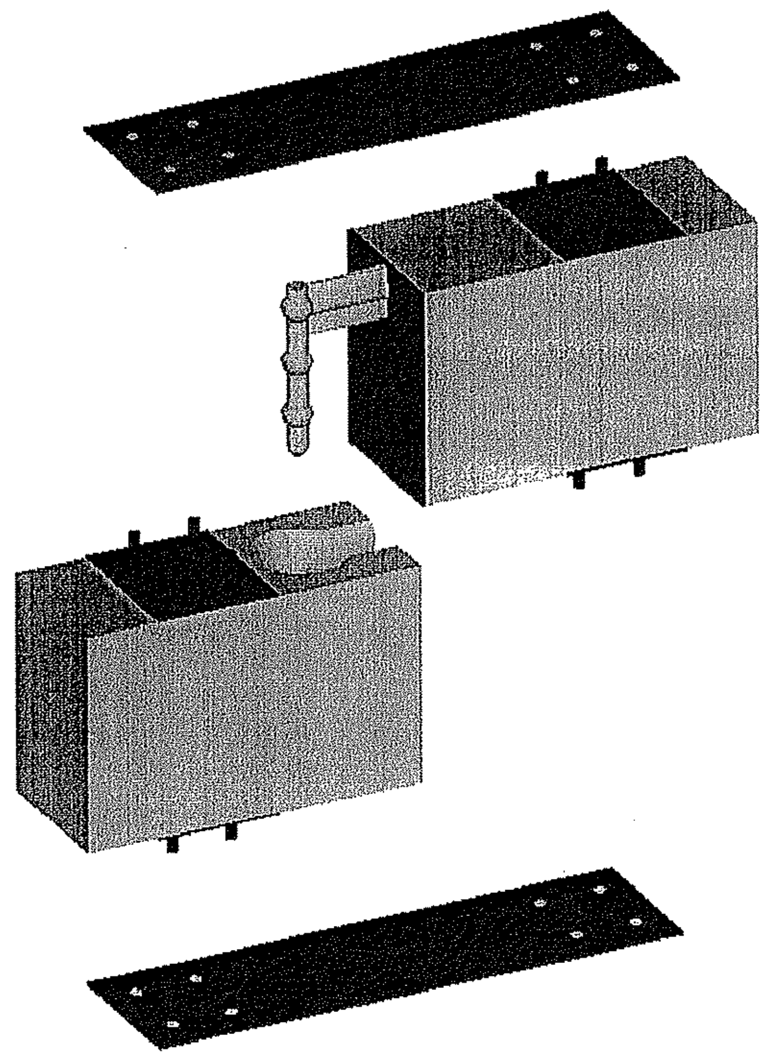


Figure 2-9 ATLSS Beam-to-Beam Connection with Optional Top and Bottom Plates for Moment Transfer

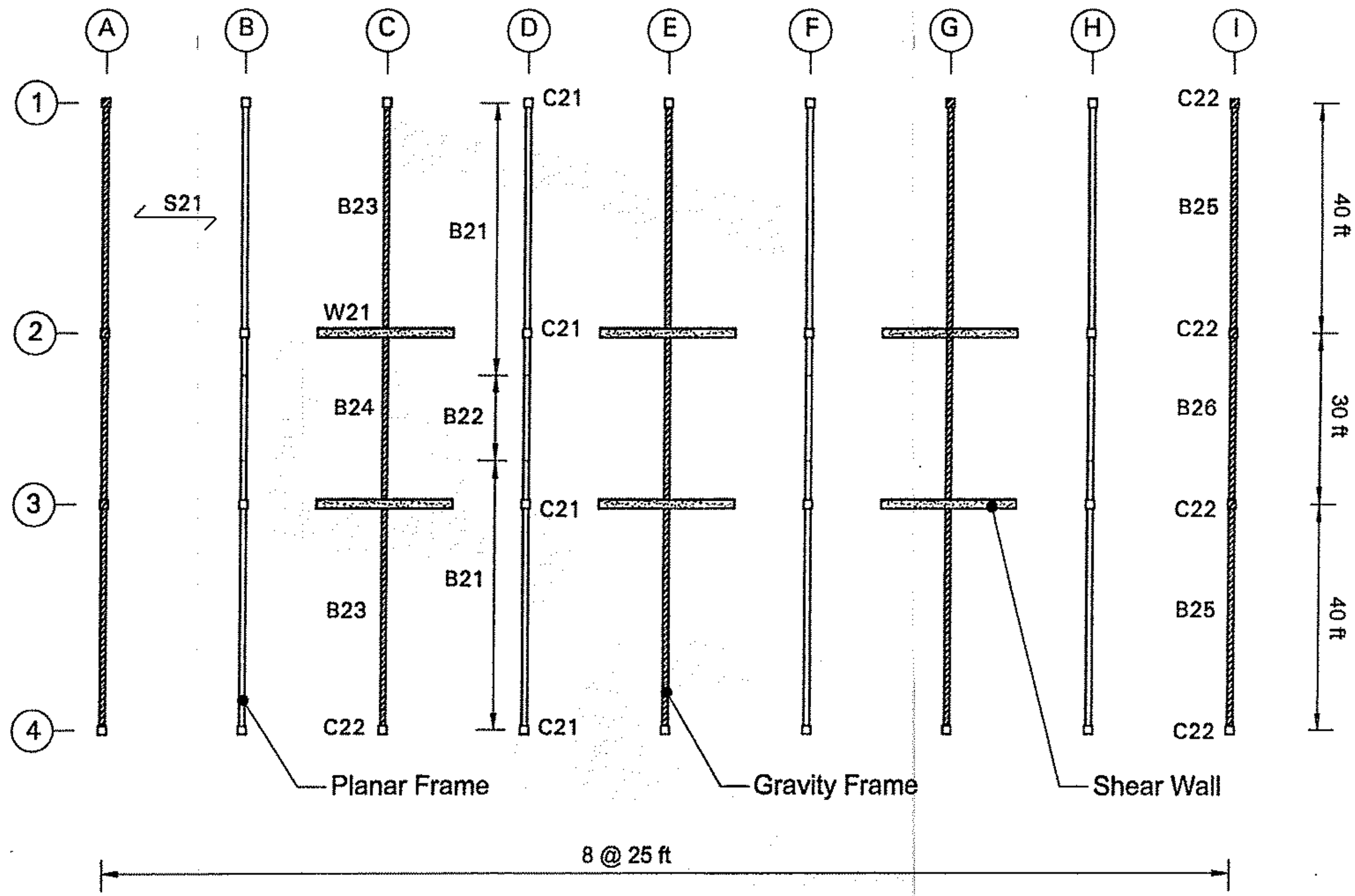



Figure 2-10 Typical Floor Plan of the Demonstration Structure Using 2D-APC System

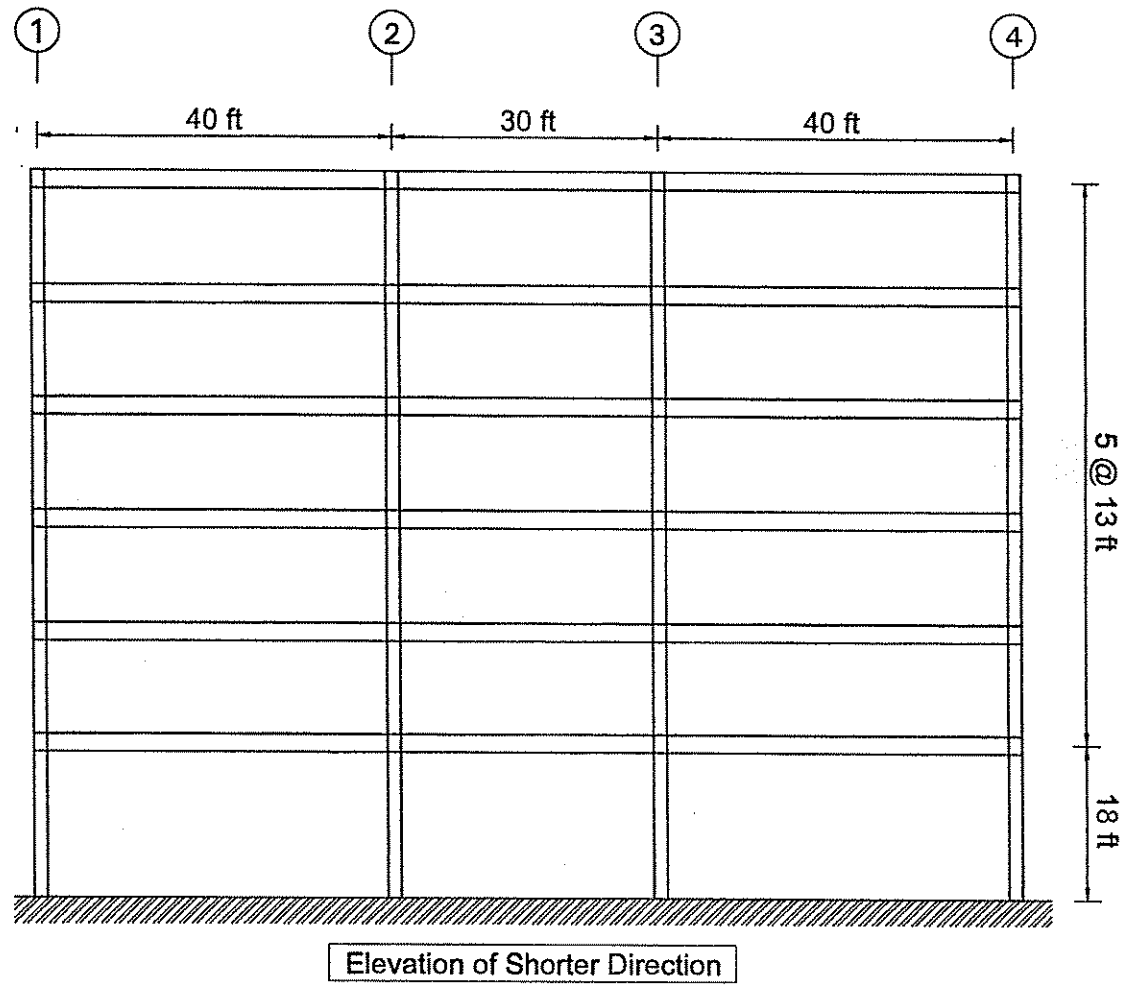



**Figure 2-11 Elevation of the Demonstration Structure Using 2D-APC System**

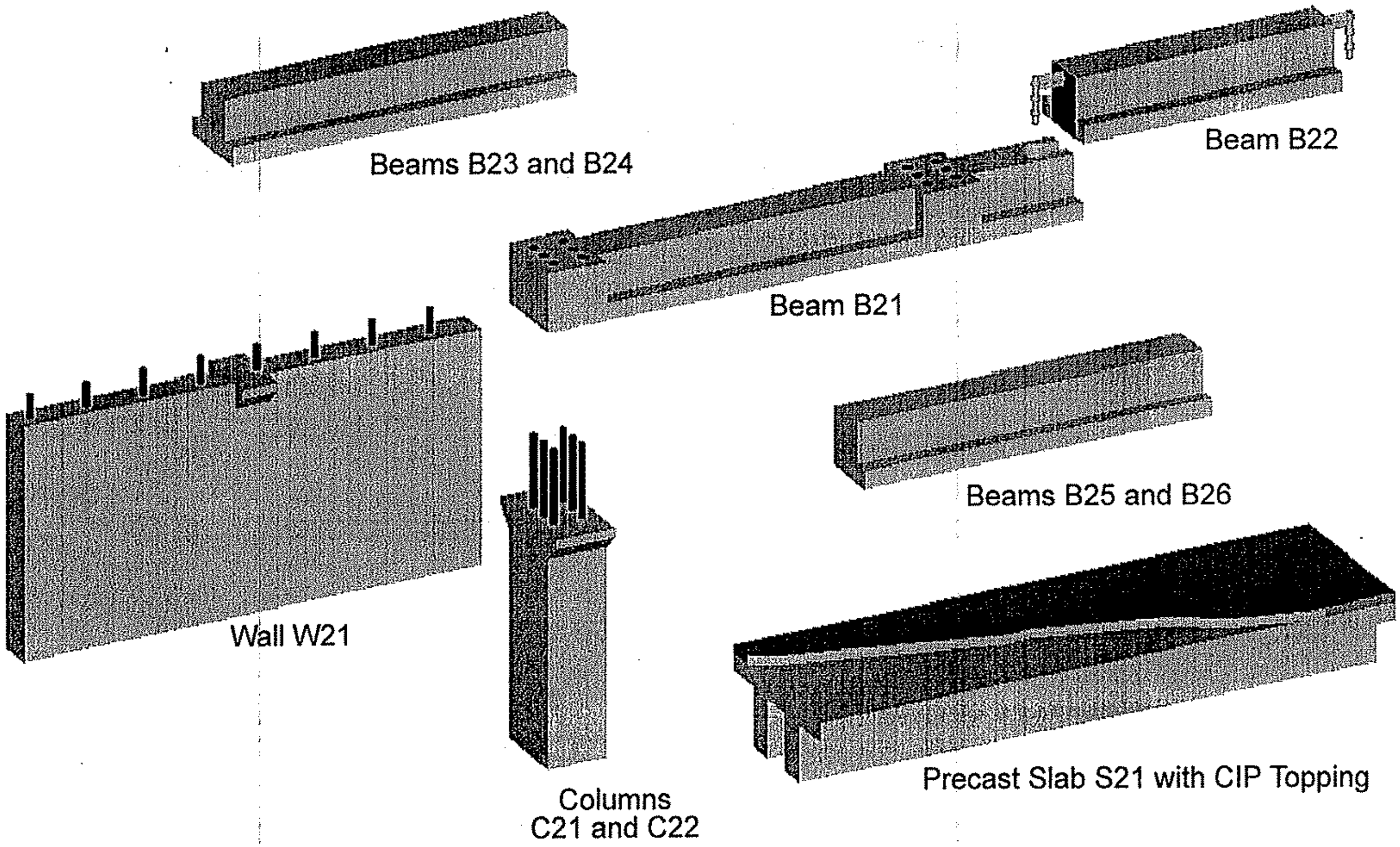



**Figure 2-12 Typical Shape of Precast Members for the Demonstration Struture Using 2D-APC System**

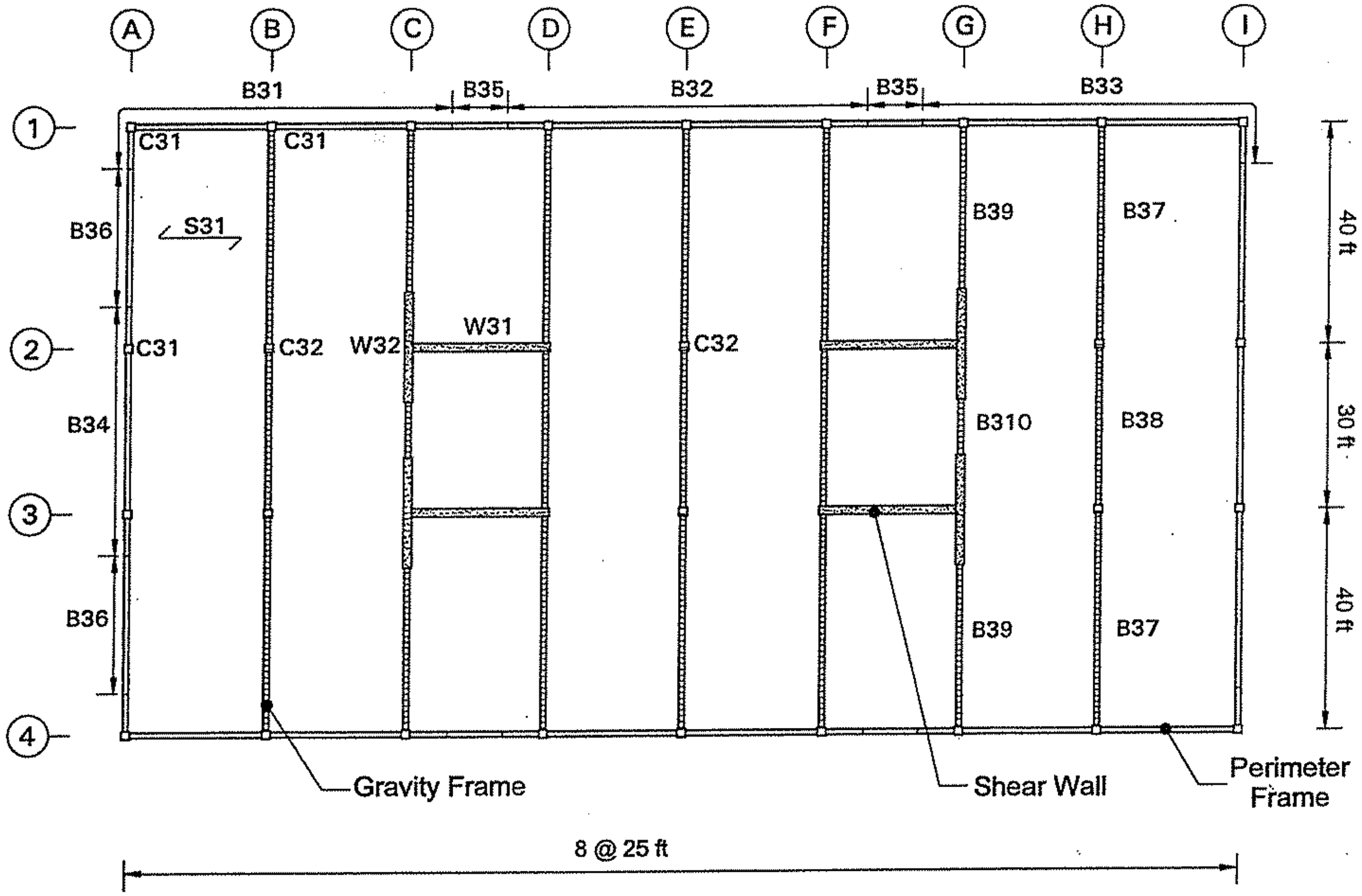



**Figure 2-13 Typical Floor Plan of the Demonstration Structure Using 3D-APC System**

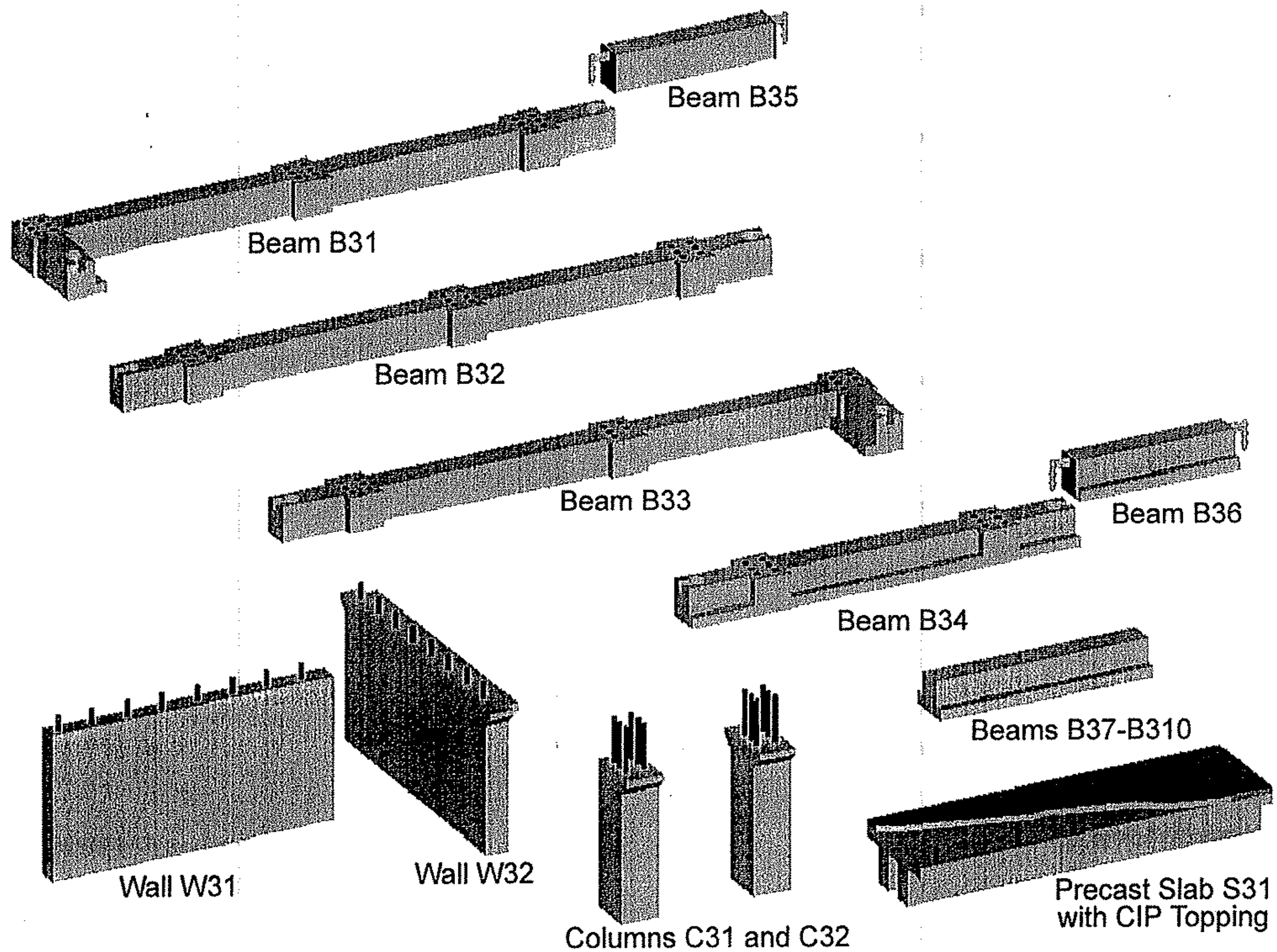



**Figure 2-14 Typical Shapes of Precast Members for the Demonstration Structure Using 3D-APC System**

# 3. SUMMARY AND FUTURE WORK

## 3.1 Summary

New precast concrete systems called the ATLSS Precast Concrete (APC) systems were conceptually developed in this study, the initial part of a larger, overall study. The APC systems are intended primarily for, but not limited to, low- to medium-rise buildings in moderate seismic zones. The APC systems incorporate "strong" precast moment connections and have adequate strength and ductility for seismic resistance, while, at the same time, maintaining certain beneficial aspects of precast construction. Additionally, the APC systems have the potential to be adapted into an automated construction environment.

The proposed APC systems are composed of two feasible alternatives: two-dimensional and three-dimensional systems. Both alternatives utilize two new precast connection schemes called the ATLSS beam-to-column and the ATLSS beam-to-beam connections. The ATLSS beam-to-column connection is further divided into two types of connections called the Modified ATLSS Passage-Type (MAPT) and the ATLSS Nodal-Type (ANT) connections. The MAPT connections have two configurations: the regular and the exterior. The ANT connections have three configurations: the regular, the exterior, and the corner.

## 3.2 Future Work

In addition to the conceptualization and development of the APC systems, the additional objectives of the overall study include the formulation of design methodologies for the precast beams, columns, and beam-column connections of the APC systems, further study of beam-to-beam connections of the APC systems, and the evaluation of the seismic performance of the APC systems using analytical methods.

In formulating the design methodologies, available design methodologies for reinforced concrete beams, columns, and beam-column connections are being reviewed. Suitable design alternatives will then be selected and applied to a prototype structure, which will incorporate the two-dimensional APC system and be assumed to be located in a moderate seismic zone. Experimental studies will be conducted to validate the selected design methodologies.

To evaluate the seismic performance, analytical studies will be conducted of APC systems designed using the proposed design methodologies for frame structures that incorporate the MAPT and ANT connections. The analyses will involve both inelastic static and inelastic dynamic, and natural period analyses.

# REFERENCES

[1-1] Parry, K. D.
**DESIGN AND EXPERIMENTATION OF AN INNOVATIVE PRECAST CONCRETE MOMENT CONNECTION FOR MODERATE SEISMIC RESISTANCE**, Thesis presented to Lehigh University, Bethlehem, Pennsylvania, Dec. 1990, in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

[1-2] Michalerya, W. D.; and Viscomi, B. V.
**AN ADVANCEMENT IN AUTOMATED CONSTRUCTION: THE ATLSS INTEGRATED BUILDING SYSTEMS**, 3rd International Conference on Fossil Plant Construction, October 26-28, 1993, Palm Beach, Florida.

[1-3] American Concrete Institute (ACI)
**BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318)**, 1989 Edition, ACI, Detroit, MI, 1989.

[1-4] Seckin, M.; and Fu, H. C.
**BEAM-COLUMN CONNECTIONS IN PRECAST REINFORCED CONCRETE CONSTRUCTION**, ACI Structural Journal, ACI, Vol. 87, No. 3, May-June 1990, pp. 252-261.

[1-5] Mast, R. F.
**A PRECAST CONCRETE FRAME SYSTEM FOR SEISMIC ZONE FOUR**, PCI Journal, Precast / Prestressed Concrete Institute (PCI), Vol. 37, No. 1, Jan.-Feb. 1992, pp. 50-64.

[1-5a] Chen, C. C., and Lu, L. W.
**DEVELOPMENT AND EXPERIMENTAL INVESTIGATION OF A DUCTILE CBF SYSTEM**, Proceedings, 4th U.S. National Conference on Earthquake Engineering, Palm Springs, CA, May 1990, Vol. 2, pp. 575-584.

[1-6] Iverson, J. K.; and Hawkins, N. M.
**PERFORMANCE OF PRECAST/PRESTRESSED CONCRETE BUILDING STRUCTURES DURING NORTHRIDGE EARTHQUAKE**, PCI Journal, PCI, Vol. 39, No. 2, Mar.-Apr. 1994, pp. 38-55.

[1-7] French, C. W.; Hafner, M. J.; and Jayashankar, V.
**CONNECTIONS BETWEEN PRECAST ELEMENTS- FAILURE WITHIN CONNECTION REGION**, ASCE Journal of Structural Engineering, American Society of Civil Engineers (ASCE), Vol. 115, No. 12, Dec. 1989, pp. 3171-3192.

[1-8] French, C. W.; et al
**CONNECTIONS BETWEEN PRECAST ELEMENTS- FAILURE OUTSIDE CONNECTION REGION**, ASCE Journal of Structural Engineering, ASCE, Vol. 15, No. 2, Feb. 1989, pp. 316-340.

[1-9] Hafner, M. J.; and French C. E.
**MOMENT RESISTANT CONNECTIONS BETWEEN PRECAST ELEMENTS SUBJECTED TO CYCLIC LATERAL LOADS**, Structural Engineering Report No. 87-08, Department of Civil and Mineral Engineering, Institute of Technology, University of Minnesota, May 1988.

[1-10] Cheok, G. S.; and Lew, H. S.
**PERFORMANCE OF 1/3-SCALE MODEL PRECAST CONCRETE BEAM-COLUMN CONNECTIONS SUBJECTED TO CYCLIC INELASTIC LOADS**, NISTIR 4433, National Institute of Standards and Technology (NIST), US Department of Commerce, Gaithersburg, MD, Oct. 1990.

[1-11] Cheok, G. S.; and Lew, H. S.
**PERFORMANCE OF 1/3-SCALE MODEL PRECAST CONCRETE BEAM-COLUMN CONNECTIONS SUBJECTED TO CYCLIC INELASTIC LOADS- REPORT NO. 2**, NISTIR 4589, NIST, US Department of Commerce, Gaithersburg, MD, June 1991.

[1-12] Cheok, G. S.; Stone, W. C.; and Lew, H. S.
**PERFORMANCE OF 1/3-SCALE MODEL PRECAST CONCRETE BEAM-COLUMN CONNECTIONS SUBJECTED TO CYCLIC INELASTIC LOADS- REPORT NO. 3**, NISTIR 5246, NIST, US Department of Commerce, Gaithersburg, MD, Aug. 1993.

[1-13] Cheok, G. S.; and Stone W. C.
**PERFORMANCE OF 1/3-SCALE MODEL PRECAST CONCRETE BEAM-COLUMN CONNECTIONS SUBJECTED TO CYCLIC INELASTIC LOADS- REPORT NO. 4**, NISTIR 5436, NIST, US Department of Commerce, Gaithersburg, MD, June 1994.

[1-14] Stone, W. C.; Cheok, G. S.; and Stanton, J. F.
**PERFORMANCE OF HYBRID MOMENT-RESISTING PRECAST BEAM-COLUMN CONCRETE CONNECTIONS SUBJECTED TO CYCLIC LOADING**, ACI Structural Journal, ACI, Vol. 92, No. 2, Mar.-Apr. 1995, pp. 229-249.

[1-15] International Conference of Building Officials (ICBO)
**UNIFORM BUILDING CODE (UBC)**, 1991 edition, ICBO, Whittier, CA, 1991.

[1-16] Dolan, C. W.; Stanton, J. F.; and Anderson, R. G.
**MOMENT RESISTANT CONNECTIONS AND SIMPLE CONNECTIONS**, PCI Journal, PCI, Vol. 32, No. 2, Mar.-Apr. 1987, pp. 62-74.

[1-17] Stanton, J. F.; et al.
**MOMENT RESISTANT CONNECTIONS AND SIMPLE CONNECTIONS**, Research Project No. 1/4, PCI, Chicago, 1986.

[1-18] Precast/Prestressed Concrete Institute (PCI)
**DESIGN AND TYPICAL DETAILS OF CONNECTIONS FOR PRECAST AND PRESTRESSED CONCRETE**, 2nd Ed., PCI, Chicago, 1988.

[1-19] Park, R.
**PRECAST CONCRETE IN SEISMIC-RESISTING BUILDING FRAMES IN NEW ZEALAND**, Concrete International, American Concrete Institute (ACI), Vol. 12, No. 11, Nov. 1990, pp. 43-51.

[1-20] Park R.
**A PERSPECTIVE ON THE SEISMIC DESIGN OF PRECAST CONCRETE STRUCTURES IN NEW ZEALAND**, PCI Journal, PCI, Vol. 40, No. 3, May-June 1995, pp. 40-60.

[1-21] Charleson, A. W.; et al
**GUIDELINES FOR THE USE OF STRUCTURAL PRECAST CONCRETE IN BUILDINGS**, Report of a Study Group of the New Zealand Concrete Society and the New Zealand National Society for Earthquake Engineering, Centre for Advanced Engineering, University of Canterbury, Christchurch, New Zealand, August 1991.

[1-22] Yoshizaki, S.; et al
**EXISTING PRECAST FRAME SYSTEMS IN JAPAN**, First Meeting of the US-Japan Joint Technical Coordinating Committee on Precast Seismic Structural Systems (JTCC-PRESSS), San Diego, CA, November 29, 1990.

[1-23] Kurose, Y.; Nagami, K.; and Saitio, Y.
**BEAM-COLUMN JOINTS IN PRECAST CONCRETE CONSTRUCTION IN JAPAN**, Design of Beam-Column Joints for Seismic Resistance, ACI Special Publication, SP-123, ACI, Detroit, MI, 1991.

[1-24] Priestley, M. J. N.
**OVERVIEW OF PRESSS RESEARCH PROGRAM**, PCI Journal, PCI, Vol. 36, No. 4, Jul.-Aug. 1991, pp. 50-57.

[1-25] Stanton, J. F.; Hawkins, N. M.; and Hicks, T. R.
**PRESSS PROJECT 1.3: CONNECTION CLASSIFICATION AND EVALUATION**, PCI Journal, PCI, Vol. 36, No. 5, Sep.-Oct. 1991, pp. 62-71.

[1-26] Nakaki, S. D.; and Englekirk, R. E.
**PRESSS INDUSTRY WORKSHOPS: CONCEPT DEVELOPMENT**, PCI Journal, PCI, Vol. 36, No. 5, Sep.-Oct. 1991, pp. 54-61.

[1-27] Priestley, M. J. N. (Editor)
**REPORT ON THE THIRD US PRESSS COORDINATING MEETING: SAN DIEGO, CA, AUG. 6-7, 1992**, Report No. PRESSS-92/02, Aug. 1992.

[1-28] Priestley, M. J. N.
**AN OVERVIEW OF THE US PRESSS PROGRAM,** Fourth Meeting of the US-Japan Joint Technical Coordinating Committee on Precast Seismic Structural Systems (JTCC-PRESSS), Tsukuba, Japan, May 16-17, 1994, Report No. PRESSS-94/03, May 1994.

[1-29] Andrew, J. (Vice President of Marketing, American Institute of Steel Construction, Inc.)
**PERSONAL TELEPHONE COMMUNICATION**, Nov. 10, 1995.