The 1994 T.R. Higgins Lecture: Composite Frame Construction

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SUMMARY

Over the last 25 years, innovative structural systems have evolved in tall building design whereby structural steel and reinforced concrete have been combined to produce a building with the advantages of each material, namely, the inherent stiffness and economy of reinforced concrete and the speed of construction, strength and light weight of structural steel.

This paper explores, through a series of recent case histories, why the designers of tall buildings in the United States, use composite frame structures. The advantages and disadvantages of this type of building system are addressed. Potential problems this type of structure poses to designers and builders, and the need for a clear understanding by the steel erector of the design assumptions, are pointed out.

The future of composite-frame construction may very well lie in the area of low-rise construction, particularly in high seismic zones. Phase Five of the United States-Japan Cooperative Research Program will focus on composite and hybrid structures. It is expected that this joint research effort will produce significant new information about the design and behavior of composite components and systems. A new chapter covering composite elements and systems will appear in the 1994 NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings by the Building Seismic Safety Council. This new design standard and the results of new research expected to come out of the United States-Japan program should afford the United States designers and builders the opportunity to expand the frontier of composite construction.

COMPOSITE FRAME CONSTRUCTION

by Lawrence G. Griffis, P.E.

HISTORICAL OVERVIEW OF COMPOSITE CONSTRUCTION

Although many modern students and practitioners of structural engineering tend to think that composite construction is a product of recent design and construction practice, it actually began just prior to the start of the twentieth century.

In the USA, composite construction first appeared in the year 1894 when both a bridge and a building were constructed. The bridge was the Rock Rapids Bridge in Rock Rapids, Iowa. A Viennese engineer named Joseph Melan obtained a patent for bending steel I-beams to the curvature of an arch and then casting them in concrete. He submitted calculations to verify his composite design. The building was the Methodist Building in Pittsburgh constructed using concrete encased steel floor beams. It so happens that a fire in a nearby building in 1897 spread to this structure and destroyed the contents but not the frame of the Methodist Building. Already, one of the advantages of construction frame construction was realized - namely fire protection.

As additional buildings and bridges were constructed using steel wrapped in concrete toward the end of the nineteenth century, a need for research testing arose to better understand the behavior. A set of systematic tests for composite columns was begun at Columbia University's Civil Engineering Laboratories in 1908. This was followed by tests of composite beams in the Dominion Bridge Company's fabrication shop in Canada by Professor H.M. McKay of McGill University in 1922-4.

The first record of composite construction appearing in a US building code was in 1930 when the New York City Building Code first allowed extreme fiber stresses of 138 MPa (20 ksi) rather than the 124 MPa (18 ksi) value traditionally allowed for noncomposite beams.

Shear connectors were also recognized in this early composite construction as an effective means to enhance the natural bond between steel and concrete. In 1903, Julius Kahn received a US patent on composite beams where shearing tabs in the beam flanges were bent upward to project into the slab. Different types of shear connectors have been proposed over the ensuing 90 years including some types still documented in the AISC *Manual of Building Construction.* It was in 1954 when welded headed metal studs were first tested at the University of Illinois. In 1956, at the completion of the tests, a formula for the design capacity of these connectors was published. The welded headed metal stud has now become the dominant method of transferring shear between steel and concrete. The first bridge to use these connectors was the Bad River Bridge in

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Pierre, South Dakota built in 1956. Also in 1956, IBM's Education Building in Poughkeepsie, New York became the first building to use headed stud connectors. The second floor was formed with a 38 mm deep, 0·6 mm thick steel composite deck, using wires welded to the top flutes of the deck to achieve composite action between the metal deck and the hardened concrete.

The widespread use of composite metal decks began to flourish in the 1950s in building construction. The metal deck acted as a form for the wet concrete thus reducing concrete formwork costs. The deck was shaped in such a manner as to ensure composite action so that it could serve as the positive one way reinforcement for the final hardened concrete slab. Composite action was first achieved through the use of wires welded to the deck. More recently, the standard way it is accomplished in modern composite decks is through embossments manufactured into the deck to achieve composite action with the concrete. The composite metal deck and the welded headed stud have gained such widespread popularity in modern building construction that it has become virtually the only system used in building floor construction for steel and composite frame buildings in the last 25 years. One of the first modern buildings using this technique of construction was the Federal Court House in Brooklyn, New York designed in 1960. Today almost all steel and composite framed buildings utilize this method of floor construction.

The first tall building boom occurred in the USA in the 1920s and 1930s when high rise structures such as the Empire State Building and the Chrysler Building were built. Many of these early vintage steel frames utilized the protection that the concrete afforded the frame when it was cast around it for resistance against fire and corrosion. Only until the 1960s with the advent of modern composite frame construction have engineers actively sought rational methods to take advantage of the stiffening and strengthening effects of reinforcing bars and concrete on the capacity of the embedded steel frame. The late Dr. Fazlur Kahn, in his early discussion of structural systems for tall buildings, first proposed the concept of a composite frame system[1,2] in the Control Data Building in Houston, Texas in 1970. Since that time composite frame construction has been utilized on many high rise buildings all over the world and its usage, with a composite column as the key element, is well documented in the work of the Council on Tall Buildings and numerous other publications[3-7].

FROM PRACTICE TO THEORY TO RESEARCH TO CODE DEVELOPMENT

The development of composite construction in the USA vividly exhibits the rather unique and backwards sequence of events leading to the widespread use of a new construction method. The first step involves the conception of a new idea that has the potential to save time and money in the final product. The first priority is to construct the system, one that usually has a very limited design experience extrapolated from a well known current theory. After the system has been developed and constructed, the design theory is refined to justify its widespread usage. Finally, usually after a considerable period of time, research is conducted to verify the design theories, and modify them as required. Only much later, often many years, are these practices codified to legalize what was already done. This practice, although scientifically illogical, is borne out of necessity and practicality and in the case of composite frame construction is still going on today.

MODERN COMPOSITE CONSTRUCTION

Over the past 25 years, numerous innovative composite floor and frame systems have developed in tall building design whereby structural steel and reinforced concrete have been combined to produce a building having the advantages of each material. The use of these systems has as its underlying principle, the combination of these two distinctive and different building materials to benefit from the advantages of both - namely, the inherent mass, stiffness and economy of reinforced concrete and the speed of construction, strength, longspan capability, and light weight of structural steel.

Composite frame construction can take on several forms as will be exhibited by examination of several different and distinct case histories. One form of composite frame construction utilizes a bare steel frame designed to carry the initial gravity, construction, and lateral loads until such time as the concrete is cast around it to form composite columns capable of resisting the total gravity and lateral loads of the completed structure. This construction sequence is shown schematically in Fig. 1. In the figure, the floor number refers to the number of levels above which concrete has encased the erection columns. With the erection guy derrick or crane positioned on the 10th level, steel for levels 11 and 12 is being set. On levels 9 and 10 the frame is being welded or final bolt tightening is occurring and metal deck is being placed. On levels 7 and 8 studs are being welded to the top of composite beams and welded wire fabric is being laid on the floor deck. At levels 5 and 6 concrete is being poured for the floor. On levels 3 and 4 composite column reinforcing cages are being erected and tied. On levels 1 and 2 column forms are being placed and concrete is being poured for the composite columns. Finished concrete floors are needed ahead of composite column and shear wall pouring in order to have a finished surface for stacking and teeing reinforcing steel and setting the column forms.

Experience gained from this type of construction indicates that, depending on the individual contractor, there exists an optimum construction sequence and spread in the various construction activities. If this relative staging is not maintained, then problems can occur. For instance, when the gap between setting steel and placing concrete beams becomes too wide, an overload of the erection columns can occur since they have been designed for a certain number of floors of construction loading or have been sized to limit column shortening to a predetermined amount. Also, frame stability can start to be of concern. If the gap becomes too close, then construction activity becomes congested with a resulting loss of construction time and efficiency. Obviously, close coordination and control of the construction process is required for this type of construction.

MODERN STRUCTURAL SYSTEMS IN HIGH RISE BUILDINGS

Prior to further discussion of the merits of composite frame construction, it will be helpful to review the family of structural systems that has evolved in the design and construction of today's high rise buildings. It will then be easy to understand how various forms and types of composite frame construction have been developed to respond to the factors controlling high rise buildings design.

From a systems point of view, the components of a high rise building can be conveniently divided into the following three categories:

floor systems,
lateral load resisting frame (columns and beams and/or walls),
column supporting gravity loads only.

FLOOR SYSTEMS

Numerous types of floor systems can and have been used as follows:

Structural steel systems:
open web steel joist/steel beam with form deck,
non-composite steel beams with form deck or composite metal deck,
composite steel beams with composite metal deck,
non-composite steel trusses with form deck or composite deck,
composite steel trusses with composite metal deck,
stub girders with composite metal deck.

Poured-in-place concrete systems:

one way pan joists and beams,
waffle slab and beams,
beam and slab,
flat slab with or without drop panels, column capitols and beams.

Each of the above systems can be reinforced with conventional mild steel or be post-tensioned with prestressed strand.

Precast concrete systems:
precast beam and slab,
precast double tees, single tees and/or channels.

Each of the above systems can be conventionally reinforced or pretensioned and can be designed noncomposite or composite with a topping slab.

LATERAL LOAD RESISTING FRAME/WALLS

Numerous forms of lateral frame resistance have been used as follows:

Shear walls:
- concrete shear walls (slipformed or jumpformed, conventionally reinforced or post-tensioned with high strength steel rods),
- composite shear walls (concrete walls with steel columns and/or beams),
- steel plate shear walls (steel plate only),
- composite steel plate shear walls (steel plate composite with concrete).

Braced frames:
- structural steel braced frame,
- concrete braced frame,
- composite steel and concrete braced frame,

Portal frames:
- concrete portal frames,
- structural steel portal frames,
- composite steel portal frames (composite columns with or without composite spandrel beams),
- concrete or structural steel portal frames with perimeter belt or outrigger trusses.
- Shear wall frame interaction.

Perimeter framed tubes:
- concrete tubes,
- structural steel tubes,
- composite tubes (composite columns with or without composite spandrel beams).

Perimeter braced or trussed tubes:
- structural steel braced tube,
- concrete braced tube,
- composite steel and concrete braced tube (composite columns with or without composite beams or braces).

Superframes or megaframes:
- structural steel superframe,
- concrete superframe,
- composite steel and concrete superframe.

Composite cladding systems.

The classification of lateral load resisting systems listed above should be considered as a broad general grouping only. Many different variations or combinations of each can be conceived limited only by the imagination of the designer in response to the myriad of building shapes used in modern architecture.

An example of one very common combination of structural systems can be found in the use of building core shearwalls with perimeter or interior concrete or steel portal frames. This particular system was listed separately in the listing above (shear wall frame interaction) because of its common usage in design today. The reader should be aware that many examples can be found in the buildings that utilize all different combination solutions such as steel braced frame with portal moment frame or perimeter tubes with core shear walls or core braced frames, etc.

COLUMNS SUPPORTING GRAVITY LOADS

Many times in high rise buildings selected columns are separated from the lateral load resisting frame and designed to carry vertical gravity loads only. This is particularly true when structural steel is selected to carry gravity loads and the beams and girders that frame into them are simply supported members with flexible connections. While this assumption can and has frequently been made in monolithic concrete construction, for example, where concrete core shearwalls are designed to carry 100% of the lateral loads and all columns and beams are designed to carry only gravity floor loads, the designer should be cautioned against this practice. Because of the monolithic nature of poured-in-place concrete and the resulting stiffness attained at beam and column joints, lateral load moments inevitably occur when sideway occurs. This contribution of stiffness can significantly alter the distribution of story shear forces in the structure. Thus practice undoubtedly was begun as a design simplification prior to the widespread usage of computers. While it has worked well for wind load design in areas of low seismicity it should not be employed in seismic zones without evaluating the affect of the lateral displacements on the beam column joints. This evaluation is now required in many building codes in seismic zones. Gravity load column types are listed below:

structural steel gravity columns,
poured-in-place concrete gravity columns,
precast concrete gravity columns,
composite steel and concrete gravity columns.

A detailed examination of several types of composite frame solutions will be made with case history examples later in this chapter. First, however, it is instructive to understand the motivation for the use of composite frame construction.

STRUCTURAL STEEL VERSUS CONCRETE

Structural steel has long been used in the design and construction of high rise buildings, ever since the advent of the skyscraper in the early 1900s. Its high strength has made it an ideal building material for heavy column loads, especially since the use of high strength steels have developed. Its light weight has allowed buildings to be taller while maintaining economical foundation costs. Its speed of construction has allowed rapid construction in all types of weather, especially important in construction financing in a world of high interest costs. Most of the world's tallest buildings are made of structural steel.

Early procedures used in the design of high rise buildings consisted of designing the frame for gravity loads (dead plus live load) and then checking the entire structure for lateral loads both for stresses induced by the wind and for drift or lateral sidesway. The initial design for gravity loads alone constitutes an optimum design since no less steel could be used for the building's height and span. Since the frame must also be designed for wind forces - both strength and stiffness, a considerable amount of material must be added. This additional quantity of steel has been labeled the 'premium for height' and is illustrated in Fig. 2. These curves, developed by the late Fazlur Kahn, show the quantity of 250 MPa steel required for a hypothetical 6 m bay building, designed only for gravity loads (lower curve), and the total steel quantity required when gravity and wind loads are considered (upper curve). The upper curve is based on a survey of a large number of portal framed rigid structures typical of the early generation of high rise buildings.

Structural steel quantities utilized in modern high rise structures are considerably less than shown in Fig. 2. The reasons for this are innovative structural systems such as braced and framed tubes, wind tunnel determination of design forces, high strength (50 ksi, 350 MPa) steels, increase in allowable stresses, computer analysis and design, weight reduction in building materials such as cladding and partitions and use of welding in place of rivets or bolts. A more representative picture of steel quantities used in tall building design today is shown in Fig. 3. The lowest curve represents the approximate 6 PSF (6000 kPa) required for floors using composite beam construction with nominal 12-2 m spans. This quantity is constant and is not a function of building height. The next curve up reflects an additional increment of weight required for columns supporting gravity loads only. This quantity increases linearly with height (approximately as $(N+1)/2$ where N is the number of floors). The next curve represents the total steel weight required for support of gravity and lateral loads. The increment from the previous curve represents the additional weight required for the lateral frame which increases approximately as the square of the height. Two additional curves are also shown for reference. The uppermost dashed curve represents structural steel weight (PSF) = number of floors (N) divided by three plus the quantity six. This 'rule of thumb' is often used as an upper bound solution for structural steel weight in modern high rise building design. Also shown is a curve passing approximately through the point defined by the Chicago John Hancock (braced tube) Building (30 PSF at 100 stories) and the Chicago Sears Tower (bundled tube building) currently the tallest building in the world (33 PSF at 110 stories). This curve is often taken as a measure of efficiency in the design since both buildings represent a near optimum design for their respective heights. Quite obviously these curves should be used as approximate comparative measures only since actual weights depend on many factors such as the windiness or seismicity of the building site, design drift ratio, design acceleration (perception to motion), building shape, surrounding terrain and grade of steel used in the design.

It is interesting to note that the structural frame and foundation typically constitute approximately 25% of the total building cost and that the lateral frame about one third of the structural cost or only 7-8% of the total building cost. Typically many design hours are spent by the structural engineer to trim every 'ounce of fat' in this portion of the building cost. By comparison, the exterior cladding can vary anywhere from one half to in excess of the total structural cost depending on its material and complexity.

Reinforced concrete has emerged in recent years as a viable building type in high rise buildings. The development of high strength readily available concrete strengths in the range of 48-131 MPa has afforded designers the opportunity to carry the heavy column loads required in high rise buildings with columns of reasonable size. New workability admixtures have eased placement problems in congested heavily reinforced columns. Hot and cold weather concreting practices have permitted concrete to be placed all year long in all kinds of weather at reasonable costs. New sophisticated forming systems have increased the speed of construction to comparable periods of structural steel. The development of lightweight aggregates has reduced the dead load of concrete affording the opportunity to build more floors. The extra mass of reinforced concrete structures has provided additional damping to reduce the problems of perception to motion in high rise buildings.

The lateral load design of high rise buildings requires consideration of lateral deflection (drift) for prevention of cladding and partition distress and acceleration to prevent disturbing perception to lateral motion. The components of drift consist of axial shortening of columns (chord drift), flexural deformation of beams and columns in unbraced frames, shear deformation in beams and columns of unbraced frames, axial shortening of diagonals and girders in braced framed (web drift), and shear deformation of the beam-column joints in unbraced frames (panel zone deformation). As the height to width ratio of a tall building increases from two to five or more, the percentages of drift caused by axial shortening of columns increases from 10% or 20% to well over 50%. Thus, the taller and more slender the building, the more pronounced becomes chord drift.

It is interesting to compare the relative cost effectiveness of steel and composite columns in providing the necessary strength and axial stiffness for tall building design. From a strength standpoint, composite columns are approximately 11 times more cost effective in resisting axial loads than are structural steel columns. From an axial stiffness standpoint, composite columns are approximately 8.5 times more cost effective in providing resistance to axial deformation than are structural steel columns. However, for resistance to a given axial load, structural steel columns are only 25% as large in area and weigh only 80% as much as reinforced concrete columns (See Table 1).

In comparing composite metal deck and reinforced concrete floor systems currently in use for high rise buildings having bays that are 9·14 m x 11·56-12·19 m, and utilizing lightweight concrete, it is interesting to note that the composite metal deck and beam floors weigh approximately 60% as much as the reinforced concrete floors (steel floors having 133 mm deck slabs, concrete floors with wide pan joists and haunch girders with a 100 mm slab). This significant difference in floor system weights for costs comparable to concrete floor costs can translate to large savings in foundation costs for tall buildings having composite metal deck floors.

Cross wind accelerations in tall buildings are frequently used as a measure of motion perception. Such accelerations are influenced by the variables as shown in the expression below:

$$A_W \ \alpha \ \frac{V_H^{2.6-3.3} * T^{1.1}}{\rho B * \beta_W^{0.5} * (WD)^{0.5}}$$

This proportionality states that maximum peak cross wind acceleration is directly proportional to the wind speed (V_H) at the building top to about the cubic power and to the building period (7) to about the first power, and inversely proportional to the building density (ρ_B) and the square root of the damping (β_W) and square root of the building's length and width. The building period is proportional to the square root of the mass divided by the stiffness. Note that building stiffness also enters the design for motion perception making the axial stiffness of composite versus steel further significant. In comparing damping furnished by steel versus composite buildings, it is presently believed that composite structures afford approximately 1 1/2 - 2% damping and structural steel 1% damping. For buildings of comparable periods the composite action will provide approximately 30% less acceleration than will structural steel alone although more research and study is required in this area.

ADVANTAGES OF COMPOSITE FRAME CONSTRUCTION

It is not hard to understand, based on the foregoing discussion, why the designers of tall buildings today are turning to composite frame construction.

Concrete can be used to economic advantage in carrying the large vertical column or shear wall loads at much lower cost per unit strength and stiffness. Steel floor construction is used to economically carry the floor loads for a reduced building mass and more economical foundation. Besides the economy of materials, composite frame structures have the advantage of speed of construction by allowing a vertical spread of construction activity so that numerous trades can engage simultaneously in the construction of the building. Structural damping from wind induced motions for composite structures can usually be justified at roughly 1.5% which is slightly greater than structural steel alone.

In evaluating the cost benefits of composite frame construction there are several factors that must be considered as follows:

1. Material cost savings in substituting concrete and reinforcing steel for structural steel.
2. Effect of time on construction schedule and the cost of construction financing.
3. Savings in fireproofing costs for the substitution of concrete for structural steel.
4. Cost and degree of reuse for the sophisticated concrete jump forming systems often used in composite frame construction.
5. Potential benefit to the owner, cost and otherwise, for earlier occupancy of the building.
6. Experience and expertise of the potential General Contractor(s) in building with composite frame construction.

7. Potential benefit or detriment of the composite frame on the building architecture, particularly the exterior cladding system. Oftentimes, the effect of unobstructed building views at the building perimeter and the effect on internal space planning and leasing is a major driving force on the selection of a structural system.
8. Cost of achieving floor levelness which is a very difficult problem to solve in all high rise buildings and made even more difficult in composite frames because of construction sequencing, shortening of light temporary erection columns and shrinkage and creep of composite columns and walls.
9. Practices and preferences of the local construction market, i.e. is the city a 'concrete town' or a 'steel town'.

It is not uncommon for the structural engineer to explore as many as 30 different combinations of steel and concrete systems in the early stages of design. Usually these are limited to three to six systems that are studied in great detail evaluating all of the above factors prior to a final system selection. This practice will be demonstrated in a case history study later in this paper.

The degree to which composite action is invoked determines the final quantity of structural steel on the project. Therefore, definitive statements and curves on steel unit weights as a function of building height are not meaningful until the degree of composite action is defined. For instance, in a framed perimeter tube solution are the columns only made composite or are the spandrels also made composite. Or, are shear walls also used in the core to replace interior steel core columns. All of these factors dramatically effect the unit weight of structural steel per unit floor area. One common technique often used by General Contractors or other party responsible for cost estimating is to convert the cost of concrete, formwork and reinforcing steel of the composite frame into an 'equivalent steel weight' by dividing that cost by the average in-place (fabrication and erection) cost of the structural steel and then adding that weight to the real steel tonnage on the project. This particular method almost always realizes a dramatic weight savings for a tall building if performed by an experienced and knowledgeable contractor in composite construction.

RETROFIT OF EXISTING BUILDINGS

With the high cost of real estate and the ever increasing costs of new construction many building owners and developers are looking for ways to renovate older buildings. There seems to be a certain magic and charm (along with a definite marketability) about tenants wanting to office in a renovated building whose design motif belongs to an earlier era. Also, many government and public buildings are being examined for ways to extend their useful life. In this area of retrofit and renovation, composite construction can really shine. When an older building in a high seismic zone is required to be in conformance with modern seismic codes composite retrofit of the existing frame may very well be the most economical answer.

The challenge for the structural engineer is to strengthen the existing steel or concrete frame while minimizing the disruption to the existing building services, layout, and function. In many instances only steel angle braces or steel plate shear walls can be slipped into existing wall cavities or behind existing building facades without major demolition. Steel in these instances can be very effective because of its lightweight, high strength, and its ease of handling and connectivity. At the same time, existing steel columns can be easily made composite by attaching shear connectors and wrapping them with reinforcing steel and concrete. The result will be the extension of useful life for an older building because it can be designed to conform to modern seismic or wind codes.

This concept is not just limited to seismic retrofit. Many older steel buildings that were designed only for strength without consideration of wind drift control are oftentimes made vulnerable when the old masonry core walls are removed or the building is recladded in glass or metal panel. In such cases, conversion to a composite frame can add the necessary stiffness to ensure serviceability under current codes and modern design practices.

WIND TUNNEL DESIGN OF BUILDING STRUCTURES

Perhaps the most significant advancement in high rise building design in recent years has been the use of wind tunnel studies in the routine office design of tall buildings. Many national and local building codes now recognize the use of wind tunnel studies as alternatives to code prescribed wind loads. Because of the high cost of constructing scale models, dynamic studies were once limited to only the tallest and elite of high rise buildings. However, with the advent of the high frequency force balance in several wind tunnel facilities around the USA and Canada, it is now possible to model routine structures in the wind tunnel at a relatively low cost that is within most building budgets. This has allowed a quantum jump in the understanding of wind forces on buildings and allowed engineers to use a much more rational approach to lateral load design.

Code prescribed wind loads tend to be conservative, particularly in light of the fact that the peak wind forces are assumed to come from any direction. This is generally very conservative in that one or two general wind directions tend to predominate for any given location with large reductions in wind forces from other less critical directions. This feature is now routinely part of the climatology studies performed in wind tunnel services. Acknowledgment of this fact alone allows the engineer to 'tune' the building frame design and place the material where it gives the most benefit. Evidence of the conservativeness in code wind loads and the potential savings possible in structural frame costs may be found in Fig. 4. This figure shows a plot of wind pressure versus building height for a 60 story composite frame building in Dallas, Texas named Momentum Place. As can be seen, wind pressure and reductions from 20% to as much as 80% combined to produce an overturning moment in the building approximately 40% less than code prescribed wind loads. Several reasons for this can be cited including worst wind directionally cited earlier, building shape, and shielding from the terrain and surrounding buildings. Reduction in design forces of this magnitude is not uncommon in wind tunnel studies for tall buildings.

Wind tunnel studies have also given the engineer vital information on accelerations at the top of the building under wind loads. This aspect of tall building design, perception to motion, often will govern the design of a tall building. Acceleration determination allows the engineer a much more rational approach to the problem than the conventional reliance on 'drift ratios'. Drift ratios can be a very misleading barometer for determining acceptance criteria of building perception to motion. Much more research is needed on establishing acceptable perception criteria for high rise building design and the wind tunnel will play a key role in that research. Figure 5 shows a typical plot of 10 year peak acceleration at the top of the building versus building period for two different damping ratios. Plots such as this one are made during the course of a typical wind tunnel investigation to aid the design engineer in an assessment of adding or reducing stiffness in the lateral load resisting frame. Several different damping values are usually used to bracket the results because damping values are probably the parameter known with the least amount of precision. Results such as these are rapidly determined by the wind tunnel investigator using a force balance scale model (usually 1:400) of the structure. A structural solution that produces a peak acceleration at the top occupied floor of less than 20-24 mG for a 10 year recurrence interval is developing into a common standard of acceptance for motion perceptibility.

DESIGN RESPONSIBILITY DURING ERECTION; TRADITIONAL STEEL FRAMES VERSUS COMPOSITE FRAME STRUCTURES

Historically, the structural steel erector is accustomed to working with steel frame structures that are stable and have their total lateral load resistance mobilized once each floor is placed and the braces or moment-connected beams and columns are welded or bolted up. This operation typically follows immediately behind, if not concurrently with, the frame erection. Since composite frames are not fully stable and completely lateral load resistant for the design loads until after concrete has been placed and cured some 10 floors behind, it is clear someone must be responsible for addressing frame stability during erection.

It is worthwhile to examine the AISC Code of Standard Practice[8] for guidance on the subject of erection design responsibility:

1.5.1 Often the owner provides the design, plans and specifications, the fabricator and erector are not responsible for the suitability, adequacy or legality of the design.

1.5.2 If the owner desires the fabricator or erector to prepare the design, plans and specifications or to assume any responsibility for the suitability, adequacy or legality of the design, he clearly states his requirements in the contract documents.

3.1 ...Structural steel specifications include any special requirements controlling the fabrication and erection of the structural steel.

3.1.2 Plans include sufficient data concerning assumed loads, shears, moments and axial forces to be resisted by members and their connections, as may be required for the development of connection details on the shop drawings and the erection of the structure.

7.1 If the owner wishes to control the method and sequence of erection, or if certain members cannot be erected in their normal sequence, he so specifies in the contract.

7.9.1 Temporary supports, such as temporary guys, braces, falsework, cribbing or other elements required for the erection operation will be determined and furnished and installed by the erector. These temporary supports will secure the steel framing, or any partly assembled steel framing, against loads comparable in intensity to those for which the structure was designed, resulting from wind, seismic forces and erection operations.

7.9.2 A self supporting steel frame is one that provides the required stability and resistance to gravity loads and design wind and seismic forces without any interaction with other elements of the structure. The erector furnishes and installs only those temporary supports that are necessary to secure any element or elements of the steel framing until they are made stable without external support.

7.9.3 A non-self supporting steel frame is one that requires interaction with other elements not classified as structural steel to provide the required stability or resistance to wind and seismic forces. Such frames shall clearly be identified in the contract documents. The contract documents specify the sequence and schedule of placement of such elements. The erector determines the need and furnishes and installs the temporary supports in accordance with this information. The owner is responsible for the installation and timely completion of all elements not classified as structural steel that are required for stability of the frame.

It is questionable whether the above statements in the AISC Code of Standard Practice[8] were written with composite frame construction in mind. However, several conclusions can be drawn from them in so far as they relate to composite frame construction:

1. The engineer, as the owner's design representative, is responsible for stating clearly in the contract documents the design assumptions used in sizing the bare composite frame. These assumptions should clearly show the required erection sequence with any load limitations (i.e., the maximum number of floors ahead that the steel erection may proceed from the finished concrete composite column installation). The bare composite frame may be viewed as a 'non-self supporting steel frame'. Clearly, the general contractor and erector must each be aware of the bare frame design assumptions and their effect on the timing and sequencing of the work so as to be able to submit a proper bid.

2. Once the design assumptions and erection sequence are defined on the contract documents, the erector is responsible for determining the required bracing and installing it as specified in Sect. 7.9.3 for a non-self supporting steel frame. However, many erectors will not assume responsibility for the erection stability of so complex a structure and are reluctant to bid under the terms as defined by AISC. The engineer-of-record has two choices in defining his role for the bare composite frame design. One, he can define the design criteria and assumptions used in sizing the bare composite frame for gravity loads only and require the general contractor to obtain a registered professional engineer to determine erection bracing required; or, he can design the bracing himself and so indicate it on the construction documents. The engineer's decision usually rests with his contractual arrangement with the architect or owner. Clearly, the engineer-of-record is the most appropriate person to determine the bracing requirements by virtue of his knowledge of the loads and familiarity with the structure. Practically speaking, time does not always exist in the normal design process for the erection bracing to be determined and shown on the construction documents. Regardless of which method is selected by the engineer-of-record, he must clearly define his intentions in the contract documents.

DESIGN CONSIDERATIONS FOR COMPOSITE FRAME STRUCTURES

Several factors must be considered in the design of the bare composite frame of composite frame structures:

Wind Load

A decision must be made on the wind pressures to use in the design of the frame and the effective building area over which to apply the wind load. It is becoming more common to design buildings for the 50-year storm, as specified in the ANSI A58.1-82 Building Code Requirements for Minimum Design Loads in Buildings and Other Structures[9]. Consideration may be given to reducing the wind pressures used in the design of the bare frame from those used in the completed building design, the rationale being to reflect the reduced exposure time (approximately 1 year for a 50-story building) for the design storm. With this idea in mind, some structures have been designed for a 25-year storm using the wind map present in the 1972 version of the ANSI A58.1 Code.

The design engineer should discuss this design issue with the owner. The question, of course, is how many dollars should be spent on a temporary structural condition, and the risk involved. Considerable judgment is involved, weighing cost, safety and risk. The designer must consider applying wind pressure, with the appropriate aerodynamic drift factor, to all elements of the structure, including the edge of the floor deck, beams, trusses, columns and any materials stored on the floors. This practice may produce design wind forces larger than those calculated using only the final projected area of the building.

Consideration also must be given to the design of structural framing elements for local wind load applied perpendicular to the surface of the element. This condition may control the design of cantilevering tree column elements prior to placement of the metal deck floors.

Diaphragm Action

Adequate consideration must be given by the design engineer to the ability of the floor diaphragm to distribute the wind load to the bracing elements. This warrants particular concern in the time period prior to placement of the concrete floor slabs. The floor deck must be attached to the steel frame with puddle welds or self-tapping screws, sufficiently to carry in-plane floor shear. In some areas of a floor or roof, temporary or permanent horizontal bracing may be required where the deck strength or stiffness are not adequate.

Removal of Temporary Bracing

The design engineer should make it clear on the contract documents as to what bracing (including connectors) and at what stage of construction the bracing may be removed to accommodate architectural or mechanical items that must be installed at a later date. Premature removal of temporary bracing could lead to overstress of the frame or out-of-plumb framing. Clear definition of these issues will avoid disputes and possible additional costs to the owner during construction.

Drift Criteria

The design engineer must give careful attention to drift criteria and lateral stiffness of the bare composite frame. Without temporary bracing, the lateral stiffness of the initial structure must be sufficient to provide overall stability including P-Delta affects.

Differential Column Shortening

Perhaps one of the most difficult tasks facing the designer of any tall building is the problem of differential column shortening. In steel buildings the problem comes about because columns carrying gravity load alone are loaded to a much higher stress level than columns that are part of the lateral load carrying system and whose design stress level is dictated by the combined action of gravity and wind/seismic forces. The problem is compounded by the fact that in tall building design the cross-sectional area is oftentimes increased to control building chord drift, resulting in an even lower axial stress under gravity loads alone. In steel buildings, the designer can reasonably predict the differential axial shortening (largely a bookkeeping exercise of $\sum PL/AE$ for each column) between columns and compensate for the difference in the fabrication process by intentionally making the heavier stress columns longer between floors. It is not uncommon on the structural drawings to see scheduled corrections of column lengths which serve as instructions to the fabricator in the fabrication process.

The problem of controlling differential column shortening to achieve a level floor is much more difficult to deal with in a composite frame. In many modern mixed use buildings, it is not uncommon to see concrete shear walls, composite columns, steel columns carrying only gravity load, and steel columns carrying gravity and lateral loads all in the same structure. Concrete introduces another variable into the already complicated equation of differential column shortening in tall buildings, namely, time. Concrete, although an excellent carrier of compressive forces, suffers from inelastic shortening due to creep and shrinkage in addition to the normal elastic axial deformations. Creep and shrinkage are very much a function of the concrete mix proportions as well as age at loading, curing conditions with time, volume to surface area ratios, humidity, concrete strength, and the stress level. Although rational methods exist to calculate these effects[10,11] and are used routinely in tall building design to deal with the problem, the calculations are estimates at best and the procedure far from an exact science. Because the shortening in various wall and column elements varies with time and occurs at different rates, one serious, well intended young engineer once asked his supervisor at what point in time his calculations should be aimed for in the shortening cycle. The supervisor wryly responded that the floors must be level when the owner signs the check for professional services.

Since composite columns often utilize a wide flange steel erection column that serves to carry 10-12 floors of construction loads until the concrete is poured around it, another variable is introduced - namely, a component of shortening occurring on the bare steel (usually a light W14 section) prior to concrete encasement. This shortening is a function of the construction gap between the top of the steel frame where erection is occurring and the point where concrete is poured encasing the bare steel columns below. This gap, subject to the whimsical gods of construction (weather, strikes, crane breakdowns, etc., etc.), has been known to vary widely in actual practice. The result of all this, despite the best intentions of the design engineer, has been floors as much as several inches out of level from one point to another in many tall buildings.

Perhaps the best way to deal with this problem is for the engineer to require monitoring of column splice elevations at four or six story increments by the contractor with subsequent 'corrections' by way of shims at the splices to compensate for the difference in calculated and field measured floor elevations. This procedure is oftentimes used in tall building construction at present.

ELASTIC, CREEP AND SHRINKAGE SHORTENING IN COMPOSITE FRAMES

The previous section cited column/wall shortening as an important design consideration for composite frames. Any objective treatment of the subject of composite frames must of necessity discuss this problem and rate it as one of the most serious drawbacks of this construction method and one that has caused serious if not unpublicized disputes among the building team including owner, developer, architect, engineer and contractor. Many knowledgeable and veteran contractors routinely up front exclude responsibility for level floors in their contract proposals with the owner. Discussion of the problem prior to beginning construction is important so that all parties know what to expect of the final construction tolerances. Unfortunately, since no party is in a position to confidently predict the outcome of floor levelness, the project can get off on a sour note. It has been the author's practice to encourage the owner to budget up front dollars to level floors (usually by floor grinding high spots and using self-leveling mortars for low spots) during tenant construction in recognition of the inevitable problem. A seemingly obvious solution to the problem is to incorporate a 2 in. topping slab into the design that is poured after frame construction is completed and most of the building load is applied to the vertical elements. While this solution would in most cases solve the problem, the cost is prohibitive because of the large increase in gravity load and resulting cost impact on column and foundation design. To the author's knowledge this solution has never been used in any major tall building design.

While the problem is a difficult one to deal with, it is certainly not impossible. The burden has been put on the structural engineer to predict and compensate for elastic, creep and shrinkage shortenings. A rational design process has evolved and is routinely practiced in tall building design. However, it must be recognized that the variables are many and their prediction at best approximate so that the final outcome is uncertain. This section will discuss briefly the variables and how they are used in predictions and compensation.

CAUSES OF UNLEVEL FLOORS

The problem of unlevel floors in tall buildings is complex and not due entirely from elastic, shrinkage and shortening of vertical elements alone. The contributors to this problem are listed below:

1. elastic shortening,
2. inelastic shortening (creep and shrinkage),
3. foundation settlement,
4. floor camber variation,
5. beam deflection (elastic and creep),
6. quality of the floor finishing operation,
7. inability to accurately measure floor levelness,
8. lack of understanding among the design team.

It is worthwhile to note that only recently have more scientific methods been implemented to define floor finish tolerances with the construction industry undergoing a marked change in recognizing and implementing the new standards.[12]

TYPICAL RANGE OF COLUMN SHORTENING

It is instructive to compare column shortening for a typical steel and concrete building and to compare the contributing components. A typical gravity loaded only column in a 244 m tall building with concrete strengths varying from 28 to 55 Mpa, shrinkage of 800 x 10^{-6} and a specific creep of 3/f'c would shorten as shown in Table 2. As the table shows, the concrete (or composite) column has a larger and more unpredictable shortening to deal with. A typical rule of thumb for the magnitude of shortening in a composite column is 25·4 mm for each 24·4 m of building height. Depending on concrete strengths and properties, the elastic shortening can be 25-35%, shrinkage 28-40% and creep 25-45% of the total shortening to occur. Shrinkage shortening is usually the largest component and creep shortening the most variable.

COMPENSATION FOR COMPOSITE COLUMN SHORTENING

The problem of compensation for column shortening is different in concrete columns, steel columns and composite columns. For steel columns the shortening is purely elastic and the calculation very simple (although tedious). At any level, the engineer simply sums for each load increment the quantity *PL/AE* where *P* is the load, *L* the clear story height, *A* the cross-sectional area and *E* the modulus of elasticity. Each floor and each load increment above the floor in question must be considered. The calculations are largely a bookkeeping exercise that lends itself nicely to computer solutions. The results are compared between exterior and interior columns and a 'compensation schedule' prepared to account for the difference in the column fabrication process. Usually adjustments are made in each tier (every other floor) or every other tier depending on building height. Compensation is usually in the order of 1·6 mm per tier. These shortening are made necessary in all steel buildings because columns carrying only gravity load are stressed higher than columns participating in the lateral frame. The problem is exacerbated when high strength steel (A572) is used for gravity loaded columns for economy sake and lower strength (A36) steel used in the lateral frame for drift control.

In all concrete buildings the problem is more complex and the calculation procedure different and more tedious. The calculations are separated at each floor into two parts. The first part is shortening that occurs in the column in question prior to installation of the floor in question. The second part is shortening that occurs after installation of the floor in question. The first part of the shortening, prior to floor installation, is of no practical importance because the contractor automatically levels the floor forms at each floor as the concrete is placed. The second part, that shortening occurring after the floor is placed, must be calculated so that the formwork can be adjusted if necessary to ensure a level floor. Depending on designer preferences, it may be decided that most of the time dependent shortening (shrinkage and creep) be compensated for at the time of construction. In such a case, the floor may have a reverse tilt at initial occupancy that will gradually disappear. On the other hand, the designer may decide to compensate for only that shortening that is expected to take place within 1 or 2 years after construction starts. Thus, at the end of that period, the floor will be level and from then on the remaining shrinkage and creep will cause the floor to tilt. The total shortening in concrete buildings is rarely of practical importance since cladding and elevator rails are built with adjustments in the design.

For composite columns utilizing temporary light erection steel columns designed for 10 floors or so of construction loads the calculation procedure is more complex yet and different from an all-concrete building. In this case because the steel columns are fabricated to exact lengths the preinstallation floor shortening is important since the floor is attached to connections on the steel column. To assure the proper floor elevation, the preinstallation length changes of these columns must be calculated and compensated for. The post-installation floor shortenings consisting of elastic, creep and shrinkage length changes must also be included and compensated for. This fact explains why column shortening is more a problem in a composite frame building than either a steel or concrete building.

ELASTIC SHORTENING

Elastic shortening in a composite column must be calculated for those loads on the base steel erection column and the composite column. Calculations must be made for shortening up to the floor in question and subsequent to installation of the floor. The general equation is as follows:

$$ES = \sum P*h/(At*E_c) = \text{elastic shortening}$$

where

P = sustained load increment,
h = clear column story height,
A_t = transformed area = $A_2 + (E_s/E_c - 1) * A_s$,
A_g = gross column area,
A_s = column steel area,
E_c = concrete modulus of elasticity = E_c (28 days) * T/4 + 0.85 * T),
T = time in days,
E_s = steel modulus of elasticity.

The summation is to be calculated for each load increment and for each floor above and below the floor in question.

SHRINKAGE SHORTENING

Shrinkage is defined as the dimensional change (strain) in concrete caused by evaporation of moisture from the surface. Shrinkage is independent of the stress level and time of construction. The general equation of shrinkage shortening is as follows:

$$SS = \sum \epsilon_s * h * V_{SS} * a_t * RHS * R$$

shrinkage shortening

where

ϵ_s = ultimate shrinkage,
h = clear column story height,
Vss = correction factor for volume/surface ratio of column
= $(0.037 * VS + 0.944)/(0.177 * VS + 0.734)$,
VS = volume to surface ratio of column,
α_t = correction factor for fraction of total shrinkage that has occurred
= $T^{0.6}/(10 + T^{0.6})$,
T = time at which shrinkage shortening is evaluated,

RHS= correction factor for relative humidity

= $1.4 - 0.010 * H$ $40 \leq H \leq 80$

= $3.0 - 0.030 * H$ $80 \leq H \leq 100$,

H = relative humidity (%)

R = correction factor for reduction in shrinkage from reinforcing steel

= $(1 - e^{-x})/(E_s * P * E_c)$

e = 2.718

p = A_s/A_g

x = $p * m/(1 + p * m)$

m = E_s/E_c

E_c = ultimate specific creep, corrected for age and volume to surface ratio.

Ultimate shrinkage values range from 500 to 800 x 10^{-6}. The summation is to be calculated for each floor above and below the floor in question.

CREEP SHORTENING

Creep is defined as the dimensional change (strain) in concrete due to sustained load. Creep can be divided into two parts. Basic creep occurs under moisture equilibrium and drying creep occurs from changes in moisture with the environment. Specific creep is defined as the ultimate creep strain per unit of sustained stress using a 28 day old loaded specimen. Specific creep is a linear function of stress up to 40% of the ultimate strength of the concrete which is a valid assumption for most structures. The principle or superposition of creep is an important postulate that makes its calculation possible and practical. The principle states that strains produced in the concrete at any time by a stress increment are independent of earlier or later stress increments. Each load increment causes creep strain corresponding to the strength/stress ratio at the time of its application. The general equation of creep shortening is as follows:

$$CS = \sum \in_c * (P / A_t) * h * VSC * age * a_t * RHC$$

= creep shortening

where

$\in_c$ = 28 day ultimate specific creep,

P = sustained load,

A_t = transformed area,

h = clear column story height,

VSC= correction factor for volume/surface ratio of column

= $(0.044 * VS + 0.934)/(0.1 * VS + 0.85)$,

VS = volume/surface ratio of column,

age = correction factor for actual age at loading different than 28 days

= $2.3 * T^{(-0.25)}$

T = time of loading,

a_t = correction factor for fraction of total creep that has occurred (same equation as for shrinkage),

RHC = correction factor for relative humidity

= $1.4 - 0.010 * H$,

H = relative humidity (%),

R = correction factor for reduction in creep from reinforcing steel (same equation as for shrinkage).

The range of ultimate specific creep values for 28 day loading is a low of 1.5/f'c to a high of 5/f'c depending on concrete characteristics. The summation is to be calculated for each load increment and for each floor above and below the floor in question.

There are many variables and factors that affect creep and shrinkage shortening including the following:

1. cement characteristics,
2. water/cement ratio,
3. aggregate characteristics and quantity,
4. cement paste characteristics,
5. concrete age,
6. column size and shape,
7. amount of reinforcement in column,
8. curing conditions,
9. relative humidity in service,
10. use of fly ash or silica fume in the mix.

Each of these variables and the fact that concrete properties are time dependent make the calculations approximate at best.

TYPICAL CALCULATIONS RESULTS

Shortening calculations involve repeated application of the above equations for each load increment and for each column segment above and below the floor in question. Remembering that many variables are time dependent makes computer application essential. Figures 6-10, taken from Ref. 11 for an 80-story composite building illustrate the calculation results in graphical form. Note that the final result is a compensation curve usually implemented on the design drawings in schedule form. This schedule is used by the contractor to adjust floor elevations to ensure a level floor.

LABORATORY TESTING PROGRAM

As previously stated, there are a great many variables in the design process for column shortening in tall building. Because of the great variation in concrete, it is highly recommended to implement a laboratory testing program to increase the confidence level of the procedure. For each different concrete mix in the vertical elements a testing program should be run that would require a minimum of twelve 6 in. x 12 in. (150 mm x 300 mm) standard test cylinders. The cylinders should be moist cured for 7 days and stored at 23 °C and 40% relative humidity. Six cylinders are tested for strength; two each at 28, 90 and 180 days. Two cylinders are used for shrinkage determination. Two cylinders are used for the creep determination. One cylinder (if required) is used for thermal expansion determination and one cylinder is kept as a spare. The creep cylinders are loaded at 28 days to the stress level expected in the structure but not to exceed 40% of the cylinder strength. Measurement for shrinkage should start after the 7 days moist curing period. All measurements should go on for a period of 1 year if possible with 90 day results used for design.

THE USE OF COMPOSITE COLUMNS IN COMPOSITE FRAME CONSTRUCTION

Definition of Composite Columns

Under the provisions of the new LEFD specification[13] composite columns are defined in Section I.1 as a 'a steel column fabricated from rolled or built-up steel shapes and encased in structural concrete or fabricated from steel pipe or tubing and filled with structural concrete'.

In order to qualify as a composite column under AISC guidelines the specification states that the cross-sectional area of the steel shape must comprise at least 4% of the total composite cross-section (Section I.2.1.). The specification states in the commentary that rolled shapes comprising a lesser proportion of the total composite column area should be designed under the rules for conventional reinforced concrete columns. This is oftentimes the case in practical composite frame construction.

Practical Uses of Composite Columns

Practical applications for the use of composite columns can be found in both low rise and high rise structures. In low rise structures such as a covered playground area, a warehouse, a transit terminal building, a canopy, or porte cochere it may be necessary or desirable to wrap a steel column with concrete for aesthetic or practical reasons such as architectural appearance or resistance to corrosion or for protection against vehicular impact. In such structures, it may be structurally advantageous to take advantage of the concrete encasement of the rolled steel column that supports the steel roof structure by designing the column as a composite column resisting the gravity loads and carrying lateral loads. In high rise structures composite columns are frequently used in the perimeter of 'tube' buildings where the closely spaced columns work in conjunction with the spandrel beam (either steel or concrete) to carry the lateral loads. In some recent high rise buildings, giant composite columns placed at or near the corners of the building have been utilized as part of the lateral frame to maximize the resisting moment provided by the building's dead load. Composite shear walls with embedded steel columns to carry the floor loads have also been utilized in the central core of high rise buildings. Frequently in high rise structures where floor space is a valuable and income producing commodity the large area taken up by a concrete column can be reduced by the use of a heavy embedded rolled shape to help carry the extreme loads encountered in tall building design. Sometimes, particularly at the bottom open floors of a high rise structure where large open lobbies or atriums are utilized as part of the architectural design, a heavy embedded rolled shape as part of a composite column is necessary to make the column work for its large load and unbraced length. A heavy rolled shape in a composite column is oftentimes utilized where the column size is restricted architecturally and where reinforcing steel percentages would otherwise exceed the maximum code allowed values.

Advantages, Disadvantages and Limitations

Advantages of composite columns can be listed as shown below:

1. Smaller cross-section than required for a conventional reinforced concrete column.

2. Larger load carrying capacity.

3. Inherent ductility and toughness for use in earthquake zones.

4. Speed of construction when used as part of a composite frame.

5. Fire resistance when compared to plain steel columns.

6. Higher rigidity when part of a lateral load carrying system.

7. Higher damping characteristics for motion perception in tall buildings when part of a lateral load carrying system.

8. Stiffening effect for resistance against buckling of the rolled shape.

Although numerous advantages exist for the use of composite columns as a structural element, several areas of concern face the designer in their use. In high rise composite frame construction, experience has shown a difficulty by the design engineer in controlling the rate and magnitude of column shortening of the composite column with respect to adjacent steel columns or shear walls. These problems and proposed solution were discussed in a previous section of this paper.

As with any column of concrete and reinforcing steel the designer must be keenly aware of the potential problems in reinforcing steel placement and congestion as it affects the constructability of the column. This is particularly true at beam column joints where potential interference between a steel spandrel beam, a perpendicular floor beam, vertical bars, joint ties, and shear connectors can all cause difficulty in reinforcing bar placement and a potential for honeycombing of the concrete. Careful attention must be given to the detailing of composite columns by the designer.

Analytical and experimental research is needed into several aspects of composite column design. One area requiring study is the need, or lack thereof, of a mechanical bond between the steel shape and the surrounding concrete. Several papers[14,15] have discussed this question but additional work is required to quantify the need for shear connectors with a practical design model that can be utilized in routine design office use. There presently is a question about transfer of shear and moment through a beam column joint. This concern is of particular importance for seismic regions where the effect of large cyclical strain reversals cause a serious degradation of the joint. Research has recently been completed at the University of Texas at Austin on physical test models to study various joint details in composite columns. A suggested detail for composite column joints is shown later in Fig. 14.

Longitudinal Reinforcing Bar Arrangement

Composite columns can take on just about any shape for which a form can be made and stripped. They can be square, rectangular, round, triangular or other with just about any corresponding reinforcing bar arrangement common to concrete columns. For use in composite frame construction, however, square or rectangular columns are the most practical with bar arrangements tending to place the vertical reinforcing bars at the four corners of the column (Fig. 11). This arrangement allows spandrel beams and a perpendicular floor beam to frame into the embedded steel shape without interrupting the continuous vertical bars and also generates the maximum design capacity for the column.

Although there are no explicit requirements for longitudinal bar spacing in the LRFD specification, it is advisable to establish minimum limits so that concrete can flow readily in spaces between bars and between bars and the embedded steel shape. Minimum spacing criteria will also prevent honeycombing and cracks caused by high bond stresses between bars. Past experience with reinforced concrete columns has shown that the requirements established by the ACI 318 Code have proven satisfactory performance. These spacing and cover requirements are listed below (Fig. 11):

1. Minimum concrete cover over vertical bars and ties shall be 38 mm (LRFD Specification, Section I.2.1.b).

2. Clear distance between longitudinal bars shall not be less than 1 1/2 bar diameters or 38 mm minimum.

3. The clear distance limitations apply also to contact lap splices and adjacent bars.

4. Clear distance between longitudinal bars and steel shape shall be 1 1/2 bar diameters or 38 mm minimum.

Ties

Reinforcing steel cages (longitudinal bars and ties) must usually be set after and around the steel column. Because the steel column is erected in an earlier erection sequence, only open U-shaped ties are suitable for composite columns. Ties are used to provide lateral stability of the longitudinal bars and confinement of the concrete. The requirements of the LRFD specification and certain requirements of the ACI 318-83 code not specifically addressed by the LRFD specification should be satisfied as follows:

1. The cross-sectional area of the tie shall be at least 0.178 mm^2/mm of tie spacing (LRFD Specification I.2.1.b).

2. The spacing of the ties shall not be greater than 2/3 of the least dimension of the cross-section (LRFD Specification I.2.1.b).

3. The spacing of ties shall not be greater than 16 longitudinal bar diameters or 48 tie bar diameters (ACI 318-83 Section 7.10).

4. Ties shall be at least 12 mm in size for 35 mm, 48 mm and 57 mm, and bundled longitudinal bars, and 9-5 mm in size for all other bars (ACI 318-83 Section 7.10).

5. Ties shall be arranged such that every corner and alternate bar shall have lateral support provided by a corner of a tie with an inclusive angle of not more than 135° and no bar shall be further than 152 mm clear on each side along the tie from such a laterally supported bar (ACI 318-83 Section 7.10).

6. A lap splice of two pieces of an open tie shall be at least equal to 1.7 times the tensile development length for the specified yield strength (ACI 318-83 Section 12.2).

Suggested details for composite column ties are shown in Fig. 12-14.

Longitudinal Reinforcing Bar Splices

The requirements for splicing vertical longitudinal reinforcing bars for composite columns shall follow the same rules as apply for conventional reinforced concrete columns as specified in Chapter 12 of the ACI 318-83 Code. Several additional comments should be made for composite columns. First, additional vertical longitudinal restraining bars (LRFD Specification 1.2.l.b) should be used between the corners where the continuous load carrying bars are located in composite frame construction. These bars usually cannot be continuous because of interruption with intersecting framing members at the floor line. They are often required to satisfy the spacing requirements for vertical longitudinal bars shown below:

> The cross section area of… longitudinal reinforcement shall be at least equal to 0.178 mm^2/mm of bar spacing (LRFD Specification 1.2. l.b).

Second, it is suggested that in high rise composite frame construction that the vertical bar splices be located at the middle clear height of the composite column. This point is usually near the inflection point (zero moment) of the column where the more economical compression lap splices or compression butt splices may be used instead of the more expensive tension lap or tension butt splices that may be required if splices were made at the floor line.

A suggested composite column splice detail is shown in Fig. 12.

Steel Beam to Embedded Wide Flange Connection

The use of composite columns in composite frame construction often utilizes steel spandrel and/or perpendicular floor beams framing into the column at the floor level. Sometimes these beams will be simply supported floor beams where conventional double angle framed beam connections (PART 5 - LRFD Manual) or single plate shear connections may be utilized. More often, however, the steel spandrel beams will be part of the lateral load resisting system of the building and require a moment connection to the composite column. Practicality will often dictate that the larger spandrel beam be continuous through the joint with the smaller erection column interrupted and penetration welded to the flanges of the spandrel beam. To increase the speed of erection and minimize field welding, the spandrel beam and erection column are often prefabricated in the shop to form 'tree columns' or 'tree beams' with field connections at midheight of column and midspan of spandrel beam using high strength bolts.

The engineer must concern himself with the transfer of forces from the floor beams to the composite column. For simply supported beams not part of the lateral frame, the simplest method to transfer the beam reaction to the composite column is through a standard double angle or single shear plate connection to the erection column. It is then necessary to provide a positive shear connection from the erection column to the concrete along the column length to ensure transfer of the beam reaction to the composite column cross-section. The simplest method to accomplish this is by the use of standard headed shear connectors preferably shop welded to the erection column. For moment connected spandrel beams, the beam shear and unbalanced moment must be transferred to the composite column cross-section. Different transfer mechanisms have been tested at the University of Texas at Austin[16,17]. One suggested detail is shown in Fig. 14.

Shear Connectors

As discussed in the previous section, it is necessary to provide a positive shear connection transfer from the floor beam to the embedded steel column when the beam connection is made directly to the embedded steel column. It is likely that a significant portion of this reaction can be transferred in bond between the embedded section and the concrete as reported in Ref. 14. An estimate of this value can be made from eqn (5) of Ref. 14 which is based on the results of a limited number of push tests in which a steel column is embedded in a concrete column.

$$P = \frac{3.6 * b_f (0.09 * f'c - 95) * l_c}{k}$$

where

P = service load capacity on the embedded shape,
b_f = steel flange width of embedded shape,
$f'c$ = concrete compressive strength (PSI),
l_e = embedded length of steel shape,
k = constant, 5.

Converting to an average Ultimate bond stress u using only the flange surfaces as being effective and applying a safety factor of 5 as reported in the tests,

$$u = \frac{P \times 5}{4 b_f l_e} = \frac{3.6 \times b_f (0.09 f'c - 95) l_e}{5} \times \frac{5}{4 b_f l_e}$$

$$u = 0.9(0.09 f'c - 95),$$

average ultimate bond stress in PSI (1)

If this equation is applied to a typical case of a W14 x 90 embedded column in 5000 PSI (34 500 kPa) concrete with a floor to floor height (h) of 13 ft (3.96 m),

$$u = 0.9\ (0.09 \times 5000 - 95) = 320 \text{ psi } (2206 \text{ kPa})$$

available shear transfer = $u \times h \times 4b_f$

$$\text{ultimate load, kips} = \frac{320 \times (13 \times 12) \times (4 \times 14.5)}{1000}$$

$$= 2895^k \ (12900 \text{ kN}) \qquad (2)$$

These results indicate that all of any typical floor reaction to the composite column could be easily transferred to the concrete in bond alone.

The above discussion considered the case where axial load alone is transferred from the embedded steel section to the concrete. For beam columns where high bending moments may exist on the composite column, the need for shear connectors must also be evaluated. Until such time as research data are provided, the following simplistic evaluation may be made. Assume a situation where a composite column is part of a lateral load resisting frame with a point of inflection at mid-column height with a plastic neutral axis completely outside the steel cross-section (Fig. 15). An analogy can be made between this case and that of a composite beam where shear connectors are provided uniformly across the member length between the point of zero moment and maximum moment. The ultimate axial force to be transferred between the embedded steel column and the concrete over the full column height is $2AFy$ where A is the steel column area and Fy is its yield strength. Assuming a bond strength is available in this case similar to the case of the push test eqn (1), then shear connectors would theoretically be required when *2AFy* is greater than the results of eqn (2). Taking as an example, a W14 x 90 erection column

$$2AFy = 2 \times 26.5 \times 36 = 1908^k \ (8486 \text{ kN})$$

Available shear transfer from bond (2) is,

$$4uhb_f = \frac{4 \times 320 \times (13 \times 12) \times 14.5}{1000}$$

$$= 2895^k \ (12900 \text{ kN}) \rangle\ 1908^k \ (8486 \text{ kN})$$

Again, it is shown that bond stress alone can transfer the shear between the embedded shape and the concrete, assuming no loss in bond as a result of tensile cracking present at high moments. The LRFD specification commentary in Section I.4. discusses design using the plastic stress distribution of the full composite cross-section and requires a transfer of shear from the steel to the concrete with shear connectors.

Until further research is conducted on the loss of bond between the embedded steel section and the concrete and more comprehensive push tests are run, the following suggestions are made with regard to shear connectors on composite columns:

1. Provide shear connectors on the outside flanges where space permits. Where space does not permit provide shear connectors on the inside flange staggered either side of the web.

2. Provide shear connectors in sufficient quantity, spaced uniformly along the embedded column length and around the column cross-section between floors to carry the greater of the following minimum shear transfer forces as applicable:

   a. the sum of all beam reactions at the floor level;
   b. whenever $P_u/\phi_c P_n$ is less than 0.3, a force equal to Fy times the area of steel on the tensile side of the plastic neutral axis in order to sustain a moment equal to the nominal flexural strength of the composite-cross section. 0.3 is used as an arbitrary point separating a composite column subjected to predominantly axial load and one subjected to predominately moment. Consideration must be given to the fact that this moment is reversible.

3. The maximum spacing of shear connectors on each flange should be 813 mm.

If minimum shear connectors are provided according to the guidelines identified herein, it is reasonable to assume compatibility of strains between concrete and embedded steel to permit higher strains than 0.0018 under axial load alone. This strain level has been identified in Ref. 18 and the LRFD commentary Section I.2.1 as the point when unconfined concrete remains unspalled and stable. Therefore, a slight increase in the allowable reinforcing steel stress from 380

Mpa corresponding to 0.0018 axial strain to 415 MPa would seem to be justified. The use of shear connectors also allows the full plastic moment capacity to be counted upon when $P_u/(\phi_c M_n)$ is less than 0.3 (LRFD Commentary 1.4) instead of the reduction specified in LRFD Specification Section I.4.

Suggested details for shear connectors on composite columns are shown in Figs. 12 and 13.

Base Plates

Normally a base plate for the embedded steel column of a composite column is specified to be the minimum dimension possible to accommodate the bolts and anchoring it to the foundation during the erection phase and to carry the erection loads. In so doing, the base plate will interfere the least possible amount with dowels coming up from the foundation to splice with the longitudinal vertical bars of the composite column. The design engineer must remember that any area of base plate assume to transmit axial load to the foundation from embedded steel column must not also be used to transmit concrete bearing stresses to the foundation from the concrete portion of the composite column. It may be necessary, depending on the size of the steel column and number of shear on it, to add additional foundation dowels to adequately transmit the foundation load carried by the concrete of the composite column.

COMPOSITE COLUMN DESIGN BY LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

In order to qualify as a composite column under the LRFD specification design procedure, the following limitations must be satisfied as defined in Section I.2.1:

1. The cross-sectional area of the steel shape, pipe or tubing must comprise at least 4% of the total composite cross-section.

2. Concrete encasement of a steel core shall be reinforced with longitudinal load carrying bars, longitudinal bars to restrain concrete and lateral ties. Longitudinal load carrying bars shall be continuous at trained levels; longitudinal restraining bars may be interrupted at framed levels. The spacing of ties shall be not greater than 2/3 of the least dimension of the composite cross-section. The cross-sectional area of the transverse and longitudinal reinforcement shall be at least 0.007 in^2/in (0.178mm^2/mm) of bar spacing. The encasement shall provide at least 1.5 in (38 mm) of clear cover outside of both transverse and longitudinal reinforcement.

3. Concrete shall have a specified compressive strength f'c of not less than 3 ksi (20 700 kPa) nor more than 8 ksi (55 100 kPa) for normal weight concrete and not less than 4 ksi (27 600 kPa) for lightweight concrete.

4. The specified minimum yield stress of structural steel and reinforcing bars used in calculating the strength of a composite column shall not exceed 55 ksi (379 000 kPa).

The required design strength P_u of axially loaded composite column is defined in Section E.2 and Section I.2.2. of the specification as follows:

$P_u = \phi_c P_n$, required axial strength

$P_n = A_s F_{cr}$, nominal axial strength (E2-1 modified)

For $\lambda_c \leq 1.5$

$$F_{cr} = (0.658^{\lambda_c^2}) F_{my} \quad \text{(E2-2 modified)}$$

For $\lambda_c > 1.5$

$$F_{cr} = \frac{0.877}{\lambda_c^2} F_{my} \quad \text{(E2-3 modified)}$$

$$\lambda_c = \frac{Kl}{r_m \pi} (F_{my} / E_m)^{1/2} \quad \text{(E2-4 modified)}$$

where

ϕ_c = resistance factor for compression = 0.85,

A_s = gross area of steel shape,

E_{my} = modified yield stress

= $F_y + C_1 F_{yr}(A_r/A_s) + C_2 f'c(A_c/A_s)$(ksi), (12-1)

E_m = modified modules of elasticity

= $E + C_3 E_c(A_c/A_s)$(ksi), (12-2)

F_y = specified yield stress of structural steel column (ksi),

E = modulus of elasticity of steel (ksi),

K = effective length factor,

l = unbraced length of column (in),

r_m = radius of gyration of steel shape in plane of buckling, except that it shall not be less than 0.3 times the overall thickness of the composite cross-section in the plane of buckling (in),

A_c = net concrete area (in^2) = A_g - A_s - A_r,

A_g = gross area of composite section (in^2),

A_t = area of longitudinal reinforcing bars (in^2),

E_c = modulus of elasticity of concrete = $W_c^{1.5}\,(f'c)^{1/2}$ (ksi),

W_c = unit weight of concrete (lbs/ft^3),

f'c = specified compressive strength of concrete (ksi),

F_{yr} = specified minimum yield stress of longitudinal reinforcing bars (ksi),

C_1 = 0.7,

C_2 = 0.6,

C_3 = 0.2.

The interaction of axial compression and flexure in the plane of symmetry on composite members is defined in Sections H.1.1, H.1.2 and I.4 as follows:

For $P_u/\phi_c P_n \geq 0.2$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{ny}} + \frac{M_{uy}}{\phi_b M_{my}}\right) 1.0 \qquad \text{(H1-1a)}$$

For $P_u/\phi_c P_n < 0.2$

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{ny}} + \frac{M_{uy}}{\phi_b M_{my}}\right) 1.0 \qquad \text{(H1-1b)}$$

where

P_u = required compressive strength (kips),

P_n = nominal compressive strength (kips),

M_u = required flexural strength (kip-in),

M_n = nominal flexural strength determined from plastic stress distribution on the composite cross-section (kip-in),

ϕ_c = resistance factor for compression = 0.85,

ϕ_b = resistance factor for flexure = 0.90.

P_{ex} = $A_s F_{my} / \lambda_{cx}^2$, elastic buckling load about the x-axis (kips),

P_{ex} = $A_s F_{my} / \lambda_{cy}^2$, elastic buckling load about the y-axis (kips),

The nominal flexural strength M_n is determined for the plastic stress distribution on the composite cross-section as shown in Fig. 15. The plastic neutral axis is first determined such that there is equilibrium of axial forces in the concrete, reinforcing steel and embedded steel column. The nominal flexural strength M_n is determined as the summation of the first moment of axial forces about the neutral axis.

In the determination of the concrete compressive axial force, a concrete compressive stress of 0.85f'c is assumed uniformly distributed over an equivalent stress block bounded by the edges of the cross-section and a straight line parallel to the plastic neutral axis at a distance $a = \beta_1 c$ where c is the distance from the edge of the cross-section to the plastic neutral axis, and,

β_1 = 0.85 for $f'c \leq 4$ ksi (28 MPa)

β_1 = $0.85\text{-}0.05\,(f'c\text{-}4) \geq 0.65$ for $f'c > 4$ ksi (28 MPa)

COMPARISON BETWEEN LRFD AND STRAIN COMPATIBILITY METHODS

Guidelines for the design of composite columns were first introduced into the ACI Building Code in 1971 (ACI 318-71). With the widespread use and popularity of composite columns in the 1970s and 1980s, many engineers designed composite columns according to these principles which are essentially the same ones used for conventional reinforcing concrete columns.

The current rules for designing composite columns by the ACI approach are found in ACI 318-83 Chapter 10. The method essentially is one based on the assumption of a linear strain diagram across the composite cross-section with the maximum failure strain at ultimate load defined as 0.003. With the maximum usable strain at failure defined and a strain diagram taken as linear, it is possible to generate strength capacities of the cross-section for each assumed location of the neutral axis. Strains at each location of the cross-section are converted to stress for the usual assumption of a linear stress-strain curve for reinforcing steel and structural steel. The first moment of forces in each element of concrete, structural steel and reinforcing steel is taken about the neutral axis to generate a point (axial load and moment) on an interaction curve.

A comparison between the strain compatibility approach and the new LRFD approach is shown in Figs. 16 (A,B,C). Interaction curves (axial load versus moment) are plotted covering the wide range of composite column sizes 711 mm square, 914 mm square, 1219 mm square steel column sizes (minimum of 4% of the composite column cross-section), and reinforcing steel percentage (1-4%) that are likely to be found in practice. Examination of these figures reveals the following about composite column design by the two methods:

1. The ACI approach yields curves that are parabolic in nature while the AISC curves are essentially bilinear.

2. The two methods yield pure moment capacities that are very close to each other. The maximum difference was approximately 15% with most values much closer than that. LRFD in all cases predicts higher moment values.

3. The two methods yield pure axial load capacities that are reasonably close when the steel column constitutes a small part of the total column capacity but are significantly different as the steel column becomes larger. With larger steel column sizes, the LRFD approach yields axial capacities as much as 30% larger than ACI. This comparison, however, is not very meaningful because the ACI approach essentially does not recognize pure axially loaded columns with its minimum eccentricity provisions.

4. Large differences in capacity are predicted (as much as 50%) for composite columns having small steel columns. The ACI method yields significantly larger axial loads for a given moment than the LRFD method. This difference is most striking in the intermediate range of the curve.

5. With larger steel columns the LRFD curve is mostly above (predicts higher values) the ACI curve. As the steel column section becomes lighter the ACI curve tends to be above the LRFD curve particularly in the middle ranges of eccentricity.

6. It can generally be stated that as the steel column becomes a larger portion of the total column capacity, design economy can be realized by designing using an LRFD approach. When the steel column becomes smaller (the column is more like a conventional concrete column) the ACI method is more economical in design.

CASE STUDIES OF COMPOSITE FRAME CONSTRUCTION

This section will describe several notable buildings using composite frame construction. Each building is unique in its own way but they all share the same common theme of combining the benefits of steel and concrete in some way to improve the cost and/or performance of each building.

Control Data Building, Houston, Texas

This building, designed by Skidmore Owings and Merrill and the late Dr. Fazlur Kahn, is only 20 stories tall and was built in 1969. Although far from spectacular by today's standards, this remarkable structure stands out for several reasons. First, it marked the beginning of the composite frame era. Secondly, it was Dr. Kahn's first use of composite construction that he later expanded to more locations and much greater heights. Thirdly, not only was it the first use of a composite rigid frame, but it also used another innovation - the precast concrete skin panels also double as the formwork for pouring concrete around the exterior steel erection columns and spandrel beams. The concept is shown in Fig. 17. In these early editions of composite frames, the steel erection columns were placed in-board of the concrete spandrel and composite column. Although more constructible this way, the intrusion of the erection column into the interior space is highly objectionable to developers and space planners so in most application today, the erection column is inside the composite column (refer to Fig. 13).

This landmark structure stand out not because of its height, but simply because it was first. This same concept was expanded to 52 stories shortly thereafter by Skidmore Owings and Merrill in the One Shell Square Building in New Orleans, Louisiana.

Three Houston Center Gulf Tower, Houston, Texas

The 52-story building in downtown Houston, Texas was structurally designed by Walter P. Moore and Associates with CRSS as the Architect. It is an example of a composite perimeter tube structure with exterior composite columns The erection column is inside the composite column) and structural steel spandrel beams. The erection column (W14 x 43) and heavy steel spandrels (W36) were shop fabricated in two story units as a 'tree column' and field bolted together at midspan of the spandrel beam using high strength bolts in a double shear web plate friction connection. No internal bracing or core walls were used in the lateral resistance. The tree column frame and floors were erected approximately 12 stories ahead of the concrete operation. A typical floor plan is shown in Fig. 18. Figures 12, 13, 14 and 19 show similar details as were used in the composite frame.

The structural system utilized only 15 psf (14 900 kPa) of structural steel and saved over 2,100,000 dollars (US) over an all steel scheme. Reference to Fig. 3 shows that approximately 4 psf (3970 kPa) in steel weight was saved by the use of exterior composite columns resulting in the dramatic cost savings.

First City Tower, Houston, Texas

Another slightly different variation on the composite frame structure can be seen in the 49-story First City Tower in downtown Houston. This particular structure uses composite columns on all four faces, with only the two short side faces having steel wide flange, moment connected wind girders acting integrally with the composite columns. Most of the lateral load resistance is provided by composite shear walls in the central core (Figs. 20 and 21). The building core was framed with steel erection columns and beams at the same time as the perimeter erection columns. The stub girder floor system and erection columns are erected first. Composite columns and shear walls are constructed 10-12 floors behind the steel frame. Concreting of the core walls was accomplished in a similar gang form fashion as in a conventional concrete building, with the columns and beams in the core encased in the shear wall concrete.

This structural system utilized only 12 psf (11900 kPa) weight of structural steel compared to approximately 18 psf (17900 kPa) that would have been required for an all steel building.

Momentum Place, Dallas, Texas

Momentum Place is a 60-story headquarters building for M Corp. in Dallas, Texas designed by Philip Johnson with John Burgee Architects. Joint developers were Cadillac Fairview Urban Development, Inc. and M Bank.

The 1.6 million ft^2 ($149000m^2$) office building over a 400,000 ft^2 (37200 m^2) underground garage utilizes a structural system of jump-formed perimeter corner shear walls with punched openings, perimeter composite columns, steel core bracing, and steel interior columns. Value engineering and pricing studies were performed by the engineer Walter P. Moore and Associates as part of The Datum/Moore Partnership, and the contractor, HCB Contractors, on four wind resisting systems and six floor systems. All the wind resisting systems were perimeter moment resisting 'tube' frames as follows:

A. All steel tube frames (columns and spandrels) with columns at 10 ft (3 m) centers in the corners, 25 ft (7.6 m) centers on the long sides and 15 ft (4.6 m) centers on the short sides.

B. All concrete tube frame with punched window openings at five ft centers at the corners and column spacing on the short and long sides the same as Scheme A above.

C. A composite column frame with column spacing the same as Scheme A.

D. A punched concrete wall at the corners like Scheme B with perimeter composite columns on the perimeter at the same spacing as Scheme A.

A total of six floor framing schemes were also studied including conventional composite beams spanning between the core and the perimeter at either 10 (3 m) or 15 ft (4.6 m) centers, a stub girder floor system, post-tensioned beams at 30 ft (9.1 m) centers with wide pan joists spanning between them, clear span wide pan joists spanning between the core and the perimeter, and concrete haunch girders at 30 ft (9.1 m) centers with wide pan joists spanning between them.

The relative wind framing systems cost comparison is shown in Table 3.

Scheme B would have added 3 months to the construction schedule. The Contractor estimated that Scheme D, which was built, saved 2.4 million dollars (US) over the all-steel Scheme A, 1.5 million dollars (US) over the composite Scheme C, and 400,000 dollars (US) over the all concrete Scheme B. The conventional composite steel floor beam scheme at 10 ft (3 m) centers was the most economical floor system (see Fig. 22). The cost of the complete building is estimated at \$65.0/SF (\$700 US/m^2) and the structural cost \$18.50/SF (\$199 US/m^2). 17500 t (15900000 kg) of structural steel was used for a unit weight of 17.5 psf (17400 kPa).

The concrete strength in the punched corner shear walls varies from 7,500 psi (51700 kPa) at the base to 5,000 psi (34500 kPa) at the top. Walls are a constant thickness of 18 in (457 mm). Reinforcing steel in the walls varies from No. 18 bars at the base to No. 7 bars at the top. Window openings in the walls allow for 18 in (457 mm) by 18-in (457 mm) columns at 5 ft 0 in (1.5m) centers with a 4 ft 6 in (1.37m) deep spandrel beam. Exterior composite columns are 32 in (813 mm) by 32 in (813 mm) with a W14 x 61 erection column and have the same concrete strength as the shear walls. Interior base columns at the core are built up 28-in (711 mm) by 28 in (711 mm) box sections using A572 Grade 42 steel. These columns transition to standard W14 rolled shapes above the 32nd level.

The perimeter composite columns were designed to proceed no more than 10 floors above the concrete encasement. The erectors use of two 100 t (91000 kg) guy derricks made it necessary to bring concrete shear wall work along no more than four floors behind the steel floor framing that braces it. The derricks were guyed to the concrete shear walls and the four floor differential provided the proper angle for the guy wires at the corners.

One of the most challenging problems in the design of the mixed system scheme was controlling differential column and wall shortening to ensure a level floor system. Differential shortening had to be considered between the corner shear walls, adjacent composite columns, and steel core columns. This column shortening consists of an elastic, creep, and shrinkage deformation. Laboratory testing determined the creep and shrinkage characteristics of the concrete mixes to be used. Estimates were made by the engineers for time and sequence of construction, volume to surface area ratios of the members and reinforcing steel percentages in order to calculate creep and shrinkage. Both elastic and inelastic components were computed at each floor level up to the time of casting and thereafter using a procedure proposed by Fintel and Kahn[10]

A 'compensation schedule' was prepared to direct the contractor to adjust the column lengths. The contractor, in turn, as construction progressed, was required to provide field measured slab elevations at each column at every fourth floor. With this data the engineers adjusted column elevations for subsequent floors using as-built conditions of the lower floors. In order to maintain a schedule of two floors per week, the contractor utilized four separate sets of single height custom made steel wall forms, one for each comer. The forms had to be jumped 12-16 h after casting, so a superplasticizer and accelerator were used to enhance early strength. Test cylinders were taken at the time of placement and placed in a box at the jobsite under job temperatures in order to determine the necessary strength for stripping. The cylinders were broken at 12 h and every 2 h thereafter until the required strength was achieved. Column and wall reinforcing steel was pretied on the ground and erected in place to speed up the construction process.

Wind tunnel studies were performed on the tower utilizing a force balance that rapidly furnished the engineer with design wind loads and accelerations for various combinations of building period, mass, stiffness, and damping. The wind tunnel tests produced lower wind loads than predicted by the local building code (see Fig. 4). The shape of the building, shielding effects of adjacent structures and a more refined application for statistical wind data all tended to combine to lower the wind forces felt by the building. Figure 5 shows plots of acceleration as a function of building period and damping that were used to arrive at a structural solution satisfying the desired acceleration of 20 mg. maximum for a ten year recurrence interval.

InterFirst Plaza, Dallas, Texas

This 72-story composite rigid frame structure was designed by architect JRJ Architects, Inc. and engineer Le Messurier/SCI Associates in association with Brockette Davis Drake Inc. The composite frame is yet another milestone in the evolution of composite construction. The 921 ft (280 m) high building is extremely slender with a 7.24 to 1 height to least width ratio. The entire structure is supported on only 16 perimeter composite columns set 20 ft (6.1 m) back from the building perimeter in order to eliminate any obstructions from the perimeter office views. The perimeter columns are 30 ft (9.1 m) on center and vary in size from 6 ft (1.8 m) x (6 ft (1.8 m) to 8 ft (2.4 m) x 8 ft (2.4 m) and utilize 10000 psi (68940 kPa) concrete. Refer to Fig. 23 for a typical floor plan. Lateral load resistance is provided by the 16 composite columns acting as a rigid frame with a two way vierendeel frames spanning across the entire building width in both directions. The building core was temporarily shored on falsework until the vierendeel frames were completed through the twelfth floor. At that time the falsework was removed and all building loads were transferred to the perimeter columns. Above the twelfth floor the steel frame was erected in self-contained four floor tiers with steel placed in a cambered position until welding was complete. The steel frame utilizes special 42 in (1067 m) deep wide flange rolled shapes from Luxemburg.

This composite frame is somewhat unique because it contains no interior columns at all. This required spanning enormous distances, 120 ft (36.6 m) to 150 ft (45.7 m) across the building to transfer gravity loads. Despite the seemingly high premium for this feature, the engineers proved that it actually was more economical overall because it put the dead weight of the building at the perimeter where it is more effective in resisting overturning from the lateral loads. It also provided the necessary stiffness to control lateral sway and building perception to motion. Steel unit weight was a respectable 25 psf (24500 kPa).

One Mellon Bank, Bank Center (Dravo Tower), Pittsburgh, Pennsylvania

This 54-story tower was designed by architect Welton Becket Associates and engineered by Lev Zetlin Associates. This all steel building utilizes a perimeter framed tube structural system with columns at 10 ft (3 m) centers. This structure, however, is very unique and represents a different class of composite frame action in that the perimeter steel tube is designed for strength only compared to most tall buildings where drift and perception to motion controls the design. This feature is made possible because the building facade was designed to add the necessary stiffness to that of the framed tube. The tower uses its steel plate face panels structurally to form a stressed skin tube which limits wind drift. Structural steel weight was thus reduced to 24.7 psf (24500 kPa) utilizing narrow W14 columns (A572 Grade 50 steel and W24 to W30 (A36 steel) spandrels. With the help of the stressed skin building drift was reduced to 1/590 times the height. Figure 24 shows a typical elevation and plan section of the perimeter frame. Because the facade is used only for deflection control and not for strength, it does not require fireproofing or flame shields.

Typical facade panels are three stories high by 10 ft (3 m) wide. They consist of 1/4 in (6.35 mm) to 5/16 in (7.94 mm) thick A36 steel face plates, stiffeners aligned with window edges and bent plates or angles at panel edges. A total of 6300 t (5714000 kg) of steel is contained in the skin. The panels required isolation from column shortening and design for thermal expansion.

This tower is unique in that it provides composite action with the building cladding thus providing yet another possible solution in the family of composite frames.

Bank of China Building, Hong Kong

The 1209 ft (369 m) tall Bank of China Building is the fifth tallest building in the world, the tallest outside the USA and the tallest composite frame structure in the world. Designed by architect I.M. Pei and engineered by Leslie Robertson this perimeter composite braced tube structure is probably the ultimate composite structure built to date.

Supported on only five megastructure composite columns, one of which is transferred out to the other four, they are connected together by giant diagonals that, together with the composite columns, provide the resistance to lateral and gravity loads. These diagonals are steel box members filled with concrete. The diagonals, intermediate minor columns, beams and stiffening trusses in different planes of the multifaceted building are connected together and encased in concrete of the composite columns to form a space frame or megaframe. Figure 25 schematically shows the composite frame form.

A prismatic structure rises from a 170 ft square base divided into four triangular quadrants along the main diagonals. Each quadrant is terminated in sequence until only a single triangular pinnacle is left at the top. Instead of a large, complex, three-dimensional, built-up welded connection at the intersection of the diagonals and columns, as in the John Hancock Building in Chicago, the members terminate inside a large concrete composite column. As a result, the concrete provides a shear transfer mechanism counterbalancing the member eccentricities of the diagonals. Steel details are greatly simplified and large tolerances can be permitted in the steel frame. Belt trusses (hidden in the building elevation) pick up intermediate columns along the perimeter and transfer the loads to the four remaining columns.

The composite frame weighs in at only 23 psf (22800 kPa) structural steel, a dramatic reduction from an all steel building. This is particularly efficient considering that Hong Kong is one of the windiest cities in the world.

THE FUTURE OF COMPOSITE FRAME CONSTRUCTION

Progress in the modern design and construction of high rise composite frame construction has been very dramatic since first introduced in the US in 1969. Unquestionably new development will occur. Recently, several tall buildings in Seattle, Washington have been built using large 10 ft (3 m) pipes filled with 19000 psi (131000 kPa) concrete. Several advances that could occur soon are:

1. Higher strength concrete to reduce column sizes and allow for taller buildings.

2. Development of more sophisticated forming systems to reduce construction time.

3. And perhaps most significantly, development of new and more efficient artificial damping systems that will reduce the perception to motion that usually governs tall building design today. Ultimately, the building lateral load frames may be designed for strength alone, reducing or eliminating the 'premium for height'.

Recently there have been numerous articles in the trade magazines relating the possibility of 200- or even 500-story buildings being planned in New York and Chicago. Whether these plans are economically feasible and will ever be built remains to be seen. One thing, however, is very likely. If such a building is ever designed and built, some form of composite frame construction will be its mainstay.

However, the real opportunity for advancing the art of composite construction may very well lie in the low rise arena. In the US at present, less than one half of one percent of the structural steel market is found in buildings over twenty stories. This is down from the glory days when it was in the order of 6 percent. Clearly, if composite construction is to continue with the momentum gained in the last twenty five years, there will need to be advancements and new development in low rise construction. The opportunity is there.

Phase 5 of the US-Japan Cooperative Research Program[19] sponsored by the National Science Foundation is currently underway. The focus of this research effort will be in the following areas:

1. New Materials, Elements and Systems.

2. Concrete Filled Steel Tube (CFT) Systems.

3. Reinforced Concrete (RC)/Steel Reinforced Concrete (SRC) Systems.

4. Reinforced Concrete (RC)/Steel Reinforced Concrete (SRC) Wall Systems

Research in these four areas will include the following topics:

1. Material and Component Studies.

    - Material behavior.
    - Interaction of materials within composite structural elements.
      Structural element behavior.
    - Scale effects, rate of loading, etc.

2. Subassemblages.

    Two and three dimensional behavior.
    Interaction between structural elements.
    - Connection behavior.

3. Complete Structure.

    - Overall behavior.
      Interaction between major structural systems.

4. Analytical Studies.

    - Modeling of behavior.
      Parametic studies.

5. Design Studies.

    Design implication.
    Design guidelines.

A common theme structure for the joint research effort in the two countries will be used to facilitate comparisons between various system types identified for research and to derive structural elements and subassemblages for detailed studies. Component and subassemblage studies will probably precede testing work on full systems. Physical research specimens will be not less than one half full size. It is envisioned that a period of five years will be needed to fulfill the major objectives set forth in this research program. It is very likely that significant knowledge will be gained in this program to allow momentum to continue in the area of composite construction.

In comparing US and Japanese practice in composite construction it is interesting to see the difference in focus, emphasis, and thrust of the research and applications. In the US, the emphasis has been on high rise construction where composite construction has allowed economy of materials, speed of construction and maximum utilization of labor trades. Applications have been mostly in non seismic zones where wind loads have controlled the design. In Japan, however, composite construction has expanded because of the perceived advantages it has in building in high seismic zones where the increase of stiffness, ductility, fire resistance capacity and the reduction in steel is important. Most of the Japanese applications are in low-rise and mid-rise construction. These differences were clearly pointed out in the joint workshop sessions[20].

Perhaps the US can learn from the Japanese experience in high seismic zones. In Japan, the numerous large construction companies are well equipped and funded for research in structural components and systems. As a result, numerous proprietary components and systems have been developed for use in the market place. Systems with longspan steel beams connecting to concrete filled steel tubes or to steel reinforced concrete columns or walls are very common in Japan. Clever connection details have been developed to make these systems practical (Figs. 26 and 27). These systems are designed according to structural standards published by the Architectural Institute of Japan and the Building Center of Japan. However, many of the new components and systems developed are not covered by the existing standards and so the composite research on the Japanese side is motivated to develop new standards.

In the US, the situation is more acute where virtually no standards have existed, until recently, covering the design of composite systems in high seismic zones. A composite task group of the Provisions Update Committee that is responsible for updating the NEHRP seismic provisions recently developed guidelines for composite construction. This new chapter in the 1994 edition of the NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings will mark the first attempt in the US to incorporate composite guidelines for seismic design. Since the NEHRP provisions are a resource document for the national building codes this new chapter is a significant milestone in expanding the use of composite construction in the US.

The challenge for US designers and builders is to develop new components and systems that take advantage of the ductility, toughness and redundancy that are inherent with steel and concrete composites. For low rise construction, the key may very well be in developing simple and economic connections between beams and columns or walls. Connections like the one shown in Fig. 28 have recently been analyzed and tested[21] and may also be applicable to concrete filled steel tubes. Perhaps standardized connections can be developed for composite systems that are similar in concepts to those developed by ATLSS[22]. These and other new connections may be able to allow composite systems to successfully compete with existing steel or composite systems in the market place.

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TABLE 1

STEEL VERSUS COMPOSITE COLUMNS

Axial (kips)	Steel/Conrete Ratio				
	Size	Weight	Stiffness	Cost Strength	Cost (stiffness)
3,000	0.82	0.83	1.29	11.2	8.64
5,000	0.60	0.82	1.28	11.1	8.64
10,000	0,58	0.82	1.25	10.9	8.64
30,000	0.59	0.82	1.27	11.0	8.64
50,000	0.57	0.82	1.28	11.1	8.64
100,000	0.60	0.82	1.27	11.0	8.64

Unit conversion: 1 kip = 4448 N.

TABLE 2

TYPICAL COLUMN SHORTENING (IN) 244 m BUILDING)

	250 Mpa steel	350 Mpa steel	Concrete
Elastic	4.6	6.2	3.5 (34%)
Shrinkage	-	-	3.6 (35%)
Creep	-	-	3.1(31%)
Total	4.6	6.2	10.2 (100%)

Unit conversion 1 in = 25.4 mm.

TABLE 3

System	Relative Cost
A. Steel columns/steel spandrels	1.24
B. Concrete columns and spandrels	1.04
C. Composite columns/steel spandrels	1.15
D. Punched concrete corner walls with composite columns and steel spandrel beams.	1.00

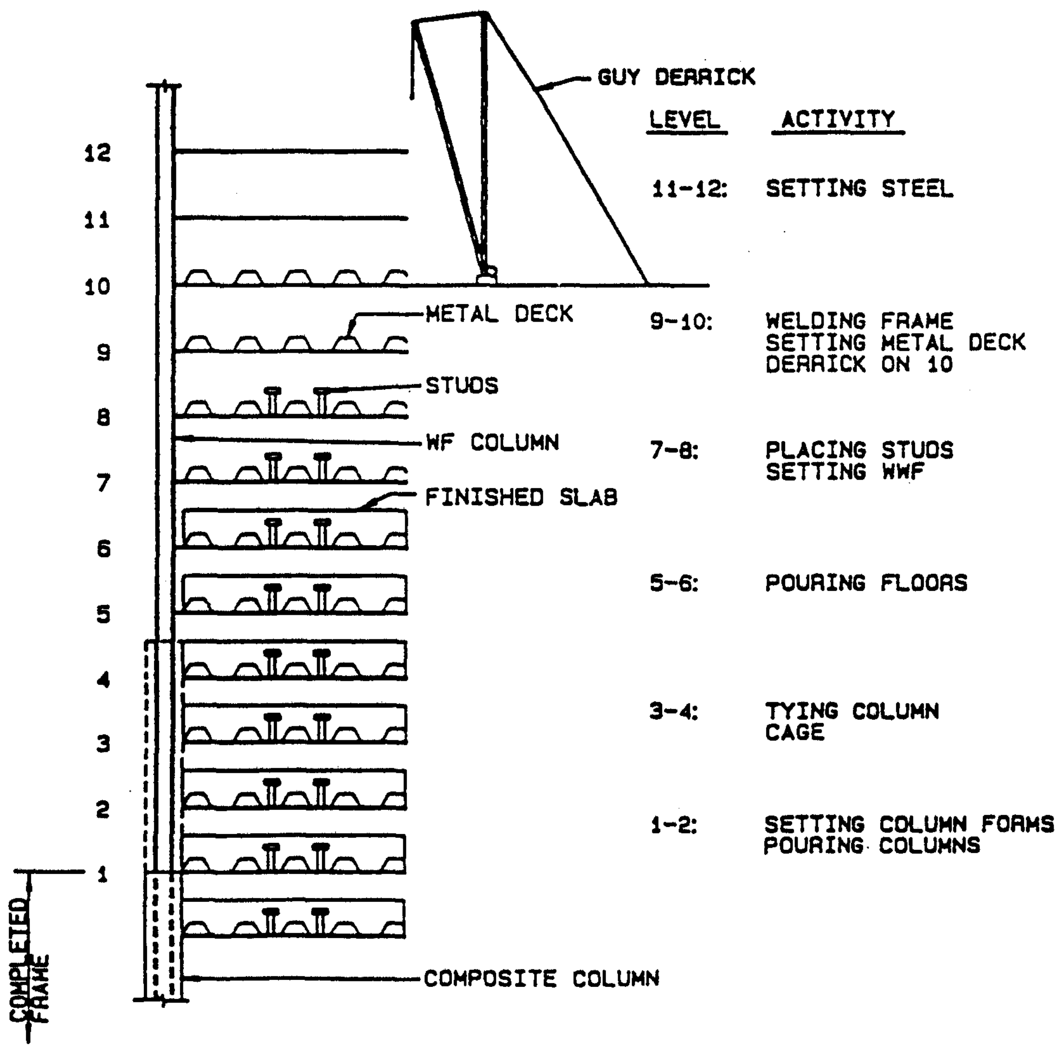


Fig. 1. Composite-frame construction sequence.

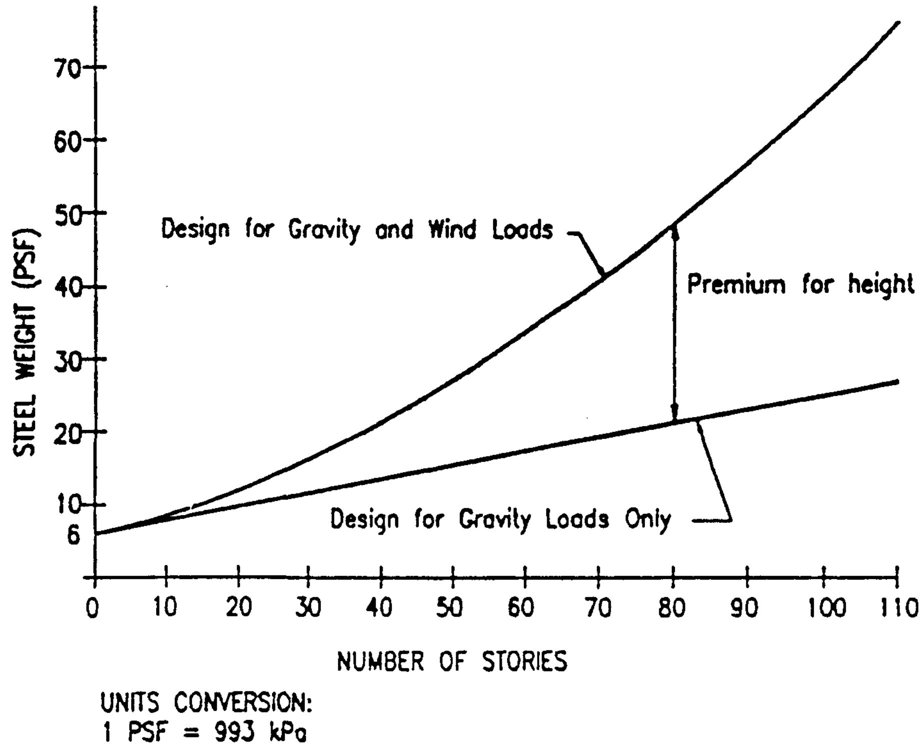


Fig. 2. Structural steel weight. Early tall building design.

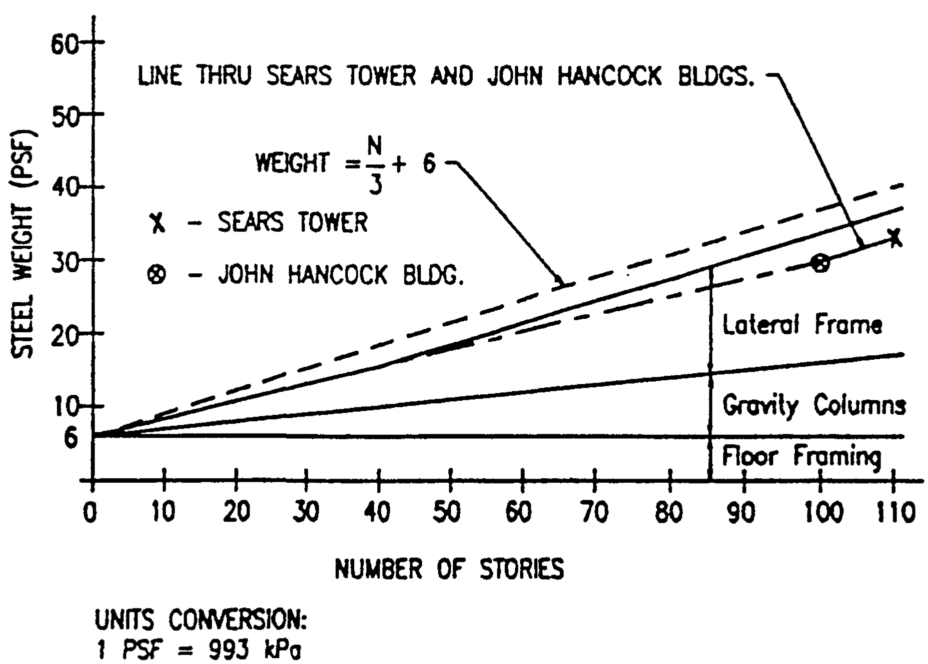


Fig. 3. Structural steel weight. Modern tall building design.

FINAL STRUCTURAL DESIGN-STRENGTH

CRITERIA: 100 YR. WIND TUNNEL
LOADS WITH DAMPING = .015

WIND LOAD COMPARISON:
A.N.S.I. VS. WIND TUNNEL

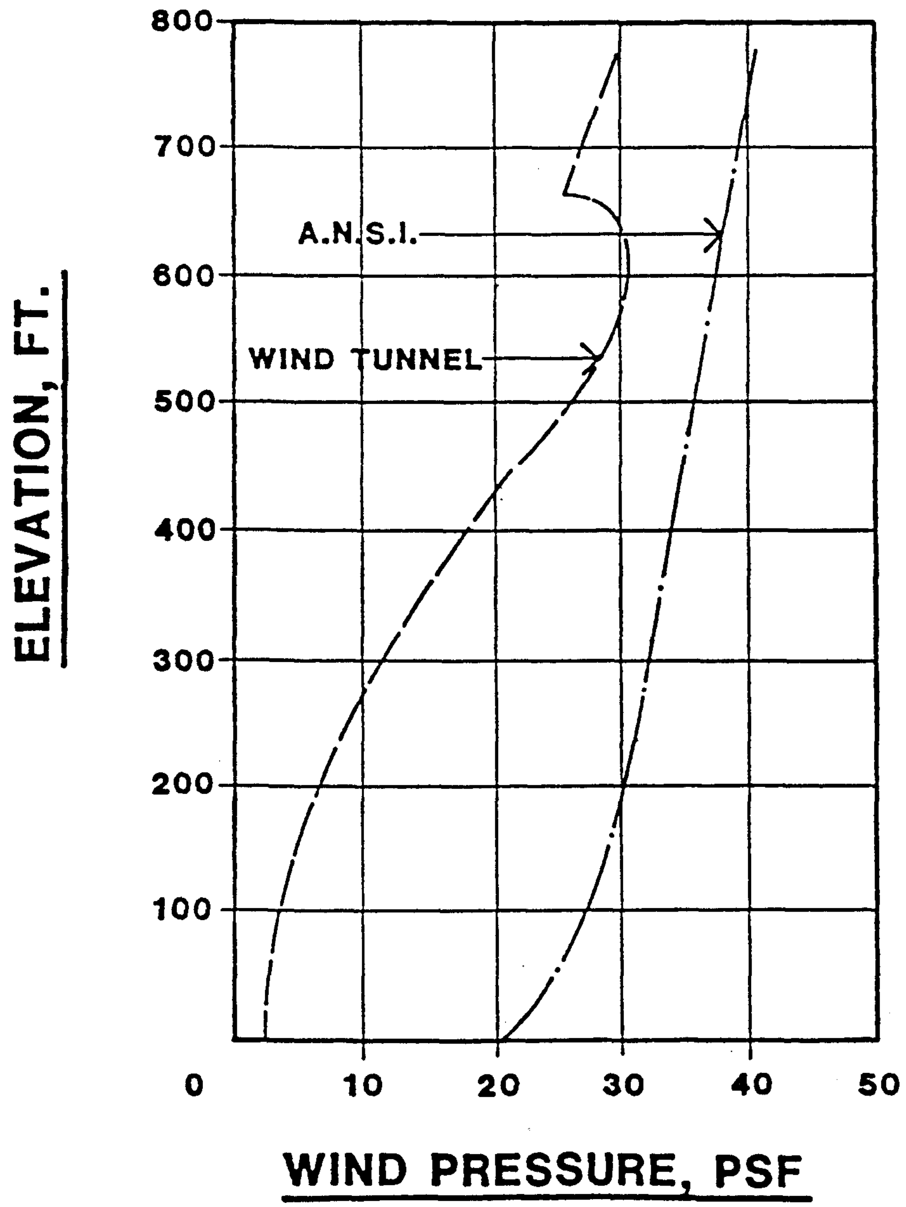


Fig. 4. Final structural design - strength.

FINAL STRUCTURAL DESIGN-SERVICEABILITY

BUILDING MOTION/ACCELERATION

CRITERIA: 20 MILLI-G AT 10 YRS.

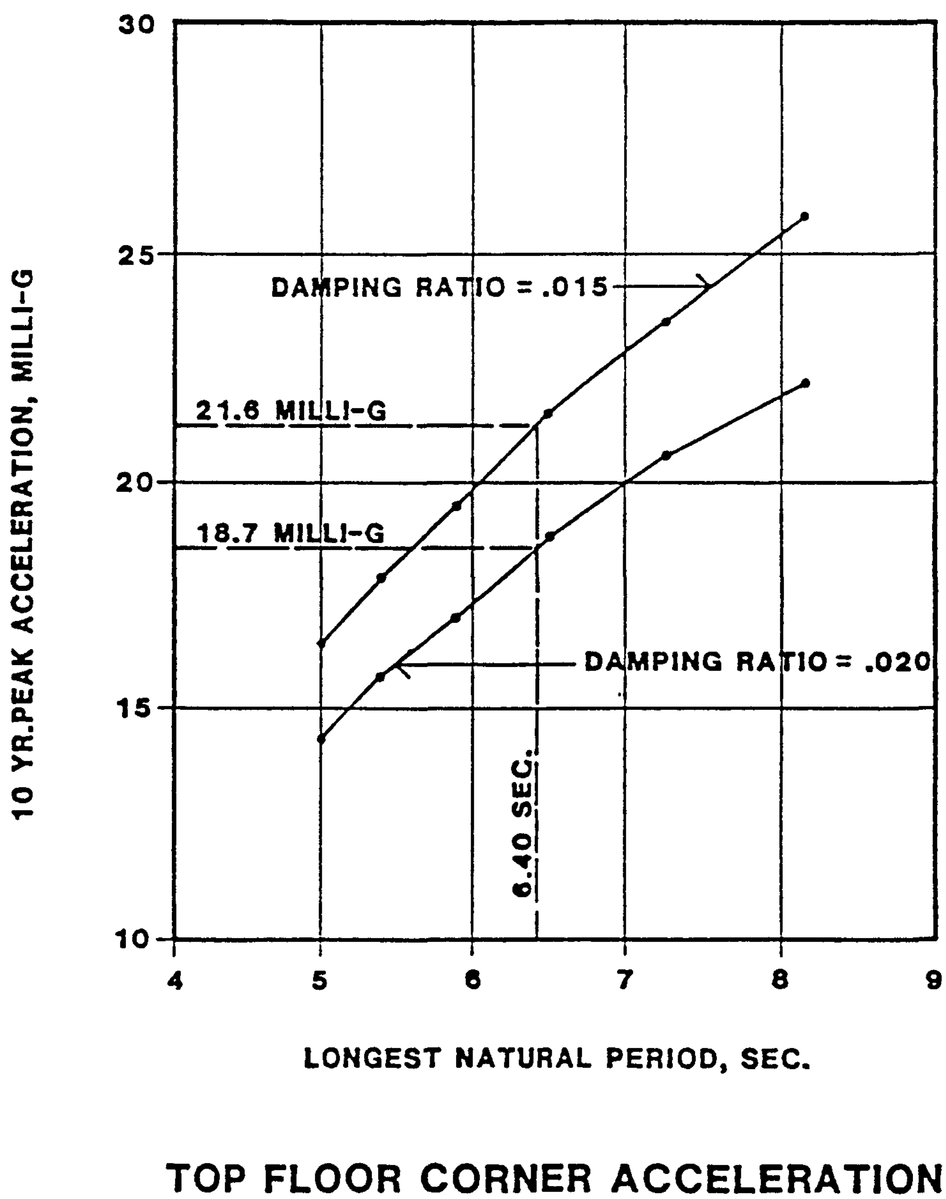


Fig. 5. Final structural design - serviceability.

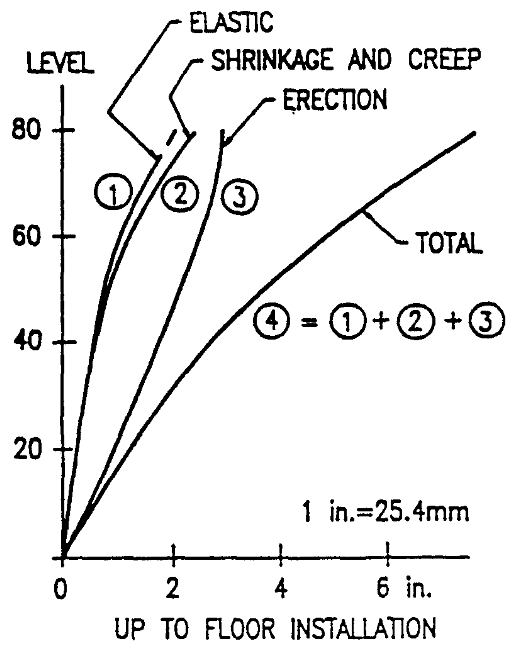


Fig. 6. 80-story composite building - exterior composite column.

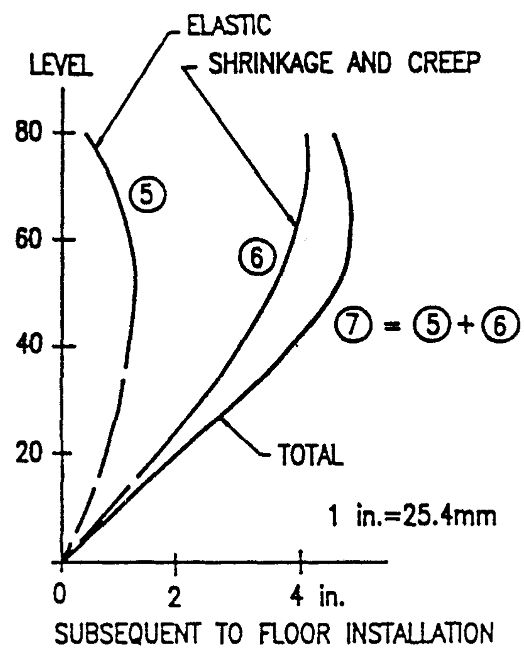


Fig. 7. 80-story composite building - exterior composite column.

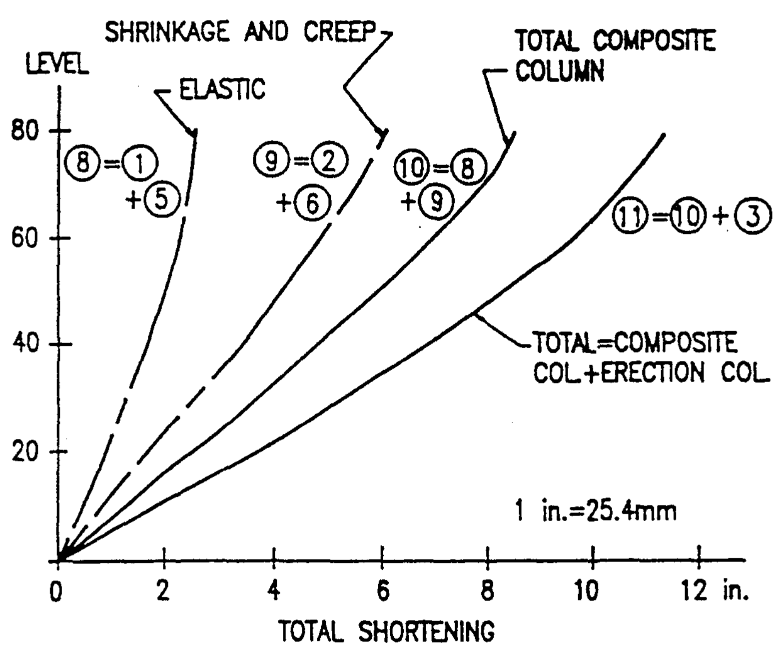


Fig. 8. 80-story composite building - exterior composite column.

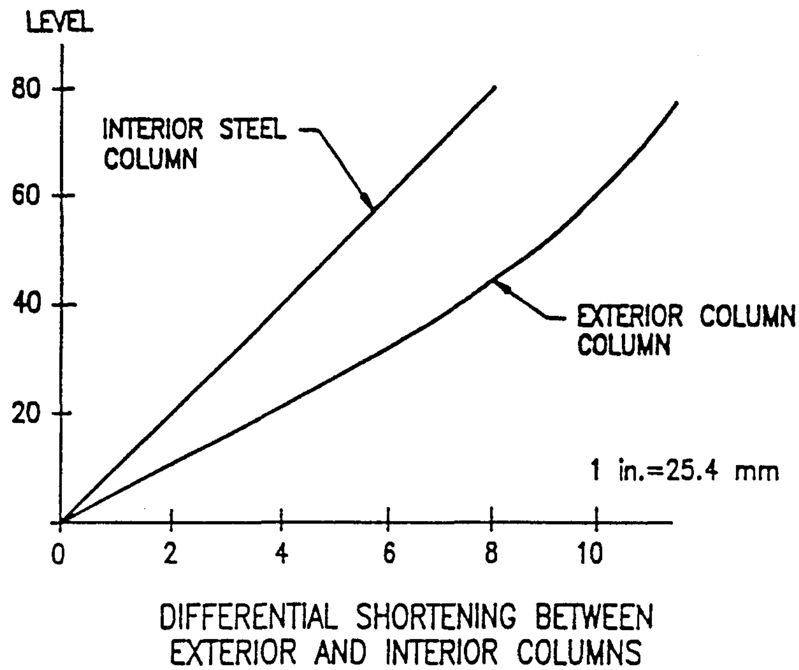


Fig. 9. 80-story composite building.

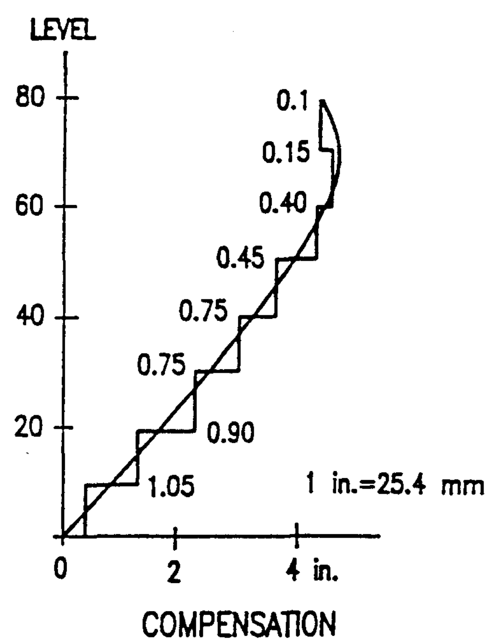
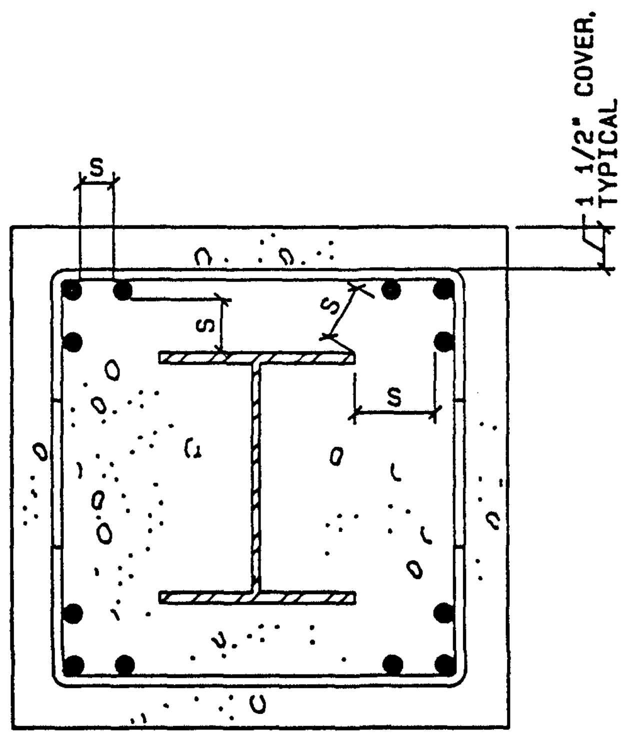


Fig. 10. 80-story composite building.



S=CLEAR DISTANCE BETWEEN BARS OR CLEAR DISTANCE BETWEEN ANY BAR AND FACE OF W SHAPE

$S \geq 1\ 1/2 \times d_b$ OR 1 1/2", WHICHEVER IS GREATER

d_b =BAR DIAMETER

Fig. 11. Composite column cover and bar spacing requirements.

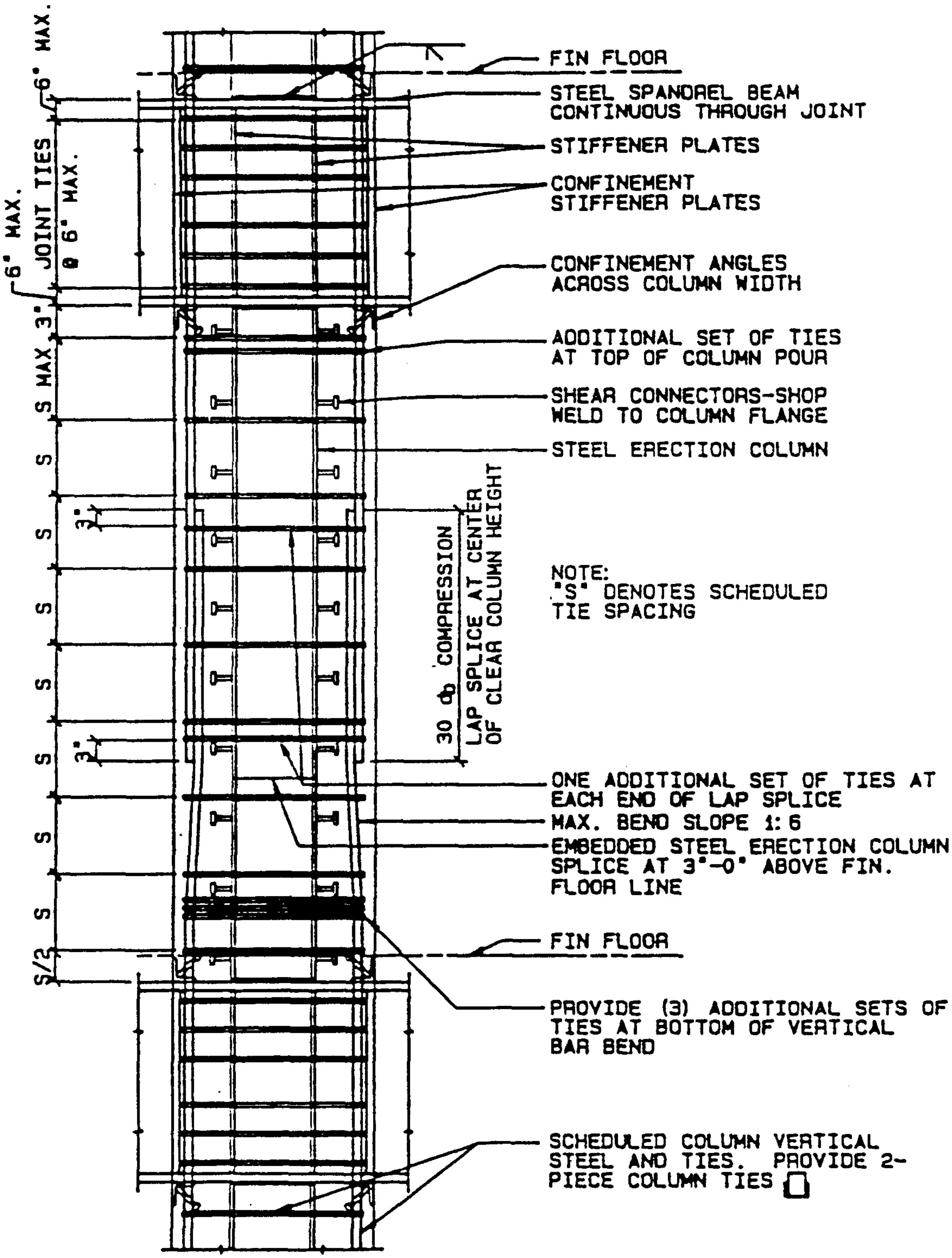


Fig. 12. Composite column elevation.

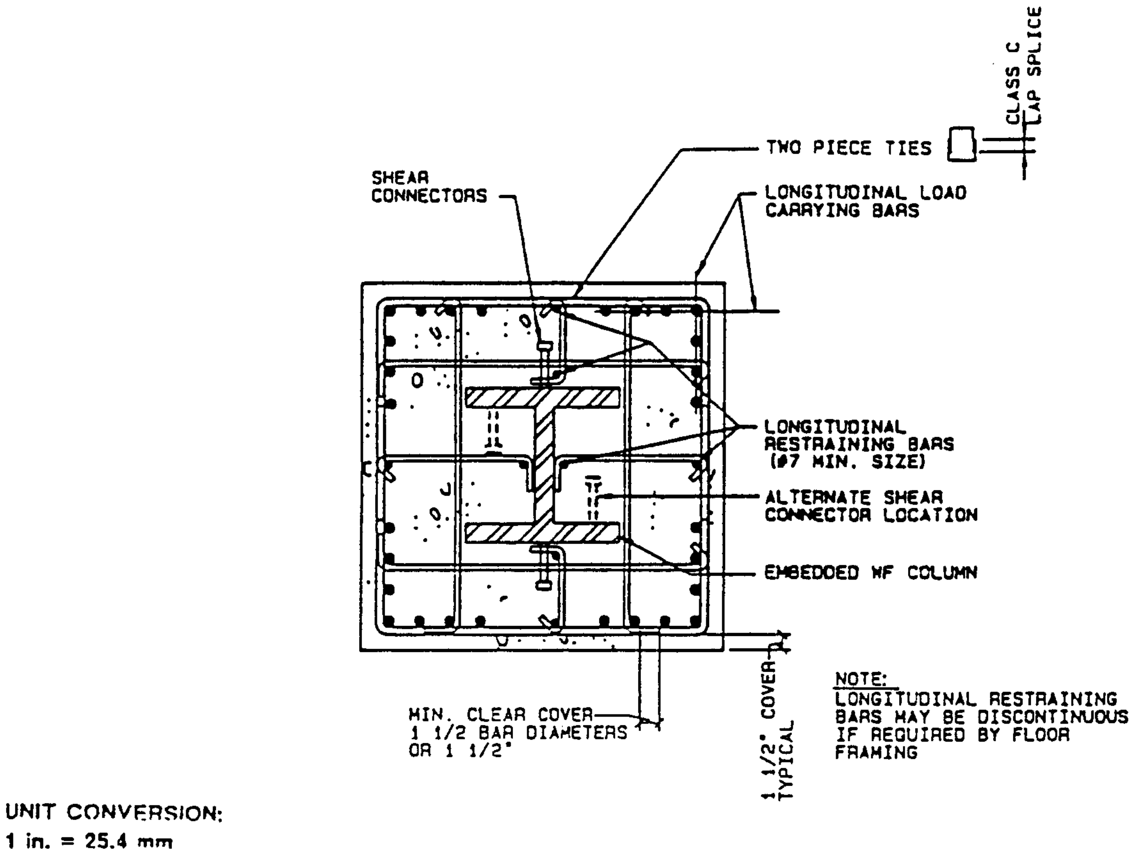


Fig. 13. Composite column cross-section.

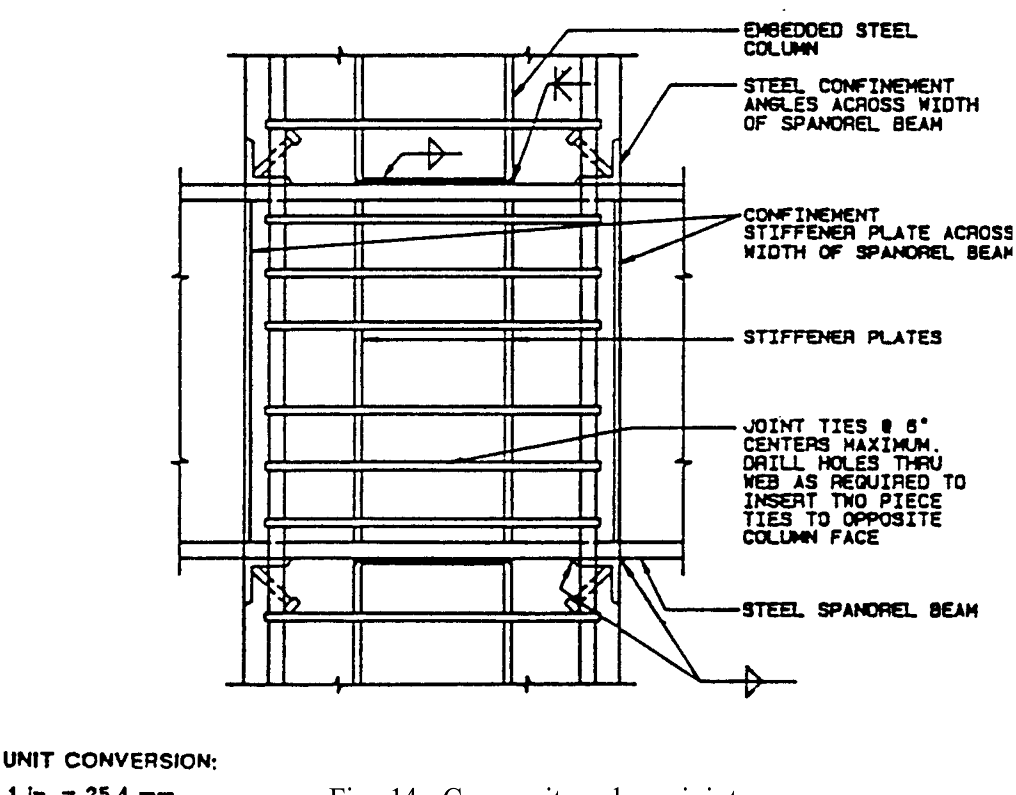


Fig. 14. Composite column joint.

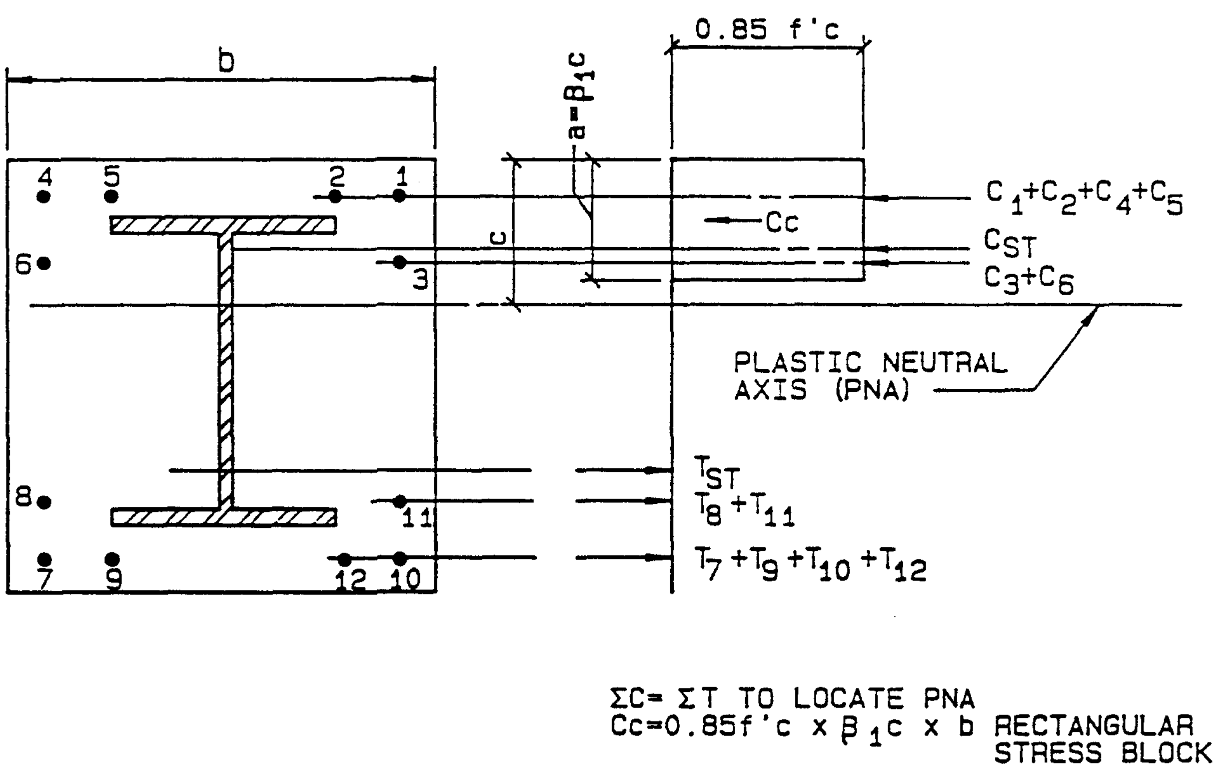


Fig. 15. Plastic stress distribution in composite columns.

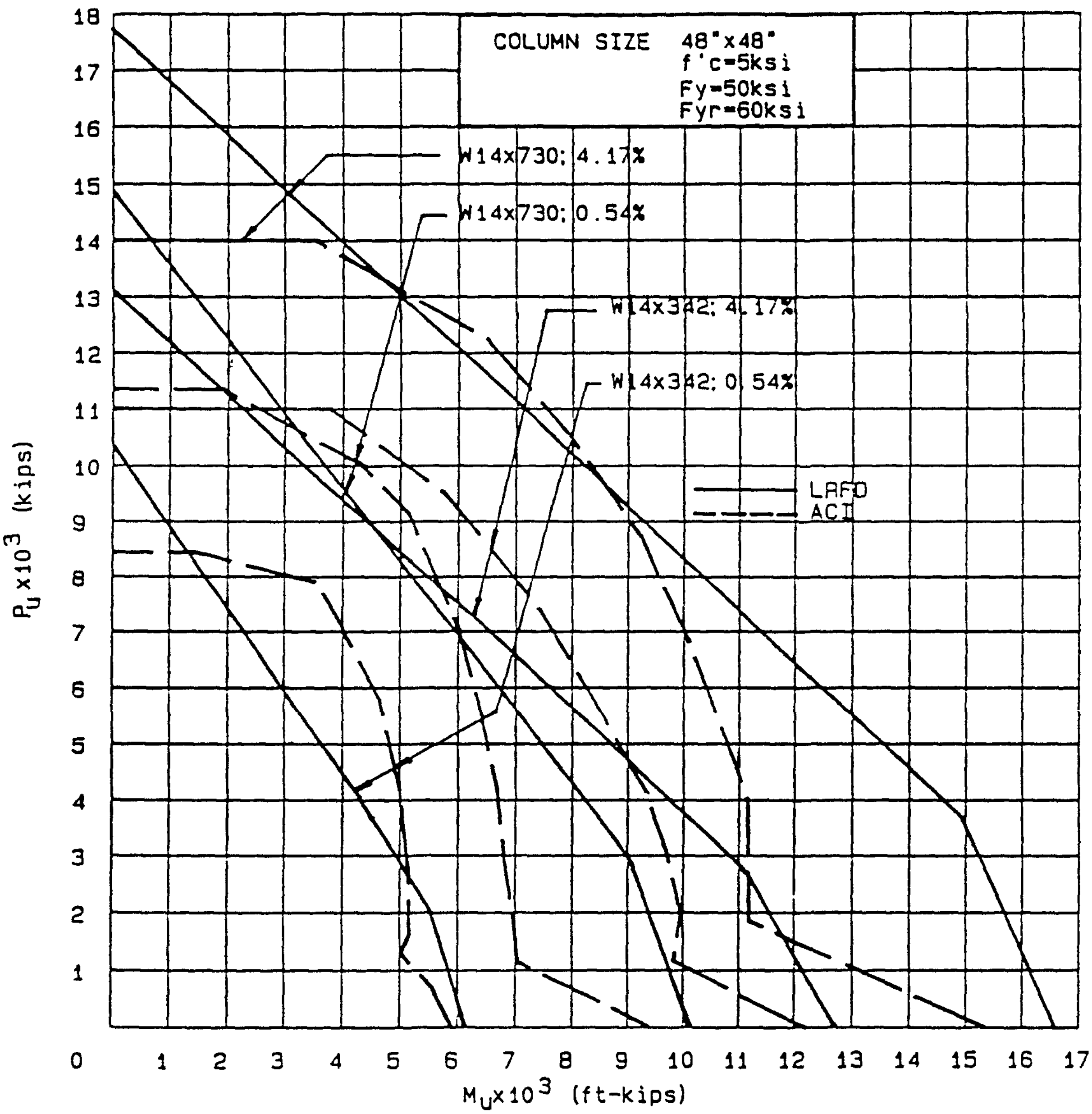


Fig. 16. - (a) Interaction curve comparisons, ACI versus LRFD

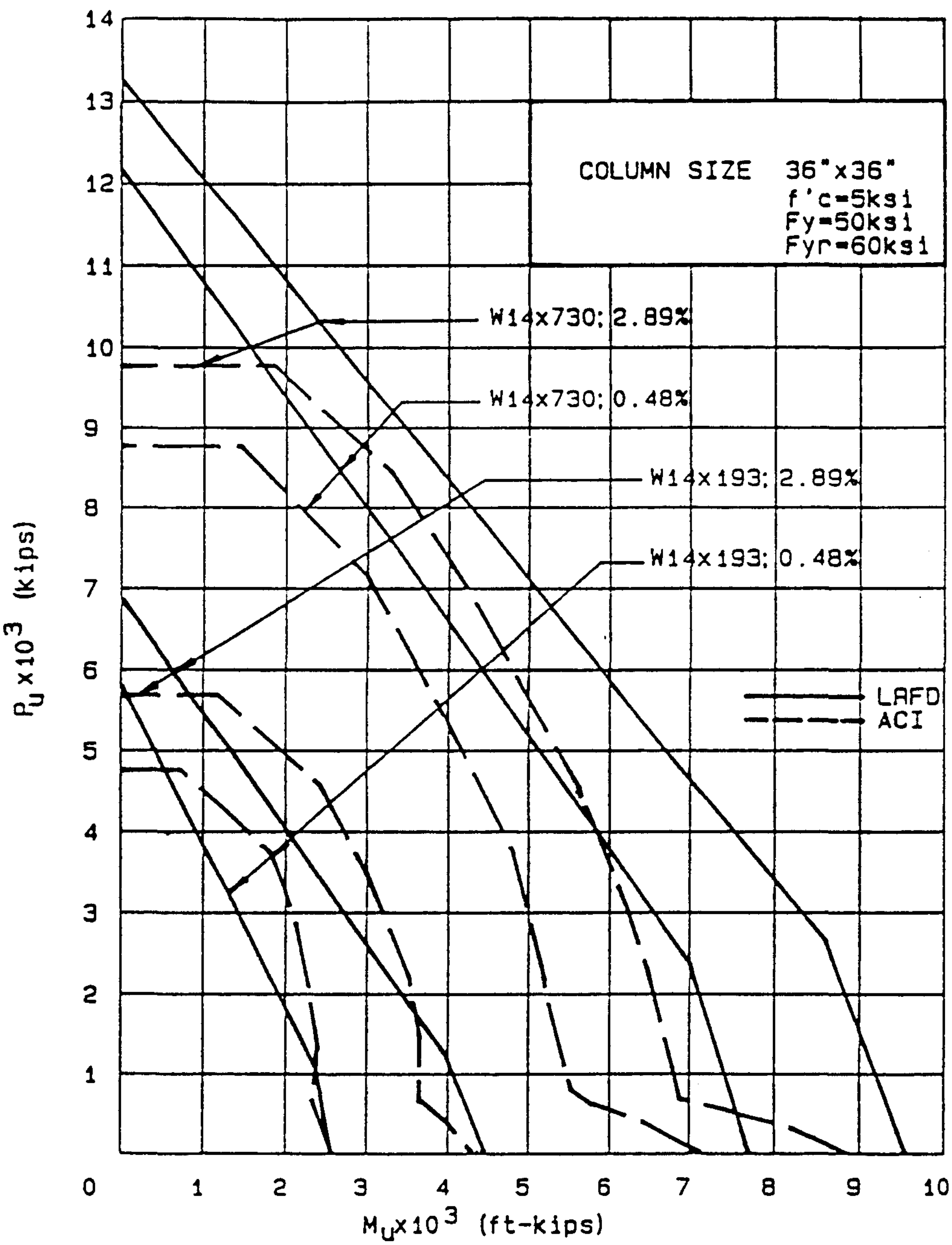


Fig. 16. - (b) Interaction curve comparisons, ACI versus LRFD

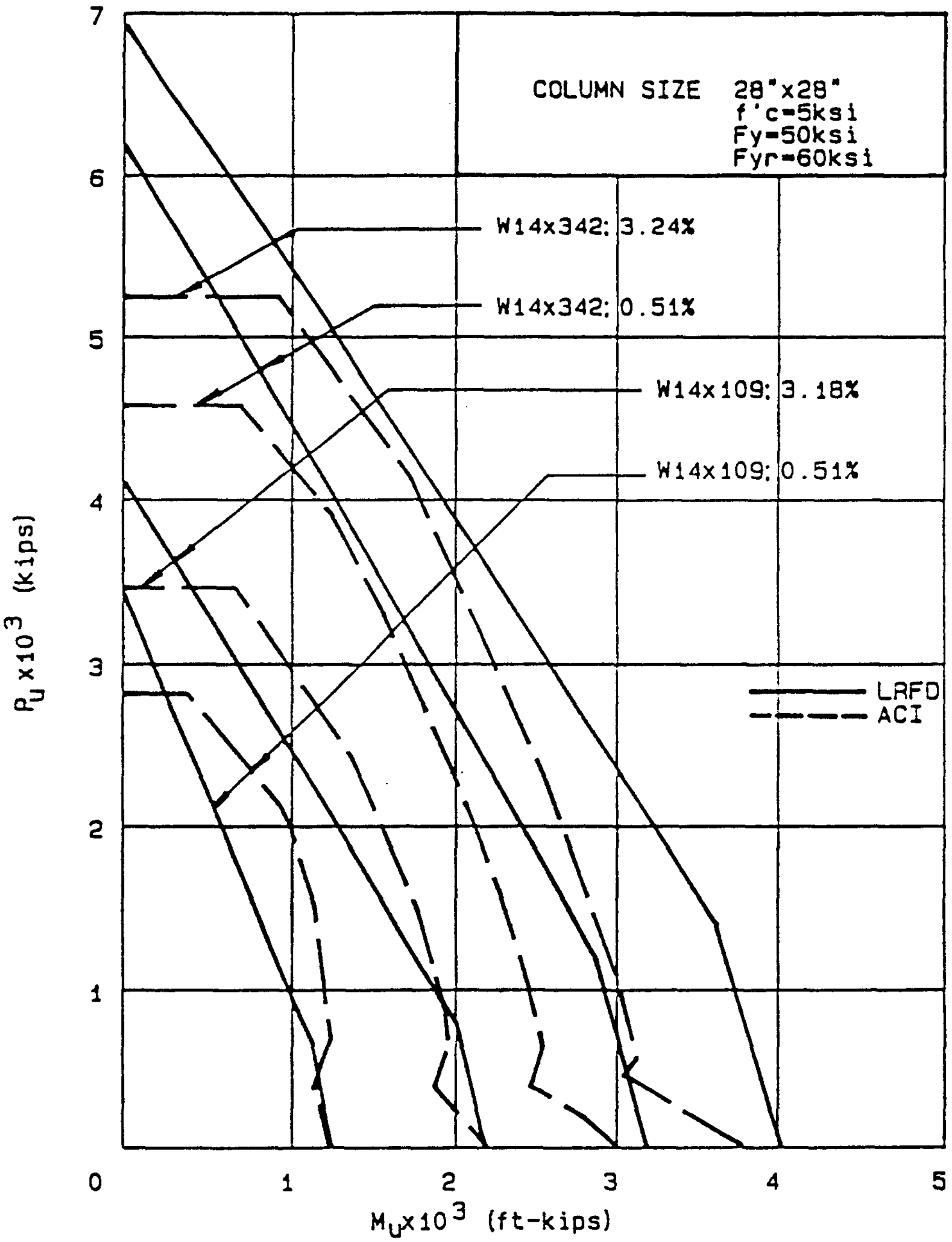


Fig. 16. - (c) Interaction curve comparisons, ACI versus LRFD

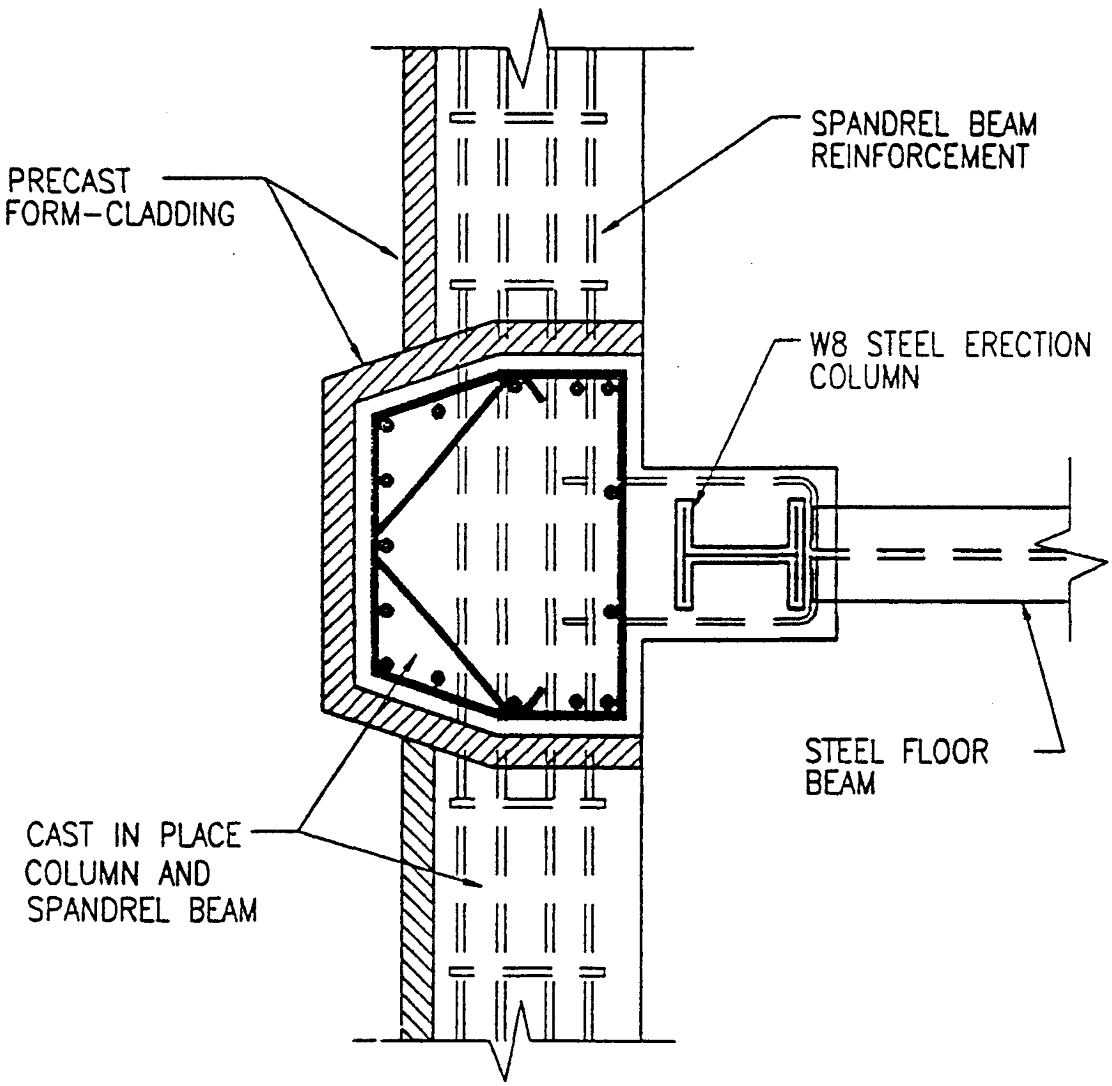

PRECAST FORM-CLADDING
SPANDREL BEAM REINFORCEMENT
W8 STEEL ERECTION COLUMN
STEEL FLOOR BEAM
CAST IN PLACE COLUMN AND SPANDREL BEAM

Fig. 17.

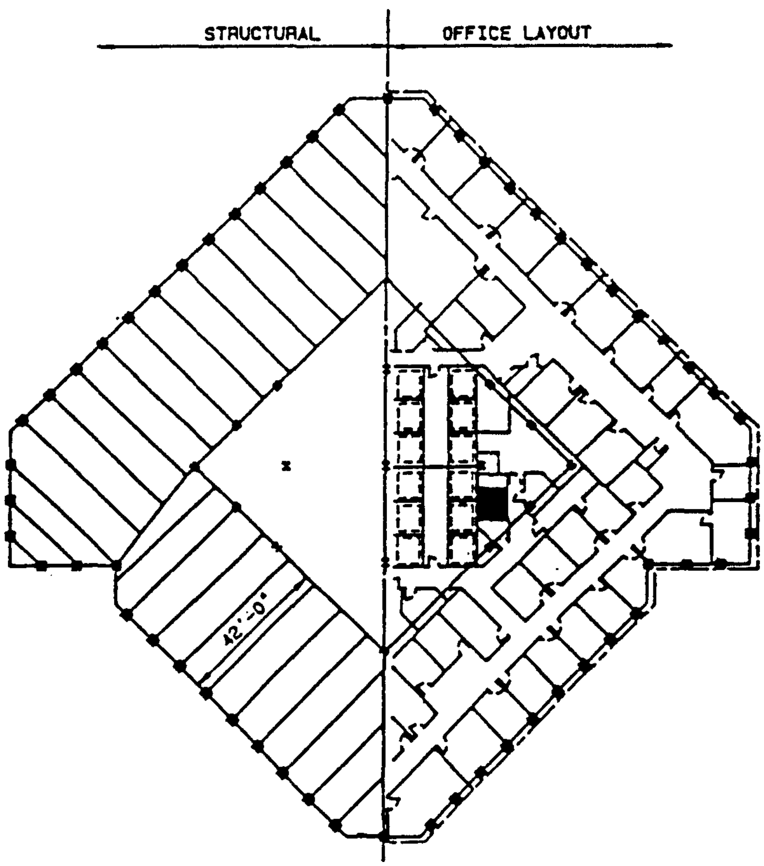


Fig. 18. Composite tube frame - Gulf Tower.

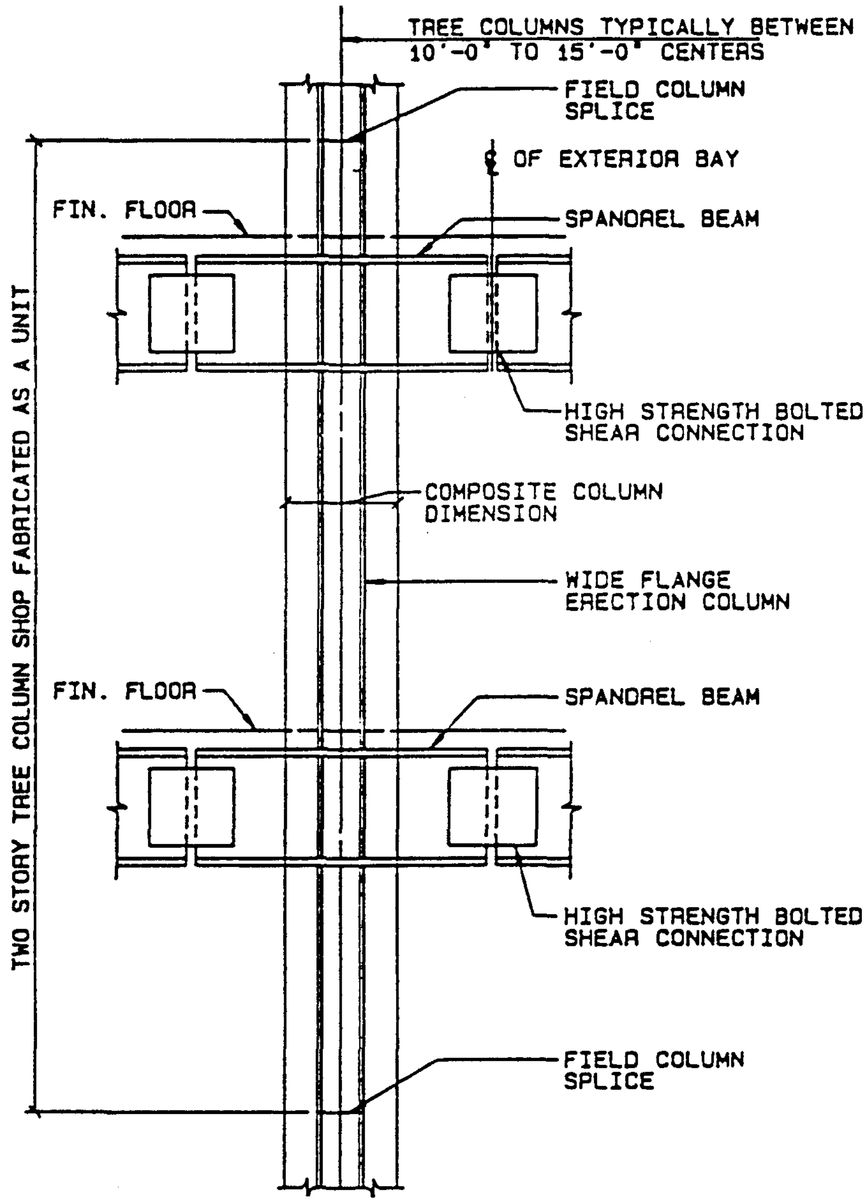
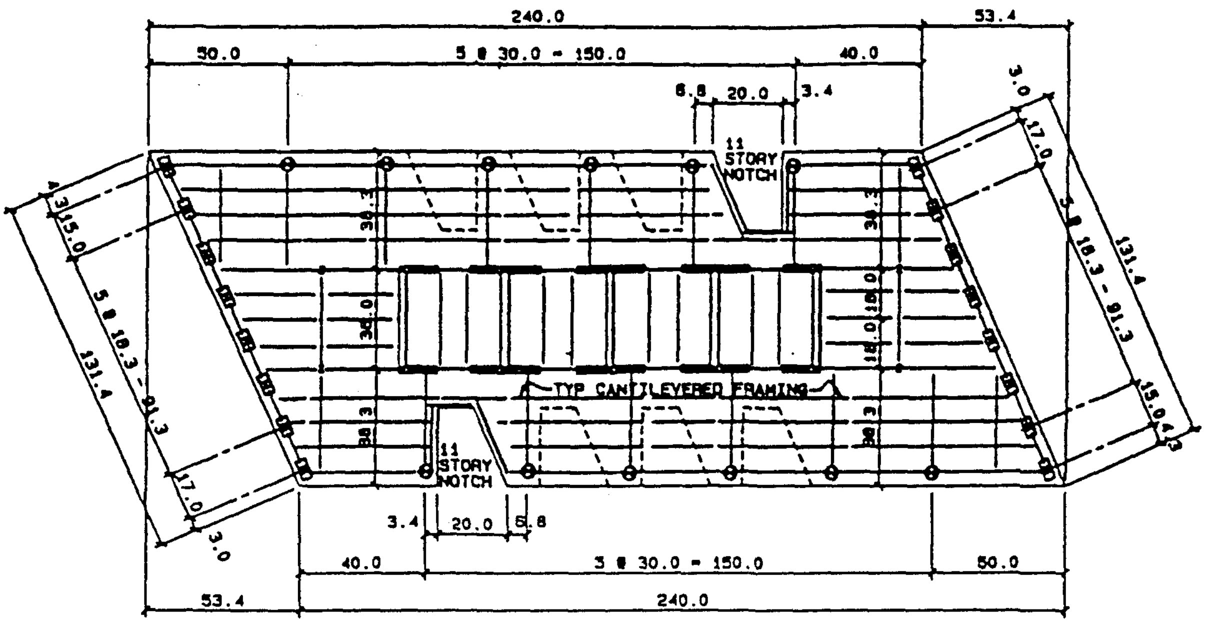


Fig. 19. Tree column in a composite frame.



UNIT CONVERSION:
1 ft. = 0.3048 M
1 in. = 25.4 mm

Fig. 20. Floor plan First City Tower.

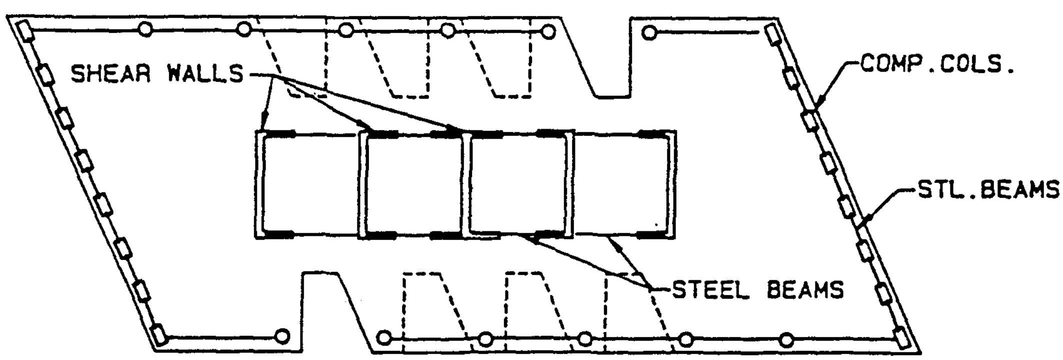
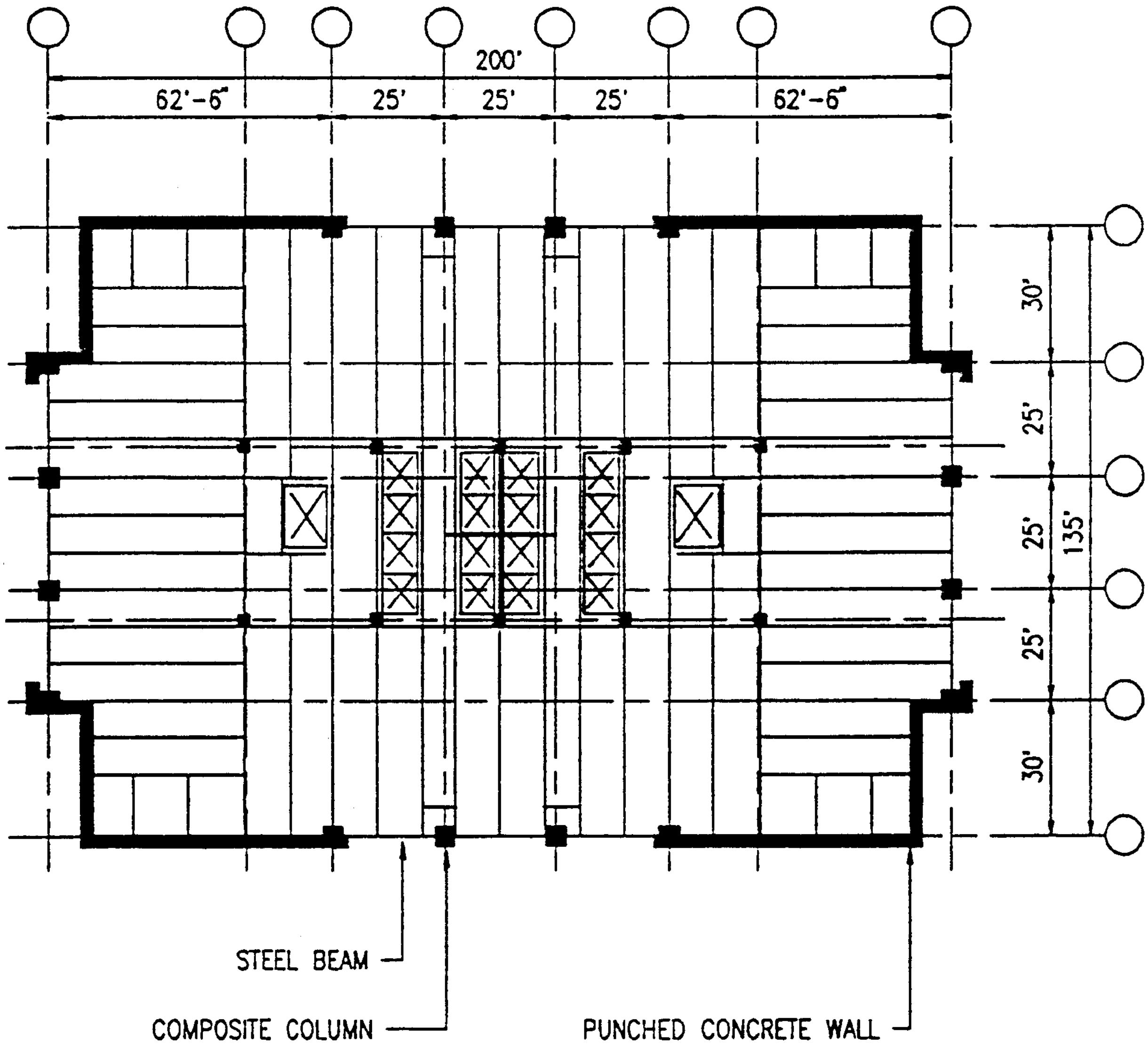


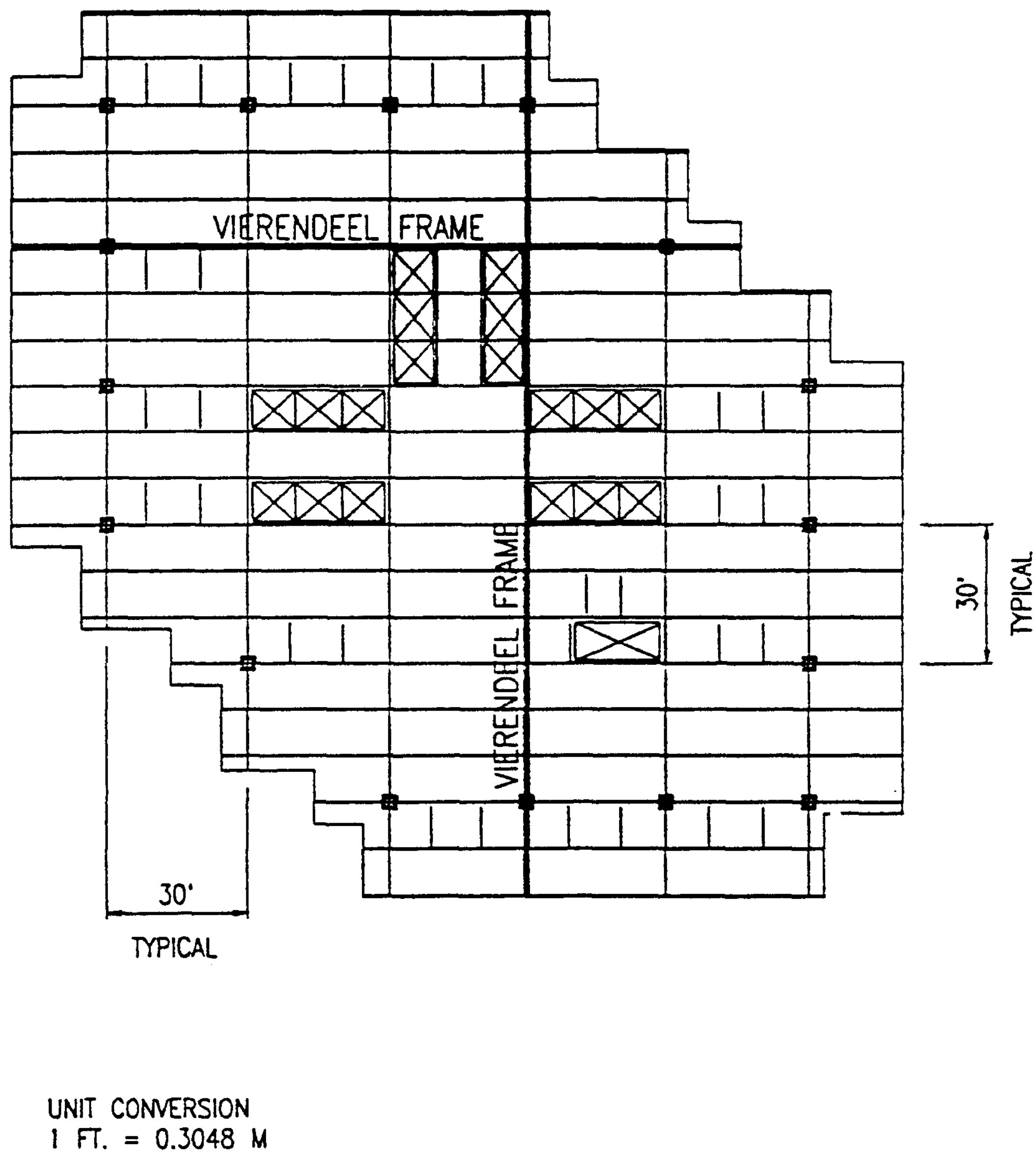
Fig. 21. Composite shear wall and perimeter frame, First City Tower.



UNIT CONVERSION
1 FT. = 0.3048 M

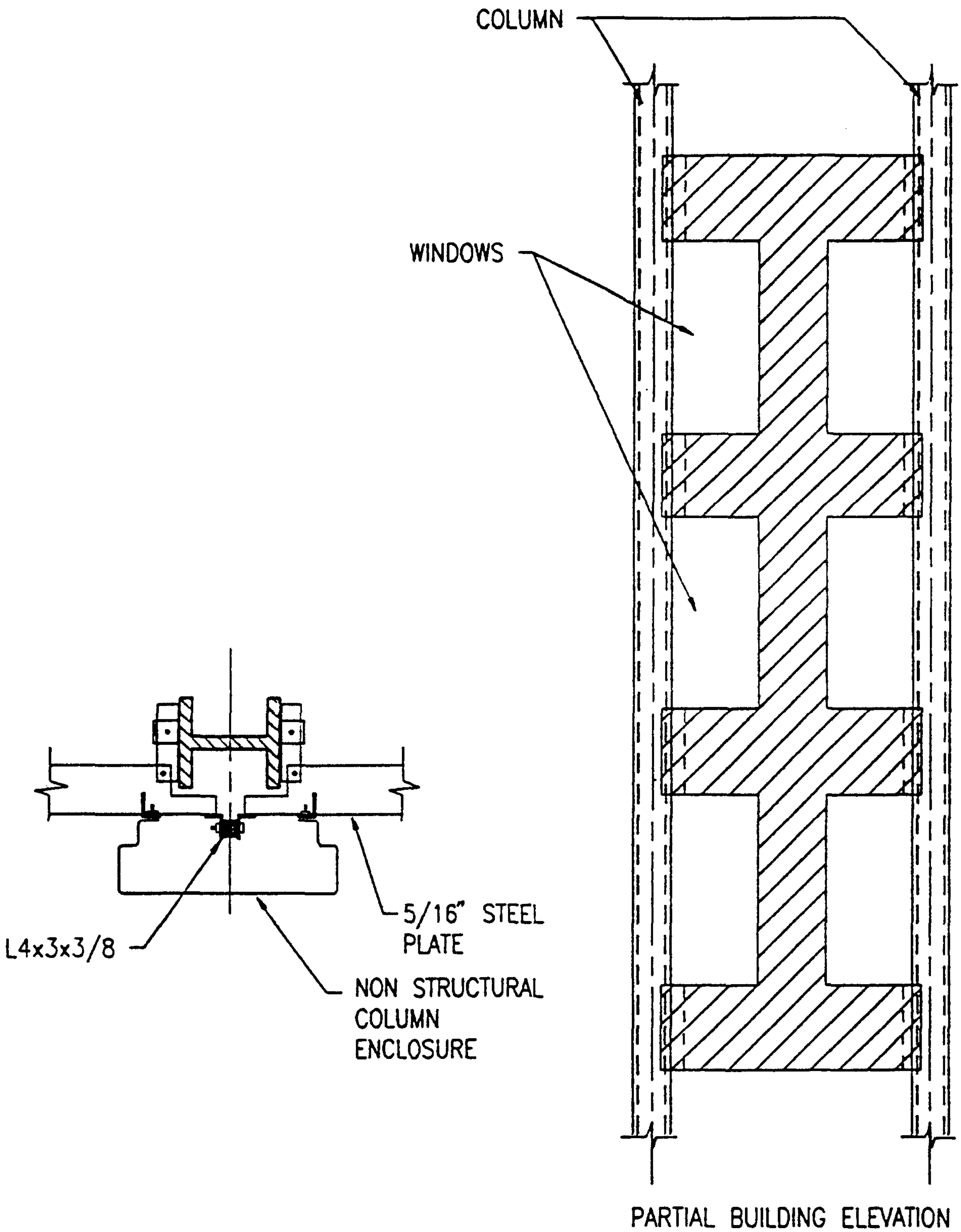
MOMENTUM PLACE

Fig. 22.



INTERFIRST PLAZA

Fig. 23.



DRAVO TOWER

Fig. 24

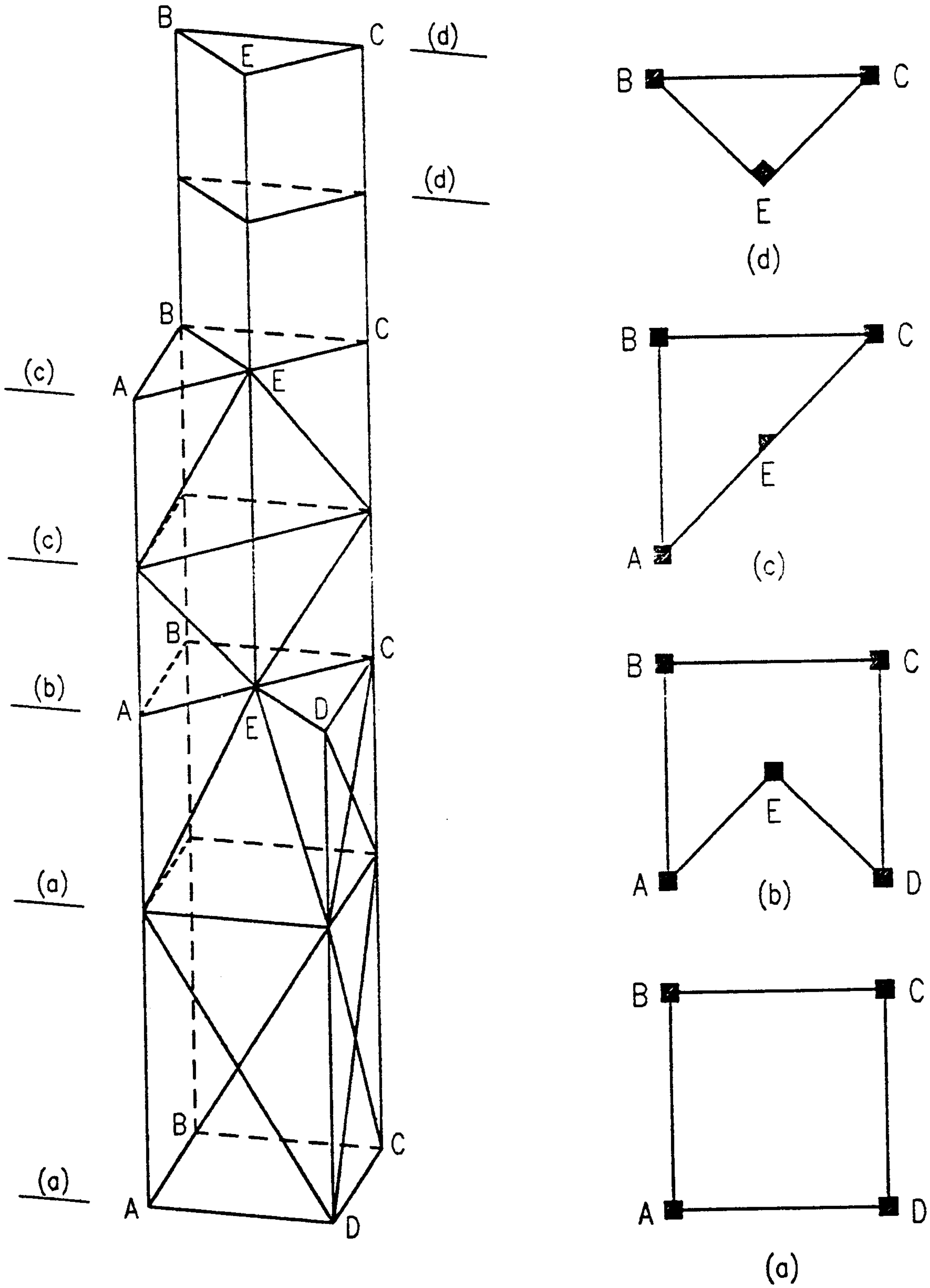
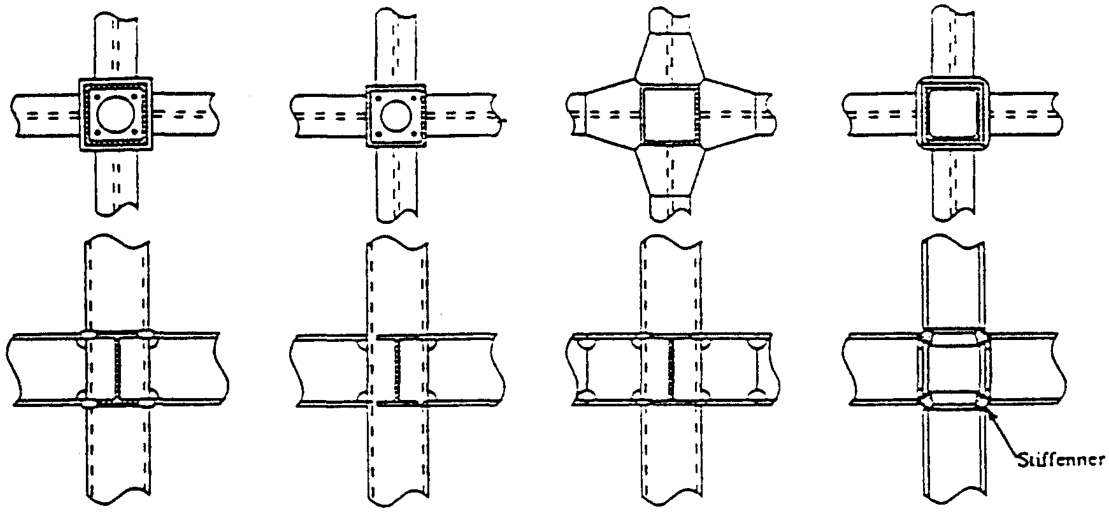


Fig. 25. Bank of China Tower



(a) Through-type diaphragm

(b) Inner diaphragm

(c) Outer diaphragm

(d) Cast-iron stiffenner

Fig. 26. Types of concrete filled tube column connections.

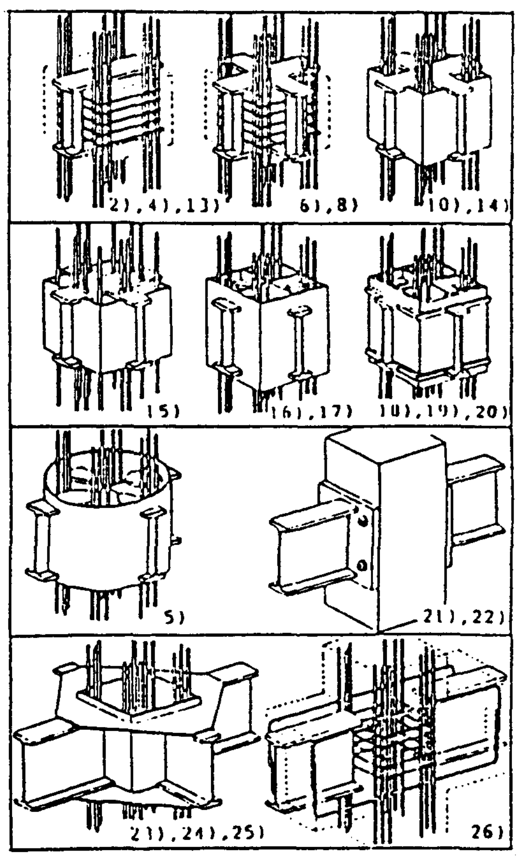


Fig. 27. Composite column connections.

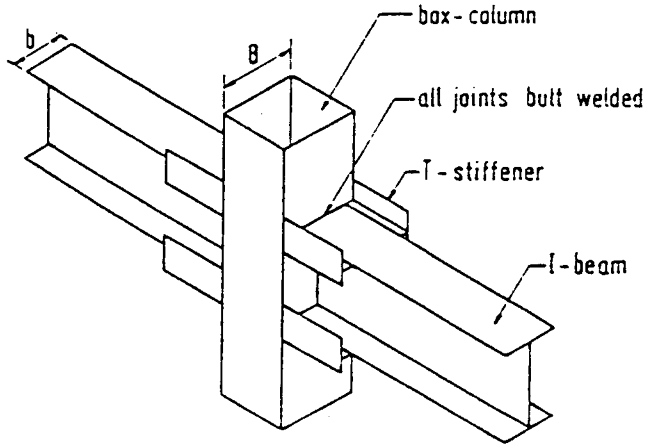


Fig. 28. Box column to I Beam connection.