18

Steel Design Guide

Steel-Framed Open-Deck Parking Structures

18

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Preface

This design guide is specifically focused on structural engineering issues in the design of open-deck parking structures and does not deal in depth with parking usage or geometric topics. General parking topics and their implementation in steel-framed parking structures are covered in a separate publication, *Innovative Solutions in Steel: Open-Deck Parking Structures* (formerly titled *A Design Aid for Open-Deck Steel-Framed Parking Structures*), also published by the American Institute of Steel Construction.

This design guide approaches the development of steel-framed parking structures in the same sequence as a designer would approach the design development. For this reason, the discussion of the steel framing system is deferred until after the section dealing with deck selection. The issues discussed in this design guide are:

- Deck Systems
- Framing Systems
- Mixed Use Structures
- Fire Protection Requirements
- Barriers and Facades
- Stairs and Elevators
- Corrosion Protection
- Structural Maintenance

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Table of Contents

Chapter 1
Introduction

1.1 Overview of Open-Deck Parking Structures

Steel-framed parking structures are increasing in popularity. The recent trend toward steel has prompted industry analyst Dale Denda of the Parking Market Research Company to comment that "exposed steel-frame construction is back as a recognized option for multi-story parking structures." (*Parking Today*, June 2001)

Recent advances in coating technologies and design innovations need to be evaluated and considered for the parking structure. In addition, the structural engineer needs to be able to intelligently evaluate the merits of various framing systems in order to provide professional guidance to garage owners and other members of the project team.

Today, owners and architects are choosing steel framing systems for their lower construction costs, reduced life-cycle costs, rapid construction, long term durability and a clean, open feel conducive to personal security. It falls to the structural engineer to optimize these benefits in the final design by taking advantage of high-performance coatings, innovative structural techniques, reduced structure weight (often at least 20 percent) and enhanced seismic performance.

Today's parking structure framing systems primarily fall into three categories:

- Cast-in-place concrete framing supporting a post-tensioned concrete deck
- Precast/Prestressed concrete framing supporting precast double tees
- Fabricated structural steel framing supporting a post-tensioned cast-in-place, conventionally reinforced concrete deck on stay-in-place metal form or precast deck

Other deck systems have been utilized in various areas of the country including concrete filigree panels (a precast panel form system) and short-span reinforced concrete on removable forms. Structural steel framing has been used to support all of these types of concrete deck systems. This allows the structural designer to choose the optimal deck system for a given project and still enjoy the benefits of a steel framing system.

1.2 Major Components of Interest to a Structural Engineer

In order to effectively design an open-deck steel-framed parking structure the structural engineer will need to evaluate a number of issues. These include:

- Relevant provisions of the governing building code for the location of the parking structure
- The geometry of the parking stalls as a function of optimum bay sizing
- The possible configuration of ramp systems to allow for smooth traffic flow within the parking structure

These three design components are introduced and discussed as part of the general parameters affecting parking design in a separate publication, *Innovative Solutions in Steel: Open-Deck Parking Structures* (formerly titled *A Design Aid for Open-Deck Steel-Framed Parking Structures*), also published by the American Institute of Steel Construction. They are summarized in this introductory section as they impact structural design.

Nine components of the structural design process have been identified and a separate section has been allocated to each. These are:

- Deck Systems
- Framing Systems
- Mixed-Use Structures
- Fire Protection Requirements
- Barriers and Facades
- Stairs and Elevators
- Corrosion Protection
- Structural Maintenance

Four appendices are included that provide design examples, additional resources relating to high-performance coating systems, discussion of the benefits of steel-framed parking structures and additional resources for the designer of a parking structure.

1.3 Code Considerations

1.3.1 Code Applicability

Over the past several decades designers have been faced with a variety of differing building codes based on the location of the constructed project. Variations existed between model building codes and local jurisdictions within areas of adoption of model building codes. The International Code

Table 1-1 Relevant Code Sections for Open-Deck Parking Structures

Topic	IBC	NFPA 88A
Structure Classification	406.3.3.1	3.3.2.2
Clear Height	406.2.3	
Guards	406.2.4	
Vehicle Barriers	406.2.5	
Vehicle Ramps	406.2.6	
Floor Surface	406.2.7	4.3
	406.3.4	
Mixed Use Separation	406.2.7	4.1.2
	406.3.4	4.1.4
		30.8.1.2 (NFPA 5000)
Area and Height	406.3.5	4.7.3
	406.3.6	
Sprinkler Systems	406.3.10	
Prohibitions	406.3.13	
Design Loads	ASCE 7-98 Table 4-1	
Load Reductions	1607.9.1	

Council released the International Building Code in 2000, consolidating three previously separate and regional model building codes: the BOCA National Building Code, the ICBO Uniform Building Code, and the SBCCI Southern Building Code. In 2002, the National Fire Protection Association released NFPA 5000 as an alternative model building code. NFPA 5000 (Section 6.4.2.55) specifies that all types of parking structures conform to NFPA 88A. Designers should verify which model building code and what local amendments are applicable for a planned parking structure.

1.3.2 Relevant Code Sections for Open-Deck Parking Structures

For a listing of the relevant code sections for open-deck parking structures, see Table 1-1.

1.3.3 Code Definitions

Care must be taken in understanding the provision of the codes based on the definition of certain terms. These include:

Height. The IBC defines the height of a parking structure as the vertical distance from the grade plane to the highest roof surface.

Openness. The IBC defines required openness for a parking structure as having uniformly distributed openings on two or more sides of the structure comprising at least 20 percent of the total perimeter wall area of each tier and the aggregate length of the openings should constitute a minimum of 40 percent of the perimeter of the tier. NFPA defines openness as having distributed openings to the atmosphere of not less than 1.4 ft^2 for each linear foot of its exterior perimeter. The openings should be uniformly distributed over 40 percent of the perimeter or uniformly over two opposing sides.

1.3.4 Fire Protection and Height

Currently, model building codes do not require fire protection for structural steel members in an open-deck parking structure less than 75 ft in height as long as any point on any parking tier is within 200 ft of an open side. It should be noted that the height of a parking structure is measured to the top of the deck for the top parking tier, not to the top of any facades or parapet walls (this is based on the treatment of the top tier as the "roof" of the parking structure with parking allowed on the roof).

It is possible for a steel-framed parking structure to exceed the 75-ft limitation based on the square footage of each tier and the number of open sides, although parking structures seldom attain this height for operational reasons. Table 1-2 presents the parameters used in determining maximum height and tier area under both the NFPA Building Code and International Building Code. The prospective owner of a parking structure should consult with the local building code official to determine any local modifications of the relevant code provisions.

Table 1-2 NFPA Building Code and International Building Code Guidelines for Height and Tier Area Perimaters

	NFPA 88A Type II (000)		IBC Type IIB	
Fire Resistive Requirement	None		None	
Definition of Open Side	1.4 sq ft of each linear foot distributed along 40% of perimeter		50% of interior wall area of exterior wall	
	sq ft/tier	# of tiers	sq ft/tier	# of tiers
2 sides open	unlimited[1]	height<=75 ft	50,000	8
3 sides open	unlimited[1]	Height<=75 ft	62,500	9
4 sides open	unlimited[1]	Height<=75ft	75,000	9
Exception[1]			unlimited	height<=75 ft

[1]the distance from any point on the deck may not be greater than 200 feet from an open side

Table 1-3 Minimum Number of Accessible Spaces

Number of Parking Spaces	Minimum Number of Accessible Spaces
1 to 25	1
26 to 50	2
51 to 75	3
76 to 100	4
101 to 150	5
151 to 200	6
201 to 300	7
301 to 400	8
401 to 500	9
501 to 1,000	2% of total
1,001 and over	20 plus 1 for each 100 over 1,000

When evaluating tier area and structure height, the impact of any future vertical expansion should be taken into account.

When parking is being provided on the lower floors of a mixed-use structure, the lower parking floors must be fire separated from the upper floors and fire rated.

1.3.5 ADA Guidelines

The Americans with Disabilities Act establishes design guidelines for addressing the needs of persons with disabilities to access all newly constructed structures. Current ADA guidelines impacting parking include:

- The provision, size and location of a required number of physically disabled accessible spaces
- The provision, size and location of physically disabled van access
- Ramp slopes
- Signage
- Trip hazards
- Exit paths

Table 1-3 indicates the required minimum number of accessible spaces in any parking facility. These spaces must be at least 8 ft wide with a 5-ft-wide accessible aisle adjacent to the space. Two accessible spaces may share the same accessible aisle if the spaces utilize 90° parking. Angled parking spaces must each have their own accessible aisle. Ceiling clearances are not impacted by accessible spaces and should conform to a 7 ft, 2 in. minimum or any applicable local codes. Accessible spaces are required to be the closest spaces to all accessible building entrances.

One out of every eight accessible spaces must be physically disabled van accessible. Access to van-accessible spaces must meet the 8 ft, 2 in. requirement for ceiling clearance. The van-accessible space is still required to be only 8 ft wide but must be adjacent to an 8-ft-wide accessible aisle. Van-accessible spaces may be grouped on one level of the parking structure, typically the ground level.

Any ramp upon which parking or pedestrian traffic is allowed is recommended not to exceed a 5 percent slope with a 6 percent maximum slope allowed. All accessible routes must be clearly marked and, if the slope exceeds 5 percent, be slip resistant. All pedestrian paths must be clearly marked with signage with raised or Braille letters and standard symbols. Local ordinances generally exceed the ADA requirements for size of lettering on directional signs for vehicular traffic.

All trip hazards, such as car bumpers and raised curbs must be eliminated from pedestrian pathways, with maximum curb slopes being 8 percent. All multi-story parking structures require either at least one accessible elevator, a pedestrian ramp to grade level or a grade-level accessible structure.

The reader is encouraged to become familiar with the full text of the ADA guidelines.

Chapter 2
Deck Systems for Parking Structures

No treatment of the introduction to structural design and construction of steel-framed open-deck parking structures is complete without a discussion of concrete deck systems. In fact the structural designer, in concert with the project owner and architect, should make the selection of the type of deck system before consideration of the framing system.

The concrete deck or floor system is one of the two structural sub-systems in a parking garage, and the one which governs the performance, life expectancy and life-cycle cost of the facility. The other sub-system is the structural frame that supports that concrete deck, the steel beams, girders and columns. As previously noted, there are several basic concrete deck systems that have been used with steel framing in parking garages:

- Cast-in-place, conventionally reinforced concrete on stay-in-place galvanized metal deck forms (in areas where road salts are not prevalent)
- Cast-in-place, post-tensioned concrete
- Precast, prestressed long-span double tees either pre-topped or site-topped
- Precast concrete forms with site-cast composite topping

Cracks, resistance to volumetric changes, poorly designed or installed deck joints, freeze-thaw cycles and chloride contamination in concrete decks have been the major causes of deterioration of open-deck parking structures. Chlorides become established within the deck when de-icing salts combine with water and penetrate into the cured concrete or through cracks and joints. This is usually followed by corrosion and volumetric expansion of the concrete reinforcing steel and destruction of the concrete. Also, concrete decks in any climate can become distressed when the concrete ingredients or additives themselves contain excess chlorides or other contaminants. Chlorides that leak through cracks or joints in the deck to structural steel framing below can attack the steel and cause breakdown of the coating system and subsequent corrosion.

It is estimated that 10 to 12 million tons of sodium and calcium chloride are used annually during wintertime de-icing operations in the United States. Approximately two-thirds of the land area in the U.S. is subject to freezing temperatures during winter on a regular basis. The corrosion of concrete reinforcing steel due to chloride contamination from road salts began to be widely recognized by state Departments of Transportation in the 1970s, as the problem was being encountered in highway bridge decks.

Only about 0.2 percent of acid-soluble chloride content by weight of portland cement is enough to contaminate conventional concrete and initiate corrosion of embedded reinforcing steel. This concentration is equivalent to about 1¼ pounds of chlorides in a cubic yard of concrete. As it corrodes, embedded reinforcing steel can expand several times in volume, generating internal pressures on the order of 50,000 psi. This results in spalling and destruction of the concrete deck. Crack control should be the structural engineer's highest-priority criterion for design. Unless the impact of cracks is controlled through proper design and regular inspection and sealing of cracks that do occur after construction, most of the other corrosion prevention measures available will not be successful over the long term.

2.1 Types of Deck Systems

Deck systems fall into three major categories:

- Conventionally reinforced concrete (site cast)
- Prestressed post-tensioned concrete (site cast)
- Precast concrete (usually plant cast)

A reinforced slab consists of concrete poured around mild reinforcing steel. This is a static type of system that reacts to load through the concrete shedding tensile load to the reinforcing steel through limited bonding between the steel and concrete, but ultimately by the steel taking on the tensile load through cracking of the concrete.

Prestressed post-tensioned concrete is cast around prestressing strands or tendons that compress the concrete to the extent that when an external load is applied, the concrete remains in compression. In a prestressed system the strands are stressed or stretched before the concrete is poured. The prestressed tendons are bare, and are consequently bonded to the concrete. Post-tensioning differs slightly in that the strands are encased in plastic sheathing, have the concrete cast against them and are then stressed or stretched. Thus the definition of prestressed or post-tensioned is delineated by when the strands are stressed relative to the placement of the concrete.

The biggest single difference between the two types of decks is that the prestressed/post-tensioned deck is typically under compression across the entire cross section and is not as susceptible to cracking when properly designed and detailed. Conversely, the conventionally reinforced concrete deck is prone to cracking on the tension side. The degree of cracking of a reinforced concrete slab is affected by many variables such as the amount of reinforcing steel used, the reinforcing location, the concrete quality, the concrete curing process, and joint-spacing.

Section 2.1 contains a discussion of each type of deck system, Section 2.2 presents climactic considerations affecting each deck system and the tables in section 2.5 summarize deck characteristics.

2.1.1 Cast-in-place reinforced concrete

Cast-in-place reinforced concrete slabs have performed admirably in floor systems in enclosed conventional buildings. In open-deck parking structures, however, concrete decks suffer from freeze-thaw cycles in cold climates, application of de-icing road salts, poor design, construction or inspection practices, and unsuitable aggregates.

Certain basic precautions are required for a parking deck to survive for the long term. These include the use of:

- High-grade concrete and aggregate
- Proper curing procedures (7 days wet cure for optimum results)
- Concrete with a minimum compressive strength of 4,500 psi
- Adequate drainage of the deck surface
- A low water/cement ratio concrete mix (0.40 or less)
- Adequate clear cover (1.5 in.) for the top reinforcing steel
- Low permeability for the cured concrete
- Proper placement of reinforcement

The minimum thickness for a cast-in-place, conventionally reinforced slab in an open-deck parking structure is dependent on bay spacing.

Reinforcing steel in a cast-in-place concrete deck must be protected. There are several options for protecting the reinforcing steel.

- Epoxy coating
- Galvanizing
- Use of stainless steel reinforcing bars
- Use of corrosion-inhibiting admixtures
- Use of Cathodic protection (may be cost prohibitive)

Recent research sponsored by FHWA indicates that a 75 to 100 year life can be expected for a concrete bridge deck by using stainless steel reinforcing, with or without cracks in the deck. It is difficult, however, to justify the increase in expense by using stainless steel for a parking structure.

2.1.1.1 Clear Cover and Permeability

Two prominent causes of distress in cast-in-place concrete decks are excessive permeability and inadequate clear cover over reinforcing steel.

Concrete is much like a "hard sponge" that will absorb moisture throughout its life. Fortunately, there are several ways to control penetration of chlorides into the deck. The permeability of the concrete itself can be reduced by:

- A water-reducing admixture (also known as a superplastizer)
- A low water-cement ratio (0.30 to 0.45)
- A microsilica fume additive
- A calcium nitrate corrosion inhibitor
- Flyash or other pozzalan
- Proper curing procedure

Recent studies have indicated that a low water-cement ratio may be the dominant factor in achieving a concrete with low permeability. A silica fume particle is only one one-hundredth the size of a cement particle. It is easy to see how this additive can fill the voids in a concrete mix—voids that would otherwise conduct moisture. Silica fume, like cement, also hydrates as it cures, so the strength of the concrete increases as well.

The specifier of such high-performance concrete additives to the concrete should be aware that their use may require changes in the way the concrete is placed, finished or cured. For example, shrinkage of superplastized concrete has been observed to be higher in some instances than that of conventional concrete, so the placement of control joints assumes added importance.

Other families of products are intended to prevent chlorides from penetrating into the deck by application after the slab is cast and cured. Examples include: elastomeric waterproofing membranes, penetrating sealers, surface sealers, and coatings or overlays. Sealers, which must be periodically re-applied, seem to be more effective when they can penetrate into the concrete. Good penetration ($^1/_8$ in. to $^1/_4$ in.) along with an adequate coverage rate affords better resistance to permeability and counters the loss of sealer at the surface due to normal wear from traffic on the deck.

In recent years, there has been significant testing and evaluation of substances that seal concrete decks. Materials examined include latex products, epoxies, urethanes, linseed oils, silanes and siloxanes. The success of any sealant depends upon factors such as:

- Chemical formulation
- Concrete quality
- Surface preparation
- Conditions at the time of application
- Rate of coverage

Sealers are considered highly sensitive to these variables, which may help explain inconsistencies among test results and ratings that have been published by both producers and independent agencies. Perhaps the best advice for an owner or specifier is to evaluate a product both by independent agency data and local field experience, when available.

A good waterproofing membrane system, unlike a sealer, will bridge small cracks (perhaps up to 1/16 in. wide). A membrane system, which is usually applied in three or four layers (binder, membrane, wearing surface), may be as much as 4 or 5 times the initial cost of a penetrating sealer. A life-cycle cost analysis is thus in order when selecting a deck surface treatment, and it must include consideration of other corrosion control measures being contemplated for the deck.

The depth of clear cover over reinforcing steel largely determines their rate of corrosion. Even the top 1/2 in. to 1 in. of high-grade concrete can eventually become contaminated by de-icing chlorides. Thus, it has been suggested that the top 1 in. of concrete be considered "sacrificial". By increasing actual concrete cover to 2 in., dramatic reductions in chloride penetration to the level of top reinforcing - and in rate of corrosion - have been observed in simulated long-term tests.

Increasing concrete cover over negative moment reinforcing steel better protects the bars, but will increase the width of any tension cracks that form on the surface. Care should be taken not to significantly exceed 2 in. of cover as cracking will occur in areas of negative reinforcement as the thickness approaches 3 in. A cover of 2 in. of actual cover allows for fabrication and construction tolerances to minimize crack width. The American Concrete Institute (ACI) recommends that top bar spacing in negative moment areas be reduced to as little as 4 in. All reinforcing steel must be strongly supported.

Another technique for protecting reinforcing steel is epoxy coating or galvanizing. Research has shown that an epoxy coating with an optimum thickness from 5 to 10 mils can reduce the rate of steel corrosion up to 41 times. Epoxy coatings are flexible, low in shrinkage and creep, and are virtually impermeable to chloride ions. One concern is damage to the coating during shipment and handling; damaged areas that expose the bar must be repaired. Galvanized bars have received mixed reviews over the years, but studies have also found them to be somewhat effective in resisting chloride corrosion. It is important to note that, when galvanizing is selected as the means of protection for the reinforcing steel, all reinforcing steel in that deck must be galvanized, and the galvanized bars must not be in contact with any ungalvanized steel. Galvanized bars are more resistant to damage from abuse; they tend to repair themselves. Both epoxy coated and galvanized reinforcing steel are used in bridge decks. Bridge owners looking for a 75- to 100-year life-span for critical bridges are likely to opt for stainless steel.

As a chemical additive to concrete, calcium nitrite has been found to be effective in interrupting the electrolytic process that causes corrosion of reinforcing steel in contaminated concrete. Even though chloride concentration at the level of the bars is far above the threshold level, corrosion activity itself is inhibited and greatly diminished.

2.1.1.2 Curing

The necessity of proper curing of the concrete deck cannot be understated. Improper curing techniques and/or the lack of an adequate curing period will often diminish deck performance.

Steam heat-curing of concrete with a low water-cement ratio provides a 28-day compressive strength equal to that of moist curing, and equal or better resistance to water and chloride absorption and intrusion. Steam curing is often utilized for plant-cast deck systems such as precast double tees. Site-cast decks should be water cured for a minimum of 7 days. Curing compounds are not recommended, particularly in warm weather as they do not prevent the escape of moisture and also prevent sealer penetration. The use of any deicer on the deck should be avoided for at least 6 months after concrete placement to minimize concrete scaling.

2.1.1.3 Joints, Cracks and Drainage

Leakage of water chlorides through cracks or joints accelerates corrosion of reinforcement and deterioration of a concrete deck. Leaks also provide the major access for corrosive chlorides to the supporting steel or concrete frame. The primary difference between how these leaks impact a concrete and steel frame is in the amount of time that elapses before the damage becomes obvious. Leakage into a concrete frame will be hidden from view, but will require expensive restoration in the long term. Leakage onto a steel frame will result in short term visible surface corrosion that will require maintenance and touch-up, but more importantly, will bring attention to the deck problem. When this problem appears, it must be resolved in a timely manner to avoid major restoration work on the deck. The tolerated crack width recommended for reinforced concrete structures exposed to deicing chemicals is only 0.007 in. The common causes of cracking in open-deck parking structures are:

- Shrinkage
- Flexure (in areas of negative moment)
- Restraint against temperature-induced volume changes during or subsequent to curing
- Corrosion of reinforcing steel
- Cracking due to long-term effects of creep and differential volume changes between the slab and other structural elements with which the slab interacts, though this is less predictable

The three types of joints in concrete decks are:

- Construction joints, located primarily for the convenience and efficiency of the contractor
- Control joints, located to accommodate shrinkage of the concrete
- Isolation joints, to accommodate expansion and contraction of the finished slab that occur with temperature changes or post-tensioning

Joint seals can be a source of problems if they are improperly installed or poorly maintained. Indeed, an increasing number of state bridge departments are placing their faith in jointless bridge decks and integral or semi-integral abutments to avoid joint problems entirely. However, thermally-induced movements of concrete (and the potential for crack development) are inevitable, and it is better to have one too many isolation joints rather than one too few.

The restraint to volume change developed at rigid elevator and stairwell cores, braced frames, shear walls or connecting structures should not be overlooked. Such restraints, when not properly located or isolated, have been the cause of major cracking in parking decks, especially at re-entrant corners or at other discontinuities. Whenever possible, core areas should be located to minimize discontinuity in the deck system. Codes require that designers strive to locate stairwell cores around the outside of the garage perimeter. If a perimeter stairwell is constructed of rigid materials it should be isolated from the deck slab.

As previously noted, some penetrating sealers are effective in reducing the permeability of concrete decks, but they are not designed to bridge or seal cracks in the slab. Owners should seal all cracks that form during the curing process and apply the penetrating sealer to the "solid" slab just prior to occupancy. The ultimate damage caused by leakage of chlorides through cracks is very dependent on crack width. Therefore, design and construction methods that limit crack width, as well as minimize crack formation, are beneficial.

Cracking and other effects of freezing and thawing cycles have been alleviated by air entrainment of the concrete as required by the ACI code. However, excessive finishing of the air-entrained concrete tends to force water to the surface, thereby increasing permeability. Again, the introduction of additives to the concrete mix may require an alteration in concrete placement procedures.

Regardless of preventive measures taken, cracks and joint leakage in a parking deck must be anticipated. In addition to adequate concrete cover and reduced permeability, there is a third provision that is important to the long-term survival of the concrete deck: drainage. Positive drainage will minimize ponding (i.e., collection of standing water) and limit the quantity of contaminants that will reach reinforcing steel in the deck and the structural steel below. A minimum slope of $^1/_4$ in. per ft is recommended for "flat" surfaces. Water should flow to locations where working drains, with 8-in. or 10-in. diameter downspouts (placed at low points) are able to remove it from the garage.

If cracking occurs, the cracks must be treated as soon as possible. Shrinkage cracks can be epoxied while working stress cracks should be routed and then caulked with a traffic-grade polymer or silicone sealant. (Note: although silicone sealants perform well, they are very soft and present potential trip hazards in pedestrian paths.)

A well-drained deck should be thoroughly rinsed off in the spring, subsequent to the last application of road salts, using a 2-in. hose. Prior to washing, loose, dried salt deposits should be swept up and the deck (above and below) should be inspected for cracks and evidence of joint seal problems.

2.1.1.4 Steel Deck

Stay-in-place metal deck offers substantial forming economy over wood and other formwork and shoring systems for concrete slabs. Caution should be given to the use of commercial galvanized deck (G-60) as it is prone to corrosion from chlorides that leak through the slab. If the speed of construction and economy of metal deck is especially attractive, the owner should be made aware of the possibility of localized rusting or staining of the deck. With a stay-in-place form this is an aesthetic, non-structural concern.

Galvanized metal deck in some parking garages is performing well, no doubt a reflection on the attention given to crack control, joint seals and fastening of metal deck seams. At least a G-90 perforated galvanized deck is recommended (i.e., 0.90 ounces of galvanizing per ft^2) for parking deck applications, as is welding or mechanical fastening of the side lap seams. Button-punching of side lap joints appears to increase the likelihood of leakage through the seam and corrosion of the underside of the deck. For extra protection a high-performance, compatible paint system should be applied to the exposed underside of the deck after installation in areas where road or marine salts are present.

There are only three conditions for which composite metal floor deck should be used in open-deck parking garages:

- As a stay-in-place form only, not relied upon as tension reinforcement for the slab
- As tension reinforcement in temperate climates, but with tension reinforcing steel in the slab as well as a backup
- As the sole tension reinforcement for a slab in a deck system that has been designed, by necessity, to be leakproof

An example of the last condition is the bottom level of a car park having finished occupied space below. Leakage through this level is unacceptable. A typical solution is to sandwich waterproofing and insulating membranes between the structural slab and a good quality paving slab above. A high priority should be placed on providing the best possible surface drainage for the paving slab, and use of a membrane system should be considered. Fortunately, an insulated structural slab in this application is not likely to be exposed to freeze-thaw cycles or extreme temperature changes.

2.1.2 Cast-in-Place Post-Tensioned Slabs and Toppings

Post-tensioning a site-cast concrete slab in a steel-framed parking garage minimizes intermediate joints and crack formation and helps to limit the width of cracks that do form. However, post-tensioning will increase elastic and creep shortening of the concrete slab.

Bracing or shear wall locations should be near the center of mass of the slab to reduce the possibility of restraint cracks. Extra care should be taken to isolate the slab from any rigid elements near the outer portions of the slab.

Post-tensioning can be done in one or both directions. Ideally, under real service loads, no tension should exist in the top of the slab in the direction(s) of post-tensioning. Some designers prefer not to post-tension in the direction of composite beams, as it is difficult to estimate the portion of the post-tensioning force being absorbed by the composite beams themselves. Unpublished tests performed by Mulach Parking Systems showed a maximum stress increase of three percent. At the least, one would expect a non-uniform distribution of post-tensioning force across the slab. Indeed, unusual patterns of hairline cracking have been observed in a few post-tensioned composite decks. However, slabs that have not utilized longitudinal post-tensioning have been noted to exhibit significantly more cracking in the affected direction and post-tensioning in both directions is encouraged.

The post-tensioned slab is somewhat more expensive than the conventionally reinforced, cast-in-place slab. In some regions there is reluctance to use post-tensioning due to a lack of availability of an experienced labor force and local concrete contractors with post-tensioning expertise.

Design recommendations issued by the American Concrete Institute and the Post-Tensioning Institute should be observed.

2.1.3 Precast Double Tees

For the long-span parking module, 10, 12 or 15 ft wide by 24 to 32 in. deep precast, prestressed double tees supported by steel framing are typical. This system, with both its frame and concrete deck shop fabricated, has a very fast erection time when both products are delivered in a timely and coordinated fashion to the job site.

Other advantages of double tees include:

- Better control and assurance of concrete quality due to prefabrication at a plant;
- Elimination of negative moments in the deck elements, as they are mostly simple span;
- Inherently low permeability and better resistance to penetration of chlorides if steam-cured, because steam curing of the double tees decreases size of capillary pores.
- Low cracking as a result of the prestressed condition of the element

One of the concerns about all precast parking structures is stability during erection. A solution to that problem is to use double steel columns and beams at interior supports. Each double tee frames into its own beam at both ends, and this avoids the large torsional loads that occur when placing the first bay of panels onto a common beam and concerns about adequate flange width to accommodate tees from two sides. The two steel columns are normally spaced 3 ft apart and tied together to form a mini-frame, which provides lateral load resistance in the long-span direction. The space between the tee ends and supporting beams can be used as a drainage pipe chase. The tee ends are bridged by a well-detailed strip of high quality site-cast concrete, which is later sealed.

If prestressed double tees frame onto one common beam, joints should be sealed with sealant systems that accommodate movement and end rotations. Joint surfaces and installation of sealers are especially important. Whatever the detail over the beams, a joint seal should be specified that is compatible with the behavior of the long-span double-tee deck system.

With double-tee decks, particular attention must be given to the longitudinal joint at abutting flanges. Every foot of joint is a foot of potential joint breakdown, leakage and subsequent deterioration of embedded metals. It is recommended that a high quality traffic-bearing polyurethane or silicone sealant be applied to longitudinal joints. Care should be taken with silicone sealants as their softness presents a possible trip hazard in pedestrian traffic areas. As a backup, all metal passing through the joint can be stainless steel, painted or galvanized for corrosion protection.

In years past, site-cast structural toppings were placed on the precast deck to help prevent joint leakage and to provide a more true, jointless surface. Toppings are subject to cracking, delamination, initial shrinkage and debonding. They are placed on concrete panels that are themselves relatively stable. For these reasons, unless diaphragm action is required, precast, prestressed double tee decks in parking structures are often left untopped and protected with penetrating sealers. In applications when the seismic response modification factor *R* is taken greater than 3, the need for a continuous diaphragm requires a reinforced topping slab.

With untopped double tees, differential camber between adjacent panels must be more carefully controlled, and be limited to a $^1/_4$ in. maximum in the driving lane area. Excessive differential camber compounds the wear and tear of joint seals; it can be controlled by minimizing the design prestress force and by field adjustment using jacking and shimming plus pour strips.

2.1.4 Other Systems

2.1.4.1 Filigree

The Filligree deck system consists of a precast, prestressed 2.5-in. concrete panel, usually cast off-site then shipped, erected and used as the formwork for a $3^1/_4$-in. topping compositely cast with the form. The system has been used in building construction for at least 35 years, originally supplied under the trade name "Filigree." That system is still produced, and in some regions local precasters are supplying competitive systems.

The precast form is usually supplied in 8-ft widths and lengths up to 40 ft, which can span two bays. The form is precast with steel elements protruding from it that develop the composite action with the site-cast topping. Filigree has most of the required reinforcing steel and supports set into the panel, but the concrete contractor must add some nominal reinforcing steel in the negative moment region, over the beams in the topping slab. Using spans of 18-ft precast formwork, little or no shoring is required. The steel beams are also composite with the topping, which is cast around standard shear connectors. For the two-bay panel holes are cast at the plant for the shear studs, which are field welded to the beam flanges. Joints should be tooled in the cast-in-place topping immediately above the joints between the filigree panels.

Parking garage owners should require some on-site presence of the supplier of this deck system during construction. The "system" is not just the precast form but the two components. The site-cast topping, like all structural concrete toppings, is subject to differential shrinkage and movement, and the panels must fit tight and proper field concreting procedures must be followed. Minimal shoring, depending on the supporting framing scheme, is usually required. Contractors not familiar with this deck system should become thoroughly familiar with it, including seeking the assistance of the supplier and/or designer prior to start of construction.

2.1.4.2 Hollow-Core Plank

Hollow-core precast plank has been popular as a floor system in residential buildings, either on steel framing, masonry bearing wall framing or concrete framing. However, neither the concrete mix nor the plank configuration is particularly designed or controlled for the challenging exposure of the open-deck parking garage. The hollow cores in the plank may accumulate water, and the top and bottom elements are slender so there is minimal cover for prestressing steel. For these reasons, hollow-core plank is not recommended for open-deck parking structures.

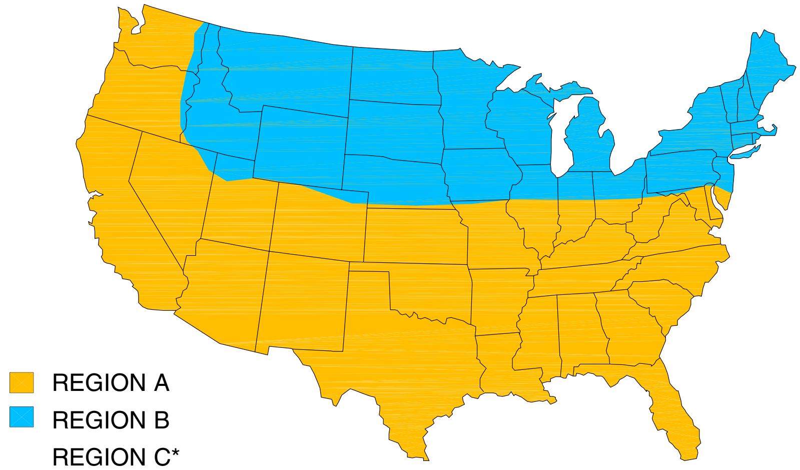
2.2 Deck System Selection by Climactic Zone

Deck system selection is a reflection of the particular climactic and environmental conditions. Such durability considerations are summarized for U.S. exposures in Figure 2-1.

2.3 Concrete Durability

The quality of concrete used in the deck system is very important. Care must be taken to ensure maximum concrete durability. The following considerations should be taken into account when specifying the concrete material:

- The minimum 28 day concrete strength should be 4,500 psi
- The minimum cementious material content should be $6^1/_2$ bags per cubic yard



*Region C is defined as any site within ½ mile of a salt water body

Fig. 2-1. Map of Durability Regions

Table 2-1 Deck System Performance by Region

Region A	Mild conditions where few freeze-thaw cycles occur and/or deicing salts are not typically used on roadways
Region B	Areas where freeze-thaw cycle is typical and deicing salts are used on roadways
	Costal zones within .5 miles of body of salt water

System Type			
Cast-in-place conventionally reinforced on metal deck	A serviceable deck suitable for the climate. It should be treated with a sealer.	With a membrane coating, this deck system is susceptible to cracking. Not the system to be used in most cases for a stand-alone garage.	With a membrane coating, this deck system is also susceptible to cracking. Underside of galvanized metal deck should be painted.
Cast-in-Place Post tensioned slab	Sealed slab not required - with a sealed slab, more durable than climate requires	With a sealed slab, historically the most durable deck for this climate zone	With a sealed slab, historically the most durable deck for this climate zone
Precast, pre-topped Double Tee	With a sealed slab, a suitable deck depending on overall cost and precast tee availability. Site geometry should be reviewed as best suited to rectangular floor plans.	With a sealed slab, tees provide a durable deck. However tee to tee joints require replacing every 6 to 8 years	With a sealed slab, tees provide a durable deck. However tee to tee joints require replacing every 6 to 8 years
Cast-in-place Post tensioned slab on filigree precast form	With a sealed slab, a more durable deck than the climate requires. Probably the highest cost of construction. Filigree forms should be checked for availability and cost.	With a sealed slab, a reasonable deck for the climatic zone. However cost and form availability must be checked.	With a sealed slab, a reasonable deck for the climatic zone. However cost and form availability must be checked.
Cast-in-place conventionally reinforced on filigree	With a sealed slab, a suitable deck. The filigree forms should be checked for availability, cost and site geometry.	With a membrane coating, this conventionally reinforced deck is susceptible to cracking, especially plank to plank. Cost and form availability must be checked.	With a membrane coating, this conventionally reinforced deck is susceptible to cracking, especially plank to plank. Cost and form availability must be checked.

Table 2-2 Foundation Loads by System

System	Estimated Foundation Load
Cast-in-place concrete frame with post tensioned concrete beams and girders and one-way post tensioned slab	107 psf
Precast, pre-tensioned, pre-topped doubles on a precast concrete frame	96 psf
Precast, pre-tensioned, site-topped double tees on a precast concrete frame	113 psf
Non-prestressed cast-in-place composite concrete slab on precast, prestressed joists and beams and concrete columns	108 psf
Precast, pre-tensioned beams and girders with one-way post tensioned slab on site-precast columns	105 psf
Precast, pre-tensioned beams and girders with composite CIP/plank slabs and site-precast columns	111 psf
Structural steel frame with cast-in-place, one-way, composite, post tensioned slab	75 - 82 psf
Structural steel frame with cast-in-place conventionally reinforced deck on stay-in-place metal deck	55 – 75 psf
Precast, pre-tensioned, pre-topped double tees on a structural steel frame	96 psf
Cast-in-place. Non-prestressed, short-span concrete	125 psf
Precast, prestressed short-span concrete	130 psf
Cast-in-place, post tensioned, flat plate short-span concrete	125 psf

- The minimum entrained air content should be 6 percent plus or minus
- The maximum water to cement ratio should be 0.4
- The minimum of a 1.5 in clear cover at the top of the deck for all reinforcing steel
- Strict adherence to ACI chloride levels must be used for new concrete

In addition to the above minimum concrete material parameters the following alternatives should be considered since even small increases in material costs during construction can reap large benefits in durability:

- An encapsulated post-tensioned system
- Calcium nitrate corrosion inhibitor
- Silica Fume
- Fly ash or other pozzalan
- Galvanized reinforcing steel
- Epoxy-coated reinforcing steel

As noted earlier, concrete parking decks require protective coatings. Leaving a concrete parking deck untreated is similar to leaving an exposed steel column unpainted. Protective coatings come in two categories, sealers and membranes. The cost, application, and protection afforded is vastly different. It is important that the proper material be chosen for use that meets the needs and requirements of the structure and owner.

Concrete sealers are a one step, light coating that is spray applied then brushed in to achieve maximum penetration on the concrete surface. They are designed to prevent water and water-borne salts from penetrating the concrete deck. The sealers themselves are not designed to be waterproof. A good sealer should allow the concrete to breathe, or allow vapors to escape. Sealers are most effective in protecting un-cracked concrete surfaces.

Concrete membranes are designed to be waterproof and are not a light one-step spray application like sealers but a heavy, multiple-step squeegee or troweled on application. Membranes are not designed to and cannot bridge cracks in the slab other than microcracks. There are also some one-step coatings available that are much heavier than a sealer but not as heavy as a three-step membrane.

If a deck system has occupied areas below the deck regardless of whether or not the deck system has a propensity to crack, a membrane coating should always be used and a plaza deck system should be considered.

2.4 Plaza Deck Systems

A plaza deck system is a multiple-layer system that provides added redundancy and protection against wear for a membrane system. Plaza deck systems are more expensive than typical membrane systems, but they may be selected to:

- Protect occupied space below
- Reduce membrane maintenance
- Meet architectural and aesthetic needs of the deck

Unlike typical membrane systems, which are directly exposed to traffic, plaza deck systems have a membrane protected by a wearing surface and a secondary drainage system. The components of a plaza deck, from top to bottom, include:

- Wearing surface
- Slip sheet
- Drainage layer
- Insulation (over occupied space)
- Protective board
- Waterproofing membrane
- Structural Slab

The plaza deck system should be designed to drain both the surface water and any water that filters through the deck system and collects on top of the membrane. The drains must contain weep holes below the surface level to accommodate the drainage from the membrane surface. Both the wearing surface and the sub-surface drainage layer should have a slope of $^1/_4$ in. per ft and an absolute minimum slope of $^3/_{16}$ in. per ft. If this minimum slope requirement is not met, the system will be highly susceptible to deterioration and leakage.

2.5 Deck System Design Parameters

Codes prescribe a minimum uniform live load of 50 pounds per square ft and a concentrated load of 2000 pounds applied over an area of 20 in.^{2} at any point on the deck. The code-prescribed minimum live loads listed above must be considered in the design. Additionally, a well-designed deck must account for the realistic loading of the structure. Realistically, the typical live load on the structure is approximately 30-35 pounds per square ft. This is found by considering a compact car in the smallest parking space in a garage (7.5 ft by 15 ft). This compact car space occupies an area of 113 ft2 and the weight of a compact car that could fit into a space that small is approximately 3,200 pounds. Allowing for an additional 500 pounds for four occupants, the realistic loading by the vehicle is a weight up to 3,800 pounds or 33 pounds per square ft. This does not account for usually unloaded areas such as driving lanes, etc.

Although conservative, a realistic live load on the order of 30 pounds per square ft must be checked as a rolling load or as pattern loading on slabs. This analysis will yield different reinforcing patterns than a simple code-specified loading, and the more conservative of the two designs should be used. When designing a post-tensioned slab, in addition to the code-specified load, the slab must be checked using a live load of 20-25 pounds per square ft or skip loading, but permitting zero tension in the top of the slab.

The foundation system for the parking structure must be investigated prior to selecting a deck system. Local soil conditions should be determined through soil borings and geotechnical testing by a qualified geotechnical engineer. If the site has poor soil conditions and requires deep foundations, a lighter deck would be beneficial, since it would be less costly and more easily installed. Relative weights of various framing systems are listed in Table 2-2. If site geology is such that the supporting underlying strata is not uniform and differential settlement will likely occur, a deck system that can accommodate differential settlement must be used. If the site has large grade differentials, a retaining wall design should be incorporated within the structural design or the ground surface should be sloped back. The deck system must have both the continuity and the structural diaphragm capacity to function as such.

Drainage Parameters for Parking Decks

Next to concrete quality, the most important factor in garage deck durability is drainage. If a parking deck does not drain it will deteriorate rapidly in the areas where water and de-icing chemicals are permitted to pond. This type of deterioration will be more significant in geographic areas where freeze/thaw cycles are a frequent occurrence and large amount of de-icing chemicals are used. In order to achieve proper drainage the topics of deck slope and drain locations and selection must be addressed.

2.5.1 Cast-in-Place Conventionally Reinforced Concrete on Stay-in-Place Metal Forms (see also discussion and figures in section 3.3.1)

Typical Parameters

- Light gauge vented metal decking available in depth of 2 in. and 3 in.
- Gauges from 20 to 16
- Widths of 36 in.
- Galvanized
- Span Range 8 ft to 12 ft
- Slab Thicknesses 5 in. to 6 in. (minimum of 3 in. over top of flutes)

Advantages

1. Low initial cost
2. In a mixed use occupancy, keeps the same type of construction
3. Easiest type of deck to rehabilitate
4. Rapid construction

Disadvantages

1. Metal deck cannot be counted on for reinforcing the slab. The slab must contain sufficient reinforcing to carry the loads imposed on it.
2. The deck requires coating/sealing because of its susceptibility to cracking and corrosion.
3. The exposed metal decking may rust and leave an objectionable appearance if the slab is left unprotected.
4. More joints are present.

Design Approach

- The design of a conventionally reinforced one-way slab poured on a permanent metal deck is the same as other one-way slabs. The end span spacing and reinforcing must be adjusted to achieve a uniform slab thickness. Also the following loading conditions must be used:
  Full dead load and full live load on all spans
  Full dead load and full live load on all alternate spans
- Slab joints in freeze-thaw areas should be set on 10 to 15 ft centers.
- Slab reinforcing must be adjusted to suit the profile of the deck being used.

Other Concerns—Use of Metal Deck

From Steel Deck Institute Manual #30...page 13:

7.1 Parking Garages: Composite floor deck has been used successfully in many parking structures around the country; however, the following precautions should be observed:

1. Slabs should be designed as continuous spans with negative bending reinforcing over the supports;
2. Additional reinforcing should be included to deter cracking caused by large temperature differences and to provide load distribution; and,
3. In areas where salt water; either brought into the structure by cars in winter or carried by the wind in coastal areas, may deteriorate the deck, protective measures must be taken. The top surface of the slab must be effectively sealed so that salt water cannot migrate through the slab to the steel deck. A minimum G90 (Z275) galvanizing is recommended, and, the deck should be protected with a durable paint. The protective measures must be maintained through the life of the building. If the protective measures cannot be assured, the steel deck can be used as a stay in place form and the concrete can be reinforced with mesh or bars as required.

2.5.1.1 Deck Slope

All the areas of a parking deck must be sloped a minimum of 1/8 in. per ft with a preferred slope of 1/4 in. per ft in all areas of the deck whether or not those areas are exposed to the weather. There should never be any flat floors in a garage even in a totally enclosed garage, because the vehicles themselves will bring in rain, snow, and ice. When establishing the slope to the drain the following factors must be considered:

- Camber in a plant-cast precast member. The slope to the drain specified should exceed the anticipated camber in the precast member.

- Deflection in cast-in-place decks. The specified deck slope to the drain should exceed the anticipated deflection of the deck for both dead and live loads. A realistic live load is approximately 20 psf. Usually cast-in-place post-tensioned slabs do not have deflection problems; however, cast-in-place slabs with mild reinforcing are very susceptible to deflection, especially shored slabs, which must be checked.

- Deflection at cantilevered sections. The specified deck slope must exceed all anticipated cantilevered member deflections. Careful attention must be paid to deflections due to concentrated wheel loads, heavy concrete spandrel panels, or heavy planters.

- Concrete wash. There must always be an installation of concrete wash at the perimeter of the garage to drain away for the slab edges and exterior panels. This concrete wash should be a minimum of 2-in. high above the finished floor.

- Drain location and selection. Locate drains away from columns, stairs, elevators, slab edges and walls. Never use an exterior panel or wall to function as part of a drain.

The catch area of drains should be limited to approximately 5,000 ft^2 of area, especially on roof areas open to the rain, snow, and ice. Drains should be specified with a removable clean-out basket that can easily be taken out and cleaned on a regular basis. If the garage has easily clogged drains, no amount of drainage planning will have any effect on the actual drainage of the deck.

2.5.2 Cast-in-Place Post-Tensioned Slabs and Toppings (see also discussion and figures in section 3.3.2)

Typical Parameters

1. Typical effective span range is 18 to 27 ft.

2. Typical thickness of deck is 5 to 7 in. (Function of span/depth ratio of 45.)
3. Usual range of reinforcing content:
   Post tensioning tendons .6 psf.
   Mild reinforcing .6-.7 psf.
4. Spacing between joints (pour strips) should be a maximum of 170 to 200 ft.

Advantages

1. Best choice for Zone III construction, refer to durability map, Figure 2-1.
2. Considered to be most durable deck available.
3. Adaptable to any site geometry.
4. Produces a joint free and crack free deck with very little incidence of leakage and maintenance problems.
5. Very light weight deck (Thin slab-long span) if foundations are a problem.
6. Can tolerate different settlement actions without distress.
7. Low life cycle costs.

Disadvantages

1. A slightly higher initial cost.
2. The in-the-field forming and stripping are weather sensitive.
3. Local field expertise may be lacking.

Design Approach

1. Post tensioned/prestressed design and construction have evolved greatly since it was first introduced.The design itself must consider the following load cases:
   A. Full dead and full live load at 50 psf(Ultimate stress analysis and design).
   B. Full dead and live load at 20 to 25 psf at the following locations. Using services loads (un-factored) analysis and design while permitting no tension in the concrete.
      CASE A: full live load on all spans
      CASE B: full live load on alternate spans
2. When post tensioning always use low relaxation style strands.

Other Concerns—Temperature and Shrinkage

1. Post tensioning should be spaced to produce a minimum P/A of 125 psi for temperature considerations, if used. It is recommended that tendon spacing not exceed 36 in.
2. Structural post tensioning should be spaced to produce a minimum P/A of 200 to 250 psi.
3. The tendons do not induce any force into beam connections when the post tensioned deck changes plane. A composite slab when post tensioning is parallel to the beams which support it, does not induce any appreciable movement into that beam.
4. Lateral frames should be located toward the center of the slab to minimize restraint of the post tensioning shortening, shrinkage and creep.
5. Slab should be isolated from perimeter walls, stairwells or other rigid elements that may cause post tensioning restraint.

2.5.3 Precast Double Tees (see also discussion and figures in section 3.3.3)

Typical Parameters

Plant cast double tee
1. Span Range: Up to 65 ft plus or minus
2. Width: 10 ft, 12 ft, or 15 ft
3. Depth: 32 in. or 34 in.

Advantages

1. Can be erected in freezing weather
2. The tee units themselves are usually crack free because they are prestressed and do not require very extensive rehabilitation. Most of the heavy structural reinforcing is in the tee stems which are well below the deck surface.

Disadvantages

1. The joints may need to be replaced every 6 to 8 years. There are many joints at 10 ft or 12 ft or 15 ft c/c.
2. Care must be taken to seal the tees completely.
3. They require a higher than standard floor height to maintain the minimum seven foot clearance.
4. They require larger than standard exterior panels to conceal the tee’s and beams.
5. They are best suited to a rectangular uniformly spaced project with many typical same spaced bays.
6. It is a heavy system-approximately 80 psf slab weight.
7. The possibility of uneven joints due to camber differences between double tees.
8. Proper site conditions are required to stage double tee delivery.

Design Approach

The precast double tees are always designed by a supplier, a precast manufacturer. However, the design of the double tees can be accomplished by procedures outlined in the PCI Design Manual or they can also be designed by commercial software if the designer wishes to have control over the design.

Other Concerns

- Erection stability

2.5.4 Filigree Precast with Post-Tensioned Deck

(see also discussion and figures in section 3.3.4)

Typical Parameters

Plant cast flat concrete form with truss reinforcing and an integral top bar support system.
1. Typical span range 18 ft requires no shoring
2. Width—8 ft form
3. Depth 2.25 in. with 3.75 in. field applied topping

Advantages

1. Braces the frame during construction.
2. Easier to form than stick forming.
3. The form contains structural reinforcing, bottom mat and some top reinforcing and bar supports.
4. Underside of slab has a smooth uniform finish.
5. Requirements for field placed concrete and reinforcing is reduced.

Disadvantages

1. Tends to crack at panel joints due to planking action.
2. Is usually a higher cost than stick forming.
3. Is not readily available in all areas.
4. Will result in a thicker, heavier post tensioned slab.
5. Large number of joints requiring caulking.

Design Approach

- The same design approach as the cast-in-place post tensioned slab except using filigree forms will result in a slightly thicker slab.

2.5.5 Filigree Precast with Conventionally Reinforced Slab (see also discussion and figures in section 3.3.5)

Typical Parameters

- Plant cast flat concrete form with truss type reinforcing.
- Span Range—18 ft (no shoring)
- Form Width—8 ft
- Slab Thickness—2.25 in. form plus 3.75 in. topping

Advantages

1. Braces the frame during construction.
2. Erects easily and is faster than stick framing a slab.
3. The form contains structural reinforcing bottom mat and some top reinforcing and bar supports due to truss type reinforcing.
4. The underside of the slab has a smooth and uniform finish.
5. Requirements for field placed concrete and reinforcing are reduced.

Disadvantages

1. Tends to crack at panel joints.
2. Depending on geographic location, may be higher priced.
3. Has the same vulnerability of conventional reinforcing slab for corrosion considerations.
4. Will require additional sealing and caulking efforts to make water tight.
5. Will require a closer support spacing or a thicker slab because it behave like any one-way reinforced slab (Span/depth ratio is plus or minus l/28 l=c/c spans)

Design Approach

The design of a conventionally reinforced one-way slab poured on a permanent stay-in-place precast filigree form is the same as any other one way flat slab. The limiting depth span ratios are as follows:
- Simply supported: height is greater than or lesser than length/20
- One end continuously supported: height is greater than or lesser than length/24
- Two ends continuously supported: height is greater than or lesser than length/28

The end span spacing must be adjusted to achieve a uniform slab thickness. Also the following loading conditions must be used:
- Full dead load and full live load on all spans.
- Full dead load on all spans and full live load on alternate spans.

2.5.6 Precast Hollow Core Slabs with Field Topping

Typical Parameters

Hollow core slabs are plant cast prestressed slabs with internal voids and formed shear keys along their sides. See Figure xx.
Widths 4’ or 8’
Depths 8, 10, or 12 “
Effective span range 25’ to 30’

Advantages

1. Easy erection process.
2. Erection not weather dependent
3. Uniform bottom finish
4. Lower initial cost

Disadvantages

1. Very vulnerable to corrosion due to water and chloride penetration into voids.
2. Due to dynamic rolling loads the shear key joints tend to fatigue and fail.
3. Topping always cracks at plank joints.

Design Approach
- This system is always purchased as a pre-engineered item. However, if the designer needed to check on a design there are charts available in the Hollow Core Slab Design Manuals or in the PCI Design Handbook.

Chapter 3
Framing Systems

3.1 Introduction

For most open, above-ground parking garages, structural design of steel framing is straightforward. Occasionally, due to site constraints, ramping configuration or other factors, a complex framing system with unusual details (such as skewed connections) is unavoidable. In order to avoid substantial cost increases associated with premiums for detailing, fabrication and erection the framing system should be kept as simple and regular as possible. The engineer's greatest challenge is to design a steel framing system that will accommodate expansion, contraction and deflection of the concrete deck such that cracking and other distress of the supported concrete deck will be minimized.

It is recommended that parking structure floor systems be designed using wide-flange filler beams and girders or castellated beams, rather than open-web steel joists or joist girders. Protection of open web steel joists can substantially increase the cost of corrosion protective coatings. Repainting of joists is very costly. In open-deck parking structures, in view of the corrosive environment, the open-web steel joist in the deck system is not recommended, even if the structure can be built "unprotected."

3.2 Economy

ASTM A992 wide-flange shapes and composite construction generally offer the most economical solution for a wide module (long-span) parking structure frame. Unless additional detailing for a high-seismic application (*R* taken greater than 3) is required, lateral load resistance is usually provided by some economic combination of conventional braced frames, moment frames and/or shear walls (in interior elevator/stairwell cores).

The importance of column grid selection has already been emphasized. Economical bay size studies have been done for certain generic building types, but because of all the aspects of functional design, it seems pointless to attempt to identify a "most economical bay size" for open-deck parking structures. Suffice it to say that, in general, long spans in the 55 ft to 65 ft range are cost-effective in detached, stand-alone garages.

For a minor premium in initial cost, a steel-framed parking garage can be designed for loads imposed by a possible future vertical expansion, with very little modification to the existing frame. Additions to a parking garage tend to be needed earlier than planned, so designing for future vertical expansion should be considered. A common technique for accomplishing this is to extend column stubs through the top level of the garage so that future column extensions can be readily spliced to the original columns. The columns are often extended a minimum of 3 ft-8 in. to afford pedestrian protection. The stubs can be initially encased in concrete and serve as a base for light stanchions. The designer should inquire very early if there is any likelihood for vertical expansion in the future (or, for that matter, for future construction of any occupancy above).

A vertical addition in steel can be readily built atop virtually any existing frame, including concrete, assuming that the existing frame can be reinforced or otherwise upgraded where necessary. During erection a mini-crane may be able to operate on the existing tip deck if temporary mats are utilized.

3.2.1 Relationship Between Deck Type and Bay Size Geometry

Bay size geometry is determined by considering the following factors:

- Deck type
- Site size, parking and ramp arrangements
- Headroom constraints
- Budget considerations

Deck Type

Each particular deck type has an optimum span range where it is the most economical. Deviating from this optimum span range may cause inefficiencies in material usage, resulting in increased costs. Optimum span ranges are listed in Table 3-1. The span ranges shown in Table 3-1 work for clear span construction. This is shown on the right side of Figure 3-1. For short-span construction, shown on the left side of Figure 3-1, these dimensions must be adjusted to a multiple of car space. The car space used is usually a full-size car or between 8 ft-6 in. to 9 ft (SUV) wide.

Also note that when using the precast double tee deck the bay width dimension shown in Figure 3-1 should be in a multiple of standard tee widths. Standard tee widths are 10 ft, 12 ft, and in some locations, 15 ft. It is common practice to utilize bay dimensions that are multiples of the selected parking stall width. While this may not be necessary if interior columns fully span the bay (typically 60 ft), it is still wise to locate columns at the extension of the parking striping to clearly delineate spaces and handle end conditions at turning bays. The designer should contact the precast manufacturer servicing the project area for their standard manufacturing widths.

Site Size and Parking and Ramp Arrangement

The number of bays shown as bay length dimension on Figure 3-1 is a function of site size, parking arrangements based on accepted standards or local zoning requirements, and ramping layouts. These topics are covered in a separate publication, I*nnovative Solutions in Steel: Open-Deck Parking Structures* (formerly titled *A Design Aid for Steel-Framed Open-Deck Parking Structures*), but only mentioned here for reinforcement and their importance in the selection of the bay geometry.

Headroom Constraints

The designer should be aware of required minimum vertical clearances and corresponding floor-to-floor height restrictions, which may impact the design of the members and in turn the bay geometry. The typical minimum vertical clearances required are 7 ft for typical decks and 8 ft-2 in. for physically disabled van access. The deck should be designed with a 2-in. margin over the minimum clearances.

If the garage is a stand-alone facility with no floor-to-floor height requirements to match an adjacent structure, the designer can use the optimum deck span ranges, set the bay geometry, and proceed with the design.

However, if there are floor-to-floor height restrictions, member span depths become critical and therefore must be reviewed as to minimize impact on material usage and cost. It is important to note vertical clearance restrictions can come from different directions such as floor-to-floor height set by matching existing or new construction levels or floor-to-floor height restrictions set by ramp lengths and slopes dictated by a small or unusual site.

These restrictions may force the designer to go to short span construction as shown on the left side of Figure 3-1.

3.3 Plan Framing Design

After the deck type has been selected and the bay geometry is settled upon, the framing plan must be addressed. The plan framing design is a function of the specific deck types to be supported, since each type has it's own special details and considerations. The types of plan framing to be discussed are for supporting the following types of decks:

- Cast-in-place conventionally reinforced slab poured on stay-in-place metal decking
- Cast-in-place post-tensioned slab
- Precast double tees

3.3.1 Cast-in-Place Conventionally Reinforced Slab Poured on Stay-in-Place Metal Decking

The usual span for a cast-in-place slab poured on metal deck is approximately 10 ft to 12 ft. This dimension is not the bay width dimension #1 shown in Figures 3-2 and 3-3. This is the dimension between the filler beams. The bay width is set at a dimension that provides for a minimum weight of filler beams and girders. The plan framing is designed in the same fashion as a standard composite commercial project with some minor differences. These are as follows:

1. The conventionally reinforced slab will crack. The designer can implement a joint control pattern that will help alleviate this problem. See Figure 3-4. The slab always cracks over the girder because of the reverse curvature of the slab. See Figures 3-5 and 3-6. These control joints should be sealed with a good quality silicon traffic grade sealer.

2. Knowing the slab will crack, the deck should be opened to traffic and allowed to flex. After the deck has been allowed to crack, the deck should be cleaned by shot blasting, the cracks routed and sealed and then a deck coating applied. A membrane coating should be used for Zone III and a good quality slab sealant in all other zones.

A typical design example is presented in Appendix A-1.

3.3.2 Cast-in-Place Post-Tensioned Slab Framing Plan

The optimum slab span range for a cast-in-place post-tensioned deck is 18 ft to 27 ft. The slab thickness is estimated as the span in inches divided by 45. Typical slab properties, as they are related to their span, are shown on Table 3-5. Typical slab profiles are shown in Figure 3-10. Typical framing sizes are shown in Table 3-6. Examples of calculations appear in Appendix A-2 using ASD and LRFD design methods. The framing itself is designed for strength and serviceability the same way any composite commercial project would be with a few additional considerations:

- The effect that post-tensioning forces have on members and their connections
- Construction loads
- Connection design

3.3.2.1 The Effect That Post-Tensioning Forces Have on Members and Their Connection

Many designers wonder what effect the post-tensioning forces have on members and their connections. Are the post-tensioned forces resisted by the beam itself as shown in the top half of Figure 3-12, or does the post-tensioning act merely as a compressive force on a composite member, producing an elastic strain compatible with the composite members strain diagram, as shown in the lower portion of Figure 3-12?

In reality, this is merely a compressive force acting on a composite member and it is not 100 percent additive to the bending stress as might be concluded. First consider the fact that the slab is going into compression due to gravity loads, both dead and live, and the slab is trying to shrink due to curing. Unpublished testing by Mulach Steel Corporation showed that an increase of 3 percent in the dead load condition that diminished in magnitude with live load application is the net result in the primary spanning beams. In most current conditions, the slab is shored then post-tensioned, then un-shored, thus the elastic shortening of the slab due to both self -weight and post-tensioning occur at the same time and are not additive but concurrent.

An excellent article on post-tensioning considerations for parking decks on steel frames appeared in the 1988, Third Quarter issue of AISC's *Engineering Journal*.

3.3.2.2 Construction Loads

In a typical steel construction project with metal decking, members are braced by the metal deck during erection. Very little load is imposed on them and consequently they are almost always laterally braced and stable. In parking garage construction, however, members may often require lateral bracing during erection and therefore construction methods and sequencing become of vital importance to the designer. This is true for all deck systems with the exception of cast-in-place concrete on metal deck.

During construction, either the beam should be designed to support the weight of the concrete form and wet construction of the slab, or supports should be provided for the forming systems. Such support should provide sufficient lateral bracing as shown in Figure 3-13. After the slab has cured and the forms are removed, the capacity of the slab to support the weight of the forms and wet concrete for the pour on the deck level above. See Figure 3-14.

3.3.2.3 Camber

Cambering girders and beams can be beneficial for achieving economical long-span construction. Camber should be limited to 3 in. as excessive camber requirements are difficult to achieve and are not predictable as to whether the camber will be relieved after the dead load is applied.

3.3.2.4 Connection Design

In the design of a conventional steel frame with reasonable spans (30 ft +/-) and light dead loads, the moments due to the self weight of the structure, although significant, are not very large. In the design of garage members, however, the spans are large and the weight supported by the members is considerable. As a result, the self-weight moments are quite large. Considering this, the designer should be cautioned about using a partially restrained moment frame unless its performance at these force levels is considered. The use of a staged connection, as shown in Figure 3-15, that can be made rigid after the slab is stressed is suggested.

3.3.2.5 Member Design in Direction of Primary Reinforcing

The number of beams spanning in the same direction as the primary post-tensioning should be limited so as to limit restraint cracking. Those beams that cannot be eliminated should be made non-composite.

3.3.3 Precast Double Tee Deck

Precast double tees can span up to 65 ft +/-. The width of the tees is typically 10 ft to 12 ft. The bay spacing is set up as a module of the typical double tee width of either 20 ft, 24 ft, 30 ft, or 36 ft. See Figure 3-16. The tees span the long direction, while the girders span the short direction. The actual design of the precast double tees is usually done by the precast manufacturer due to the variation in casting beds, strand sizes, and stressing bulkhead layouts. Also, when using double tees, the floor-to-floor heights must be increased to accommodate the deeper construction depth. See Figure 3-17. When designing a steel frame that supports a double tee deck, there are differences that the design must accommodate. The designer must consider the following in the design of tee-supporting girders:

- The girders will not be laterally braced for their entire length, particularly during construction. See Figure 3-18.
- If a beam supports tees from both sides, specify the construction sequence and check torsional and un-braced loading the girder can experience during construction. Also check that the flange is actually wide enough to accommodate bearing for two tees.
- The double tees must be detailed in such a way that they do not induce torsion on the steel beams. See Figure 3-19.
- Make sure the beam flange and web can accommodate the large point loads imposed by the double tees. See Figure 3-20.
- Continue the double tees beyond the beams so as not to induce torsion in the members. See Figure 3-18.

Since the double tees span the bay length dimension noted as #2 in Figure 3-16 and the supporting girders span the bay width dimension #1 there is no steel framing spanning in the direction of dimension #2, except what is required for frame lateral resistance. The designer must select locations and design the appropriate number of rigid frame bays as required. See Figure 3-16.

Girder-to-tee connections are unique because tees require bearing on elastomeric pads. Refer to Figure 3-19 for typical details. To complete diaphragm actions, the tees must be connected to each other. Typical tee-to-tee connections are shown on Figure 3-21. For typical girder sizes, see Table 3-8. For typical girder design examples, see Appendix A-3.

3.3.4 Cast-in-Place Post-Tensioned Slab on Filigree Forms

The cast-in-place post-tensioned deck on Filigree forms follow the geometry of a post-tensioned deck in that the typical spans range from 18 to 27 ft. The slab thickness is estimated as the span in in. divided by 45, however as a practical matter the total slab thickness should not be less than 6 in. (compared to 5 in. for a slab cast on removable forms). The thicker slab is required because of the thickness of the concrete form. The filigree form must be shored for spans greater than 18 ft. The manufacturer should be consulted for specific slab span/thickness conditions. This combination of post tensioning will carry a cost premium but will combine better crack control with a more uniform underside slab finish. Also the same concepts for the post-tensioning effects on members and their connections, construction loads, and connection design as previously listed in Section 3.3.2 are applicable. (See Figure 3-22.)

3.3.5 Cast-in-Place Conventionally Reinforced Slab on Precast Forms

The typical effective range for conventionally reinforced cast-in-place slab on Filigree Forms is up to 18 ft and should conform to the typical span/depth limitations used for conventionally reinforced slabs. Slab thickness can be estimated from the conditions listed below:

Support Condition	Minimum Thickness
Simply Supported	Span (in.)/20
One End Continuos..................	Span (in.)/24
Both Ends Continuos	Span (in.)/28

Because this deck is conventionally reinforced it will be susceptible to cracking over the girder as well as between the panels. Accordingly, the engineer should employ crack control measures similar to those illustrated for the cast-in-place slab on metal deck in later sections. With proper crack control and joint sealer /deck coating application this combination can provide a deck with desirable visual appearance. It is also a good choice for multi-use facilities in which the owner or architect wants an upgraded appearance. (See Figures 3-23–3-27.)

3.4 Other Framing Considerations

3.4.1 Connection Type: Rigid or Semi-Rigid

Connection type selection is critical in parking structure construction. Parking structures differ from typical commercial construction due to the span of the members and the weight that they support. This subject has been briefly discussed in the post-tensioning deck section but will be covered in greater detail in this section.

For example, it is common for a parking structure beam to be 60 ft in length supporting a dead load of 1.1 kips-ft and live load of 0.9 kips-ft, requiring larger than normal camber. If a fully restrained moment frame approach is selected, and the beam-to-column connection is used to develop the full end fixity of the member, the design moment will be in the range of 600 kip-ft. Designing both the column and the connection for the large moment leads to an efficient economical frame design. Conversely, using a partially restrained moment frame approach would lead the designer to a huge disparity in end-connection design values, especially at the roof level or upper level floor beam. More importantly, how does the end connection behave or deform when the camber is relieved in the beam? If the construction logistical challenges can be overcome, a staged connection approach can be used that is free to rotate while dead load is applied and fixed before live and lateral loads are applied. For an illustration of this concept, see Figures 3-28, 3-29 and 3-30.

3.4.2 Composite Beams

Composite beams are widely used in commercial construction for both economy and function. Parking structure construction is no different. Composite beams should be used whenever possible. The following is a list of deck types and their composite classification:

Deck Type	Composite?
Cast-in-place post-tensioned	Yes
Cast-in-place on metal deck	Yes
Cast-in-place on Filigree	Yes
Precast Double Tee Deck	No

The only deck type that precludes the use of composite beams is the precast double tee deck, as there is no way to develop any sort of effective composite action between the precast double tees and the steel beams. The actual mechanics of composite beam design are covered in other AISC publications, and will not be addressed here, however Figure 3-31 illustrates the typical concrete deck to steel beam construction that creates the composite action for the various deck types.

3.4.3 Shored Versus Un-Shored Composite Beams

In composite beam design, it is important to consider whether or not to shore during construction. Shoring can substantially add to the cost and schedule of construction in office and commercial buildings, and can interfere with mechanical and electrical trades that are eager to begin work as each floor is installed. In an open parking garage, however, these other trades have minimal impact, and the presence of shoring should not significantly affect construction schedule.

Although shoring may provide better control for leveling floors, concrete cracking is more likely to occur over girders in shored construction, and the long-term creep loading of the concrete slab itself is more of a concern. Since level floors are not a design or construction objective, it would seem that unshored composite construction with cambered beams may have more advantages in achieving a durable concrete deck. The paper "Cambering of Steel Beams," by Lawrence Kloiber in the *Proceedings* of the ASCE Structures Congress '89, ASCE, May 1, 1989, suggests that composite beams should generally be cambered for dead load of the wet concrete, the super-imposed dead load, and a part of the long-term live load. A minimum length of around 24 ft is suggested for beams that are to be cold-cambered. Because of the need to have the connection face of beam ends vertical, beams with moment connections probably should not be cambered.

The decision to provide shoring and the amount of shoring required will depend on the details of the deck system, spans, the ability to camber beams and other factors.

3.4.3.1 Cast-in-Place Post-Tensioned Deck

If the deck forming system is self-supporting from either the ground or the slab(s) below, it is considered to be shored because when the weight of the slab is transferred from the slab shores to the beam, the beam will be composite as in the upper portion of Figure 3-32. If the deck forming system is supported by the beam such as in the lower portion of Figure 3-32, a panelized system the beam must be designed as an un-shored beam. It is quite important that the designer know and specify what type of forming system is to be used. Also note, when using a forming system that is supported by the steel frame, the beams must be braced laterally, and unbalanced loading from wet concrete placed on one side of the beam must be considered in the design. Finally, the designer must specify the designation of shored or un-shored construction on the drawings.

3.4.3.2 Cast-in-Place Slab on Metal Deck

The cast-in-place slab on metal deck system can be either shored or un-shored. The decision to shore is usually influenced by such factors as convenience and the availability of either grade or an existing deck below to shore to. Only in a very small set of circumstances is it cost effective to shore. The designer should consult with local contractors to evaluate the cost-effectiveness of shoring and as always specify shoring criteria on the drawings.

3.4.3.3 Cast-in-Place Slab on a Filigree Deck

Usually if the filigree deck spans are below 18 ft and the deck does not require shoring, it is probably not cost effective to design beams for shored construction. On the other hand, if the filigree deck needs to be shored the designer should design the beams for either the reduced load as un-shored or designed as shored. Shoring in a multi-story application is almost impossible. The designer should consult with a local contractor to see which is more cost effective and as always specify either shored or un-shored on the drawings.

3.4.4 Non-Composite Beams

The only decks that drive the designer to a non-composite beam design are the precast double tee deck and short span concrete. All others should be composite. Please refer to the precast double tee deck section for details.

3.4.5 Castellated Beams

This system uses steel beams, cut longitudinally mid-web to create two long toothed pieces, and then the two pieces are offset and welded to form a stronger and deeper web with either hexagonal or round holes. Castellated beams can be very economical in long-span construction. Castellated beams can be used with galvanized metal deck to form a cast-in-place concrete slab or with a shored post-tensioned slab. They create a sense of openness in a parking structure, as the holes in the beam webs allow light to pass through. The design of castellated beams is specialized and the designer should consult with a manufacturer for technical assistance when using them.

3.4.6 Perimeter Beams

If the design of a parking garage requires an exterior architectural precast panel connected to the column, a beam at the perimeter is not required. Many garages have been built successfully using large precast panels for the structural element at the exterior. The panel's size and stiffness make it a substantial perimeter member. Listed below are a few details that must be carefully considered when using an exterior panel for a perimeter structural element.

- The panel must be tied into the slab in order to make it effective (lateral braced). See Figure 3-33

- Panels must be attached to columns with details that facilitate erection as well as accommodate future slab deflections. See Figure 3-34

- Panels must contain sufficient reinforcement for in plane loads as well as out of plan loads (car impact)

3.4.7 Steel Joists

Steel joists should not be used in parking structures. Vibrations and deflections inherent in joist systems create crack control problems for the deck system. Joists can also create unique challenges for the application and maintenance of high performance coating systems.

3.4.8 Control/Expansion Joints

There are three basic types of joints in a structure:

- Construction Joints

- Control Joints

- Expansion Joints

Construction Joint

Construction Joints are used in structures with cast-in-place deck and are most effectively located between ramps, or if this is not possible, at the quarter point of the slab span. The purpose of this type of joint is to define the boundaries of each day's concrete pour. See Figure 3-36.

Control Joint

Control joints are used for crack control. They are joints that are tooled, cut or formed (by plastic strips) into conventional reinforced slabs at points were cracks are expected or to break up slab widths in order to relieve slab shrinkage stresses. (See Figure 3-37.)

Expansion Joint

Expansion joints are used to break up contiguous lengths of construction. There is a physical limit to how much of a structure can be contiguous before thermal effects will cause distress to the structure. Therefore the designer should check a thermal map of the United States (Figure 3-38) for control joint spacing.

When an expansion joint is introduced, the structure must be designed as two independent structures.

3.5 Vertical Framing Design

The vertical framing design of a parking structure is similar to typical commercial projects except for the following:

- The structure will never be dimensionally stable because it is not in a thermally controlled environment. The structure will expand and contract with changes in ambient temperatures. As mentioned previously this expansion and contraction will occur about the center of the mass of the deck. The overall length of the deck will vary from floor-to-floor and is also affected by the time of day. For example, the top floor may be 30° warmer than the first supported level due to warming of the sun. This warming will cause the deck to lengthen.

- The behavior of materials used to construct the deck will not be the same.

- Concrete elements will shorten from their original lengths due to curing, shrinkage, creep, and elastic shortening depending upon such factors as prestressing levels and post-tensioning forces. Another factor is concrete quality such as water-cement ratios, aggregate size, curing.

- Steel does not shrink but does expand and contract with temperature variations. Of importance is the fact that steel and concrete expand and contract at different rates. Relief joints must be utilized when there is a long contiguous element of concrete together with a long contiguous steel element.

3.5.1 Lateral Load Considerations

In applications with the seismic response modification factor R taken greater than 3, it is advantageous to use the most cost-efficient lateral system possible and locate braces linked on the exterior of the building. Consideration should be given to avoiding architectural details that may impact the location of these braces and unnecessarily increase the cost of the frame.

3.5.2 Braced Frames

Braced frames are in general simpler to design in conventional construction than a moment frame. However, in an open parking structure they require additional planning and detailing. This is due to:

- Length change due to thermal effects

- Shortening of the deck due to concrete shrinkage and creep

- Effect on aesthetics and parking functional issues

3.5.2.1 Length Changes Due to Thermal Effects

The designer must consider that the design is for a structure whose length will vary substantially. The idea that the length of a structural element will vary with temperature changes is not a new concept to structural engineers. However, in ordinary commercial type design temperature is not a concern because most conventional commercial projects are heated and cooled in order to maintain a constant temperature and consequently a constant length. An open-deck parking structure is at the ambient temperature, and thus it will change length. Please refer to Figure 3-38 that gives the maximum seasonal climactic temperature change contours for the United States. Figure 3-38 shows that a garage in a Southern State may only experience a maximum temperature variation of 30 °F. A garage in one of the Northern States on the other hand could experience a temperature variation up to 100 °F. Due to this temperature variation, coupled with the fact that most garages are long structures, 300 ft or more, expansion joints are not uncommon. Also in a garage of multi levels different floors will be at different temperatures at different times. The roof level exposed to the sun will be substantially warmer than the levels below it.

3.5.2.2 Shortening of the Deck Due to Concrete Shrinkage and Creep

As all engineers are aware of concrete wants to shrink as it cures. The rate at which it will do so is subject to many variables such as:

- The concrete mix itself (water to cement ratio, etc.)
- Curing (water cured, chemically treated, or no cure at all)
- Weather conditions that the concrete is subjected to during curing (humidity, temperature, wind, etc.)
- Type of reinforcing (post-tensioned, prestressed, or conventionally reinforced)
- The strength of the concrete (at the time of stressing)

The effect that the concrete shortening will have on the structure's length is also dependent on several factors such as:

- The presence of beams framed at the column lines or precast panels (See earlier discussion)
- If there are beams framed on the column lines, how large are they and do they have moment resistant connections?
- Are there expansion/contraction joints in the structure?

It suffices to say that an open structure will not stay the length it was when constructed for some or all of the above reasons. The next section describes the importance of these length changes.

3.5.2.3 Length Changes and How They Relate to Bracing

The designer knowing that the structure will vary in length can plan the location of the braced bay. This planning should be done to minimize the effect that the length changing has on the bracing. The relationship of the center of mass and the center of rigidity should be particularly considered in seismic zones. Never locate the bracing at the ends of the building. Please refer to Figure 3-39. Locating the braced bay at the end of the building could result in a buckling/tension failure of the bracing members and/or their connections. Conversely, if the bracing were designed to resist the shortening/lengthening of the structure it would cause additional stresses or cracking in the deck.

3.5.3 Shear Walls

In many enclosed commercial projects with thermally controlled environments the elevator/stair shaft walls are utilized as shear walls to provide lateral stability for the structure. From a practical standpoint the elevator/stair shaft wall must be constructed anyway and the additional cost of added reinforcing to upgrade the shaft walls to shear walls is far less than introducing braced bays or moment resistant frames. The above design of a shear wall as described is not very complex because the only forces on the shear wall are the lateral forces it must resist. On the other hand, the design of a shear wall in a parking structure is quite complex and if not properly planned, the design will not be successful. Shear walls are typically constructed of reinforced concrete or reinforced masonry. Neither of these materials are as elastic or forgiving as steel bracing. The open structure variation of length that was described for the braced bay structure applies to shear walls also and the designer needs to be even more concerned with the effect these changes in length will have on shear walls. Many early garage structure designers tried to utilize the stair shafts that were located at the ends of the building, as shear walls. The stair shafts, being very rigid elements, tried to resist the structure's changes in length. This conflict resulted in distress to the masonry, eventual failure of the connections of the deck to the masonry, and loss of the lateral restraint system of the structure. Please refer to Figure 3-40 for an illustration of this point. Also a very important detail that requires the designer's attention in using a shear wall is the wall to deck connection. If the shear wall is not used for load bearing purposes, the deck to shear wall connection is simply reduced to an attachment of one element to another.

The designer is cautioned to provide a connection that will permit vertical deflection of the deck member while restraining lateral movement of the structure. Refer to Figure 3-41 for an illustration of this concept. Failure to accommodate this deflection will result in the connection transmitting vertical load that it not designed to do.

3.6 Erection Considerations

Steel-framed parking structures require additional considerations over and above traditional commercial buildings. These considerations fall into two categories; those appropriate to all steel-framed parking structures and those that apply to specific deck types.

3.6.1 Considerations For All Steel-Framed Parking Structures

Usually steel erection consists of beams, columns, joists, deck and studs. However, parking structures have more erectable components such as:

- Precast architectural panels
- Barrier systems including guard rail, barrier cables, etc.
- Stairs, hand rails, etc.

These components must be scheduled, coordinated, and erected with the steel frame to save time and cost. There are scheduling and cost benefits derived from having a single erector with one mobilization erect the additional components listed above. If more than one erector is used, there may be no or limited crane access to erect these components after the steel is erected. A normal steel frame erection is stable once the deck and connections are complete. With parking structures this may not always be the case, especially if the deck type is cast-in-place, since the deck is required for stability. Conditions both during construction and in completed structures should be reviewed to evaluate the need for any special temporary shoring. Also if barrier cables are used, the erector must be advised of pre-tensioning forces and the engineer must consider the pre-tensioning forces in the design. All of this coordination should be done in accordance with responsibilities established in the contract.

3.6.2 Considerations for Deck-Specific Types

Listed below are deck-specific types of additional erection considerations:

- Cast-in-place post-tensioned deck may require the following: Additional temporary bracing cables that must be left in place until a sufficient number of decks are poured to ensure frame stability. The issue of shored versus un-shored construction is extremely important. For un-shored construction the frame must be checked for unbalanced form loads causing torsion during concrete pours. All the beams and girders must be laterally braced either by the forms themselves or sub-forming which can be permanent or temporary. In shored construction the deck must be designed to carry the weight of the wet concrete pour above it or the designer must specify re-shores to the deck below it.
- Stay-in-place precast concrete form decking requires that the erector be advised of the temporary shoring required for forms. An engineer must evaluate frame stability for all phases of construction in accordance with responsibilities established in the contract.
- Beams supporting forms with either unbalanced loading or long un-braced lengths during the erection of forms and during concrete pours must be checked for stability. Design drawings should advise the erector of a proposed sequence and/or the need to provide temporary shoring or lateral bracing during construction.
- For a precast twin tee deck the erector should be advised of a possible sequence of erection that doesn't cause distress to the frame due to torsion from unbalance loading. The erector must also be advised to provide temporary shoring or bracing to prevent unstable conditions during the construction phase.
- Cast-in-place on metal deck should require no additional considerations other than those listed at the beginning of this section.

Chapter 3
Tables

Table 3-1 Optimum Deck Span Ranges

Deck Type	Optimum Span Range*
Cast-in-place, conventionally reinforced, placed on metal deck	9 feet to 12 feet w/o filler beams 18 feet to 26 feet w/filler beams
Cast-in-place post tensioned	18 feet to 27 feet
Precast Double Tee	55 feet to 65 feet **
Cast-in-place post tensioned placed on filigree deck	18 feet to 20 feet ***
Cast-in-place conventionally reinforced, placed on filigree deck	18 feet to 20 feet ***

Notes
* Span range is for bay width dimension shown in figure 3-1 except for precast double tees which span the bay length dimension
** Precast double tees span dimension shown is for bay length not bay width
***Filigree deck requires temporary shoring beyond 18 feet. Consult with the manufacturer.

Table 3-2 Bay Width Dimensions for Precast Double Tees

Manufacturer's Tee Width	Bay Width 2 Tees Wide	Bay Width 3 Tees Wide	Bay Width 4 Tees Wide
8 feet *	16 feet	24 feet	32 feet
10 feet	20 feet	30 feet	**
12 feet	24 feet	36 feet	**
15 feet ***	30 feet	**	**

Notes
* this is used in an older style and is probably not available
** this bay module is not effective from a steel usage standpoint
*** this size tee has limited availability and designer should consult the area manufacturer

Table 3-3 Typical Beam Sizes for Cast in Place Conventionally Reinforced Slab on Metal Deck—Configuration 1

TYPICAL BEAM SIZES FOR CAST IN PLACE SLAB ON METAL DECK WITH CONVENTIONAL REINFORCEMENT																								
BAY GEOMETRY	BEAM A			BEAM B			BEAM C			BEAM D			BEAM E			BEAM F			BEAM G			BEAM H		
DIMENSION ①	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S
55'–0"	W12x16	0"	10	W12x19	0"	12	W36x160	0"	76	W30x99	1"	66	W33x118	3/4"	94	W14x22	0"	14	W24x62	0"	42	W30x99	3/4"	26
57'–0"	W12x16	0"	10	W12x19	0"	12	W36x160	0"	76	W30x108	1"	88	W33x118	1"	96	W14x22	0"	14	W24x68	0"	46	W30x99	3/4"	26
59'–0"	W12x16	0"	10	W12x19	0"	12	W36x182	0"	76	W30x116	1"	96	W33x130	1"	100	W14x22	0"	14	W24x84	0"	70	W30x99	3/4"	26
61'–0"	W12x16	0"	10	W12x19	0"	12	W36x182	0"	76	W33x118	1"	96	W33x130	1 1/4"	104	W14x22	0"	14	W24x84	0"	70	W30x99	3/4"	26
63'–0"	W12x16	0"	10	W12x19	0"	12	W36x182	0"	76	W33x130	1 1/4"	106	W33x141	1 1/2"	108	W14x22	0"	14	W27x84	3/4"	70	W30x108	3/4"	26
65'–0"	W12x16	0"	10	W12x19	0"	12	W36x182	3/4"	76	W33x141	1 3/4"	110	W40x149	1 1/2"	110	W14x22	0"	14	W30x99	1"	80	W30x116	3/4"	26

NOTES:
1. STEEL IS ASTM A992 (Fy=50 KSI).
2. C – DENOTES CAMBER.
3. S – DENOTES NUMBER OF STUDS.

Table 3-4 Typical Beam Sizes for Cast in Place Conventionally Reinforced Slab on Metal Deck—Configuration 2

TYPICAL BEAM SIZES FOR LONG SPAN FILLER BEAMS AND SHORT SPAN GIRDER WITH C.I.P. ON METAL DECK												
BAY GEOMETRY	BEAM A			BEAM B			BEAM C			BEAM D		
DIMENSION ①	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S
55'–0"	W21x50	1 3/4"	30	W24x62	1"	40	W24x76	–	49	W21x44	1"	28
56'–0"	W21x50	1 3/4"	30	W24x76	1 3/4"	49	W24x76	–	49	W21x44	1"	28
57'–0"	W21x50	2"	30	W24x76	2"	49	W24x76	–	49	W21x44	1"	28
58'–0"	W21x50	2"	30	W24x76	2"	49	W24x76	1"	49	W21x44	1"	28
59'–0"	W21x57	2"	30	W24x84	1 3/4"	54	W24x76	1"	49	W21x44	1"	28
60'–0"	W21x57	2"	30	W24x84	2"	54	W24x76	1"	49	W21x44	1"	28

NOTES:
1. STEEL SHOWN TO BE ASTM A992 (Fy=50 KSI).
2. C – DENOTES CAMBER.
3. S – DENOTES NUMBER OF STUDS.

Table 3-5 Typical post-tension Slab Properties by Span Length

TYPICAL POST TENDON SLAB PROPERTIES BY SPAN LENGTH								
SLAB SPAN	SLAB THICKNESS	POST TENSIONING TENDON SPACING		REBAR SIZE, LENGTH & SPACING				REMARKS
		STRUCTURAL	TEMPERATURE & SHRINKAGE	A	B	C	D	
18'–0"	5"	24"	33"	#4 x 13'–4" @ 16"	#5 x 9'–6" @ 16"	#4 x 9'–6" @ 16"	#4 x 9'–6" @ 16"	
19'–0"	5 1/4"	22"	33"	#4 x 13'–4" @ 16"	#5 x 10'–0" @ 16"	#4 x 10'–0" @ 16"	#4 x 10'–0" @ 16"	
20'–0"	5 1/2"	20"	33"	#4 x 13'–4" @ 12"	#5 x 10'–6" @ 16"	#4 x 10'–6" @ 16"	#4 x 10'–6" @ 16"	
21'–0"	6"	19"	30"	#4 x 13'–4" @ 12"	#5 x 10'–6" @ 16"	#4 x 10'–6" @ 16"	#4 x 10'–6" @ 16"	
22'–0"	6"	18"	30"	#4 x 13'–4" @ 12"	#5 x 10'–6" @ 16"	#4 x 10'–6" @ 16"	#4 x 10'–6" @ 16"	
23'–0"	6 1/4"	18"	29"	#4 x 13'–4" @ 12"	#5 x 11'–6" @ 16"	#4 x 11'–6" @ 16"	#4 x 11'–6" @ 16"	
24'–0"	6 1/2"	16"	28"	#4 x 13'–4" @ 12"	#5 x 12'–0" @ 16"	#4 x 12'–0" @ 14"	#4 x 12'–0" @ 14"	
25'–0"	6 3/4"	15"	26"	#5 x 13'–4" @ 16"	#5 x 12'–0" @ 16"	#4 x 12'–0" @ 12"	#4 x 12'–0" @ 18"	
26'–0"	7"	14"	25"	#5 x 13'–4" @ 16"	#5 x 12'–6" @ 16"	#5 x 12'–6" @ 16"	#5 x 12'–6" @ 16"	
27'–0"	7 1/4"	13"	25"	#5 x 15'–0" @ 16"	#5 x 13'–0" @ 16"	#5 x 13'–0" @ 16"	#5 x 13'–0" @ 16"	
28'–0"	7 1/2"	13"	24"	#5 x 15'–0" @ 16"	#5 x 13'–6" @ 16"	#5 x 13'–0" @ 16"	#5 x 13'–6" @ 16"	

NOTES:

1. VALUES GIVEN REPRESENT TYPICAL VALUE OF DECK OF 5 OR MORE CONTIGUOUS SPACES DESIGNERS SHALL VERIFY APPLICABLILY FOR EACH PARTICULAR PROJECT.
2. REFER TO FIGURE #3 FOR TYPICAL SLAB PROFILES AND DETAILS.

Table 3-6 Typical Beam Sizes for CIP Post-Tensioned Deck

BAY GEOMETRY		BEAM A			BEAM B			BEAM C		
DIMENSION ①	DIMENSION ②	SIZE	C	S	SIZE	C	S	SIZE	C	S
	18	W27x84	1 1/2	28	W28x84	1 1/2	88	W14x22	–	12
	19	W27x84	1 1/2	28	W27x84	1 1/2	96	W14x22	–	12
	20	W27x84	1 1/2	34	W27x94	1 1/4	82	W14x22	–	12
	21	W27x84	1 1/2	44	W27x94	1 1/4	94	W14x22	–	12
	22	W27x84	1 1/2	50	W27x94	1 1/4	110	W14x22	–	12
55'–0"	23	NOTE "A"	*	*	W27x108	1 1/4	104	W14x22	–	12
	24		*	*	W27x108	1 1/4	108	W14x22	–	12
	25		*	*	W27x116	1 1/4	88	W14x26	–	8
	26		*	*	W27x116	1 1/4	120	W14x26	–	8
	27		*	*	W27x118	1 1/4	104	W14x26	–	8
	28		*	*	W27x118	1 1/4	132	W14x26	–	8
	18	W27x84	1 1/2	28	W27x84	1 1/2	28	W14x22	–	12
	19	W27x84	1 1/2	28	W27x90	1 1/4	28	W14x22	–	12
	20	W27x84	1 1/2	42	W27x90	1 1/2	42	W14x22	–	12
	21	W27x84	1 1/2	52	W27x99	1 1/2	52	W14x22	–	12
	22	W27x84	1 1/2	54	W27x108	1 1/2	54	W14x22	–	12
57'–0"	23	NOTE "A"	*	*	W27x116	1 1/2	54	W14x22	–	12
	24		*	*	W27x118	1 1/2	54	W14x22	–	12
	25		*	*	W27x118	1 3/4	54	W14x26	–	8
	26		*	*	W27x130	2	54	W14x26	–	8
	27		*	*	W27x130	2	54	W14x26	–	8
	28		*	*	W27x130	2	54	W14x26	–	8
	18	W27x84	1 1/2	46	W30x90	2	96	W14x22	–	12
	19	W27x84	1 1/2	46	W30x90	2	96	W14x22	–	12
	20	W27x84	1 1/2	72	W30x99	2	124	W14x22	–	12
	21	W30x90	1 1/2	30	W30x108	2 1/4	124	W14x22	–	12
	22	W30x90	1 1/2	30	W33x118	2	62	W14x22	–	12
59'–0"	23	W30x90	1 1/2	34	W33x118	2	108	W14x22	–	12
	24	W30x90	1 1/2	36	W33x130	2	72	W14x22	–	12
	25	W30x90	2	36	W33x130	2	108	W14x26	–	8
	26	W30x90	2	46	W36x135	2 1/4	108	W14x26	–	8
	27	W30x90	2	52	W36x150	1 3/4	60	W14x26	–	8
	28	W30x90	2	56	W36x150	2	86	W14x26	–	8
	18	W27x84	1 1/2	76	W30x99	2 1/4	90	W14x22	–	12
	19	W27x84	1 1/2	76	W30x99	2 1/4	90	W14x22	–	12
	20	W30x90	1 1/4	38	W30x108	2 1/4	114	W14x22	–	12
	21	W30x90	1 1/2	46	W33x118	1 1/2	62	W14x22	–	12
	22	W30x90	1 3/4	48	W33x118	1 3/4	104	W14x22	–	12
61'–0"	23	W30x90	1 3/4	52	W33x130	1 3/4	76	W14x22	–	12
	24	W30x90	1 3/4	62	W33x130	2	120	W14x22	–	12
	25	W30x90	2	60	W36x135	2	100	W14x26	–	8
	26	W30x90	2	72	W36x150	2	70	W14x26	–	8
	27	W30x90	2	78	W36x150	2	100	W14x26	–	8
	28	W30x90	2	84	W36x160	2	86	W14x26	–	8
	18	W30x90	1 3/4	46	W30x108	2 1/4	78	W14x22	–	12
	19	W30x90	1 3/4	46	W30x108	2 1/4	78	W14x22	–	12
	20	W30x90	1 3/4	66	W33x118	2	54	W14x22	–	12
	21	W30x90	1 3/4	74	W33x110	2 1/4	106	W14x22	–	12
	22	W30x90	1 3/4	76	W33x130	2 1/2	84	W14x22	–	12
63'–0"	23	W30x90	1 3/4	80	W33x130	2 1/2	130	W14x22	–	12
	24	W30x90	2	90	W36x135	2 1/4	110	W14x22	–	12
	25	W30x90	2	90	W36x150	2 1/4	70	W14x26	–	8
	26	W30x99	1 3/4	68	W36x150	2 1/4	120	W14x26	–	8
	27	W30x99	1 3/4	74	W36x160	2 1/4	106	W14x26	–	8
	28	W30x99	2	82	W36x170	2	108	W14x26	–	8
	18	W30x90	2	76	W33x118	1 3/4	38	W14x22	–	12
	19	W30x90	2	76	W33x118	1 3/4	38	W14x22	–	12
	20	W30x90	2	100	W33x118	2 1/4	94	W14x22	–	12
	21	W30x99	2	68	W33x130	2 1/4	82	W14x22	–	12
	22	W30x99	2	70	W36x135	1 3/4	72	W14x22	–	12
65'–0"	23	W30x99	2	74	W36x135	2	104	W14x22	–	12
	24	W30x99	2	88	W36x150	2 1/2	76	W14x22	–	12
	25	W30x99	2	88	W36x150	2 1/2	116	W14x26	–	8
	26	W30x99	2	100	W36x160	2 1/4	120	W14x26	–	8
	27	W30x108	2	68	W36x170	2 1/4	106	W14x26	–	8
	28	W30x108	2	74	W36x182	2	102	W14x26	–	8

NOTES:

1. STEEL SHOWN TO BE ASTM A992 (Fy=50 KSI)
2. C - DENOTES CAMBER.
3. S - DENOTES NUMBER OF STUDS.

NOTES:

A. USE BEAM B AND DEVIDE UP TURN-A-ROUND BAY INTO 2-SPANS FOR 45' BAY + SINGLE SPAN 24' BAY.

Table 3-6 Typical Beam Sizes for CIP Post-Tensioned Deck (Continued)

BAY GEOMETRY	BEAM D			BEAM E			BEAM F			BEAM G		
DIMENSION 2	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S
18	W21x44	1	46	W16x26	1/2	10	W24x76	–	–	W36x150	–	–
19	W21x50	1	32	W16x26	1/2	10	W24x76	–	–	W36x150	–	–
20	W21x50	1	50	W16x26	1/2	10	W24x76	–	–	W36x160	–	–
21	W21x55	1	28	W16x26	1/2	10	W24x76	–	–	W36x160	–	–
22	W21x55	1	40	W16x26	1/2	10	W24x84	–	–	W36x170	–	–
23	W21x55	1	62	W16x26	1/2	10	W24x84	–	–	W36x170	–	–
24	W21x62	1	30	W16x26	1/2	10	W24x84	–	–	W36x170	–	–
25	W21x62	1	70	W16x26	1/2	10	W24x84	–	–	W36x182	–	–
26	W21x68	1	40	W16x26	1/2	10	W24x84	–	–	W36x182	–	–
27	W21x68	1	54	W16x26	1/2	10	W24x84	–	–	W36x182	–	–
28	W21x76	1	40	W16x26	1/2	10	W24x84	–	–	W36x182	–	–
18	W21x50	3/4	30	W16x26	1/2	10	W24x76	–	–	W36x160	–	–
19	W21x50	3/4	30	W16x26	1/2	10	W24x76	–	–	W36x160	–	–
20	W21x55	1	48	W16x26	1/2	10	W24x76	–	–	W36x160	–	–
21	W21x62	1	40	W16x26	1/2	10	W24x84	–	–	W36x170	–	–
22	W21x62	1	62	W16x26	1/2	10	W24x84	–	–	W36x170	–	–
23	W21x68	1	56	W16x26	1/2	10	W24x84	–	–	W36x194	–	–
24	W21x68	1	62	W16x26	1/2	10	W24x84	–	–	W36x194	–	–
25	W21x76	1	38	W16x26	1/2	10	W24x84	–	–	W36x194	–	–
26	W21x76	1	50	W16x26	1/2	10	W24x84	–	–	W36x194	–	–
27	W21x84	1	34	W16x26	1/2	10	W24x84	–	–	W36x194	–	–
28	W21x84	1	36	W16x26	1/2	10	W24x84	–	–	W36x194	–	–
18	W24x55	3/4	42	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
19	W24x55	3/4	42	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
20	W24x62	3/4	64	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
21	W24x68	3/4	38	W16x26	1/2	10	W27x84	–	–	W36x170	–	–
22	W24x68	1	54	W16x26	1/2	10	W27x84	–	–	W36x182	–	–
23	W24x76	1	28	W16x26	1/2	10	W27x84	–	–	W36x182	–	–
24	W27x84	1	28	W16x26	1/2	10	W27x84	–	–	W36x182	–	–
25	W27x84	1	28	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
26	W27x84	1	38	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
27	W21x84	1	54	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
28	W30x90	3/4	70	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
18	W24x55	3/4	72	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
19	W24x55	3/4	72	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
20	W24x68	3/4	62	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
21	W24x68	3/4	74	W16x26	1/2	10	W27x84	–	–	W36x182	–	–
22	W24x76	1	54	W16x26	1/2	10	W27x84	–	–	W36x182	–	–
23	W24x76	1 1/2	54	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
24	W27x84	1 1/2	34	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
25	W27x84	1 1/2	52	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
26	W27x84	1 1/2	72	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
27	W30x90	1	46	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
28	W30x99	1	76	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
18	W24x62	3/4	72	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
19	W24x62	3/4	72	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
20	W24x76	3/4	60	W16x26	1/2	10	W24x76	–	–	W36x182	–	–
21	W24x76	1	70	W16x26	1/2	10	W27x84	–	–	W36x182	–	–
22	W24x76	1	94	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
23	W27x84	3/4	30	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
24	W27x84	1	34	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
25	W27x84	1 1/2	68	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
26	W27x84	1 1/2	92	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
27	W30x90	1	58	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
28	W30x99	1	72	W16x26	1/2	10	W30x90	–	–	W36x232	–	–
18	W24x68	3/4	72	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
19	W24x68	3/4	72	W16x26	1/2	10	W24x76	–	–	W36x170	–	–
20	W27x84	3/4	28	W16x26	1/2	10	W27x84	–	–	W36x182	–	–
21	W27x84	1	54	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
22	W27x84	1	60	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
23	W30x90	1	52	W16x26	1/2	10	W27x84	–	–	W36x194	–	–
24	W30x90	1	74	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
25	W30x90	1	102	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
26	W30x99	1	62	W16x26	1/2	10	W27x84	–	–	W36x210	–	–
27	W30x108	1	46	W16x26	1/2	10	W30x90	–	–	W36x232	–	–
28	W30x108	1	70	W16x26	1/2	10	W30x90	–	–	W36x232	–	–

NOTES:

1. STEEL SHOWN TO BE ASTM A992 (Fy=50 KSI)
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3. S - DENOTES NUMBER OF STUDS.

NOTES:

A. USE BEAM B AND DEVIDE UP TURN-A-ROUND BAY INTO 2-SPANS FOR 45’ BAY + SINGLE SPAN 24’ BAY.

Table 3-7 Typical Beam Sizes for Cast in Place Slab Poured on Filigree Deck

BAY GEOMETRY		BEAM A			BEAM B			BEAM C			BEAM D			BEAM E			BEAM F			BEAM G			BEAM H		
DIMENSION ①	DIMENSION ②	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S	SIZE	C	S
55'–0"	18	W24x84	1 1/2"	48	W24x68	1 1/2"	40	W24x55	0"	32	W18x35	0"	20	W12x19	0"	12	W24x55	0"	32	W36x150	0"	84	W27x84	3/4"	44
	19	W27x94	1"	54	W24x76	1 1/2"	44	W24x55	0"	32	W18x35	0"	20	W12x19	0"	12	W24x55	0"	32	W36x150	0"	84	W27x94	3/4"	46
	20	w27x94	1"	54	W24x76	1 1/2"	44	W24x55	0"	32	W18x35	0"	20	W12x19	0"	12	W24x55	0"	32	W36x150	0"	84	W27x94	3/4"	48
	21	W30x108	1"	60	W24x76	1 1/2"	44	W24x55	0"	32	W18x35	0"	20	W12x22	0"	14	W24x55	0"	32	W40x149	0"	90	W30x99	3/4"	52
	22	W30x108	1"	60	W24x76	1 1/2"	44	W24x55	0"	32	W18x35	0"	20	W12x22	0"	14	W21x68	0"	38	W40x149	0"	90	W30x99	3/4"	52
57'–0"	18	W27x94	1"	54	W24x84	1 1/4"	48	W24x62	0"	36	W18x35	0"	20	W12x19	0"	12	W24x55	0"	32	W36x160	0"	90	W27x94	0"	54
	19	W30x99	1"	56	W24x84	1 1/4"	48	W24x62	0"	36	W18x35	0"	20	W12x19	0"	12	W24x62	0"	36	W36x160	0"	90	W27x94	1"	54
	20	W30x108	1"	60	W27x84	1 1/4"	48	W24x62	0"	36	W18x40	0"	24	W12x19	0"	12	W21x68	0"	38	W36x160	0"	90	W30x99	0"	54
	21	W30x116	1 1/4"	66	W27x94	1 1/4"	54	W24x68	0"	38	W18x40	0"	24	W12x22	0"	14	W24x68	0"	38	W40x167	0"	90	W30x99	0"	54
	22	W30x116	1 1/4"	66	W27x94	1 1/4"	54	W24x68	0"	38	W18x40	0"	24	W12x22	0"	14	W24x68	0"	38	W40x167	0"	90	W30x99	0"	54
59'–0"	18	W30x99	1"	56	W24x84	1 1/4"	48	W24x62	0"	36	W18x40	0"	24	W12x19	0"	12	W24x62	0"	36	W36x160	0"	90	W30x99	0"	54
	19	W30x108	1"	60	W27x84	1 1/4"	48	W24x62	0"	36	W18x40	0"	24	W12x19	0"	12	W24x62	0"	36	W40x167	0"	90	W30x99	0"	54
	20	W30x108	1 1/4"	60	W27x84	1 1/4"	48	W24x68	0"	38	W18x40	0"	24	W12x19	0"	12	W24x76	0"	44	W40x167	0"	90	W30x99	3/4"	54
	21	W33x118	1"	66	W27x84	1 1/4"	48	W24x68	0"	38	W18x40	0"	24	W12x22	0"	14	W24x76	0"	44	W40x167	0"	90	W30x99	3/4"	54
	22	W33x118	1"	66	W27x84	1 1/4"	48	W24x68	0"	38	W18x40	0"	24	W12x22	0"	14	W24x76	0"	44	W40x167	0"	90	W30x99	3/4"	54
61'–0"	18	W30x99	1 1/4"	56	W24x94	1 1/2"	54	W24x68	0"	38	W18x40	0"	24	W12x19	0"	12	W24x62	0"	36	W40x167	0"	90	W30x99	3/4"	54
	19	W30x108	1 1/4"	60	W27x94	1 1/4"	54	W24x68	0"	38	W18x40	0"	24	W12x19	0"	12	W24x76	0"	44	W40x167	0"	90	W30x99	3/4"	54
	20	W30x116	1 1/4"	66	W27x94	1 1/4"	54	W24x68	0"	38	W21x44	0"	26	W12x19	0"	12	W24x76	0"	44	W40x167	0"	90	W30x99	3/4"	54
	21	W33x118	1 1/4"	66	W27x94	1 1/2"	54	W24x68	0"	38	W21x44	0"	26	W12x22	0"	14	W27x84	0"	48	W40x167	0"	90	W30x108	3/4"	54
	22	W33x130	1"	72	W27x94	1 1/2"	54	W24x68	0"	38	W21x44	0"	26	W12x22	0"	14	W27x84	0"	48	W40x167	0"	90	W30x108	3/4"	54
63'–0"	18	W30x108	1 1/4"	60	W27x94	1 1/2"	54	W24x68	0"	38	W21x44	0"	26	W12x19	0"	12	W24x68	3/4"	38	W40x167	0"	90	W30x99	3/4"	54
	19	W33x118	1"	66	W27x94	1 1/2"	54	W24x68	0"	38	W21x44	0"	26	W12x19	0"	12	W24x76	0"	44	W40x167	0"	90	W30x99	3/4"	54
	20	W33x118	1 1/4"	66	W27x94	1 1/2"	54	W24x68	0"	38	W21x44	0"	26	W12x19	0"	12	W24x76	0"	44	W40x167	0"	90	W30x108	0"	54
	21	W33x130	1 1/4"	72	W27x94	1 1/2"	54	W24x68	0"	38	W21x44	0"	26	W12x22	0"	14	W27x84	0"	48	W36x182	0"	90	W30x108	0"	54
	22	W33x130	1 1/4"	72	W27x94	1 1/2"	54	W24x68	0"	38	W21x44	0"	26	W12x22	0"	14	W27x84	0"	48	W36x182	0"	90	W30x108	0"	54
65'–0"	18	W30x108	1 1/2"	60	W30x99	1 1/2"	56	W24x68	0"	38	W21x44	0"	26	W12x19	0"	12	W24x68	0"	38	W40x167	0"	90	W30x99	3/4"	56
	19	W33x118	1 1/4"	66	W30x99	1 1/2"	56	W24x68	0"	38	W21x44	0"	26	W12x19	0"	12	W24x68	0"	38	W40x167	0"	90	W30x99	3/4"	56
	20	W33x118	1 1/4"	66	W30x99	1 1/2"	56	W24x68	0"	38	W21x44	0"	26	W12x19	0"	12	W24x68	0"	38	W40x167	0"	90	W30x108	3/4"	56
	21	W33x130	1 1/4"	72	W30x99	1 1/2"	56	W24x68	0"	38	W21x44	0"	26	W12x22	0"	14	W24x68	0"	38	W36x182	0"	90	W30x108	3/4"	56
	22	W33x130	1 1/4"	72	W30x99	1 1/2"	56	W24x68	0"	38	W21x44	0"	26	W12x22	0"	14	W24x68	0"	38	W36x182	0"	90	W30x108	3/4"	56

NOTES:

1. STEEL SHOWN TO BE ASTM A992 (Fy=50 KSI).
2. C – DENOTES CAMBER.
3. S – DENOTES NUMBER OF STUDS.

Table 3-8 Typical Girder Sizes

BAY GEOMETRY		BEAM A	BEAM B	BEAM C
DIMENSION ①	DIMENSION ②	SIZE	SIZE	SIZE
55'–0"	30'–0"	W36x260	W33x118	W24x76
	36'–0"	W36x260	W33x118	W27x94
57'–0"	30'–0"	W36x260	W33x118	W24x76
	36'–0"	W36x260	W33x118	W27x94
59'–0"	30'–0"	W36x260	W33x130	W24x76
	36'–0"	W36x260	W33x130	W27x94
61'–0"	30'–0"	W36x260	W33x130	W24x76
	36'–0"	W36x260	W33x130	W30x99
63'–0"	30'–0"	W36x260	W33x130	W24x84
	36'–0"	W36x260	W33x130	W30x99
65'–0"	30'–0"	W36x260	W36x135	W24x84
	36'–0"	W36x260	W36x135	W30x99

NOTES:
STEEL IS ASTM A992 (Fy=50 KSI).

Chapter 3
Figures

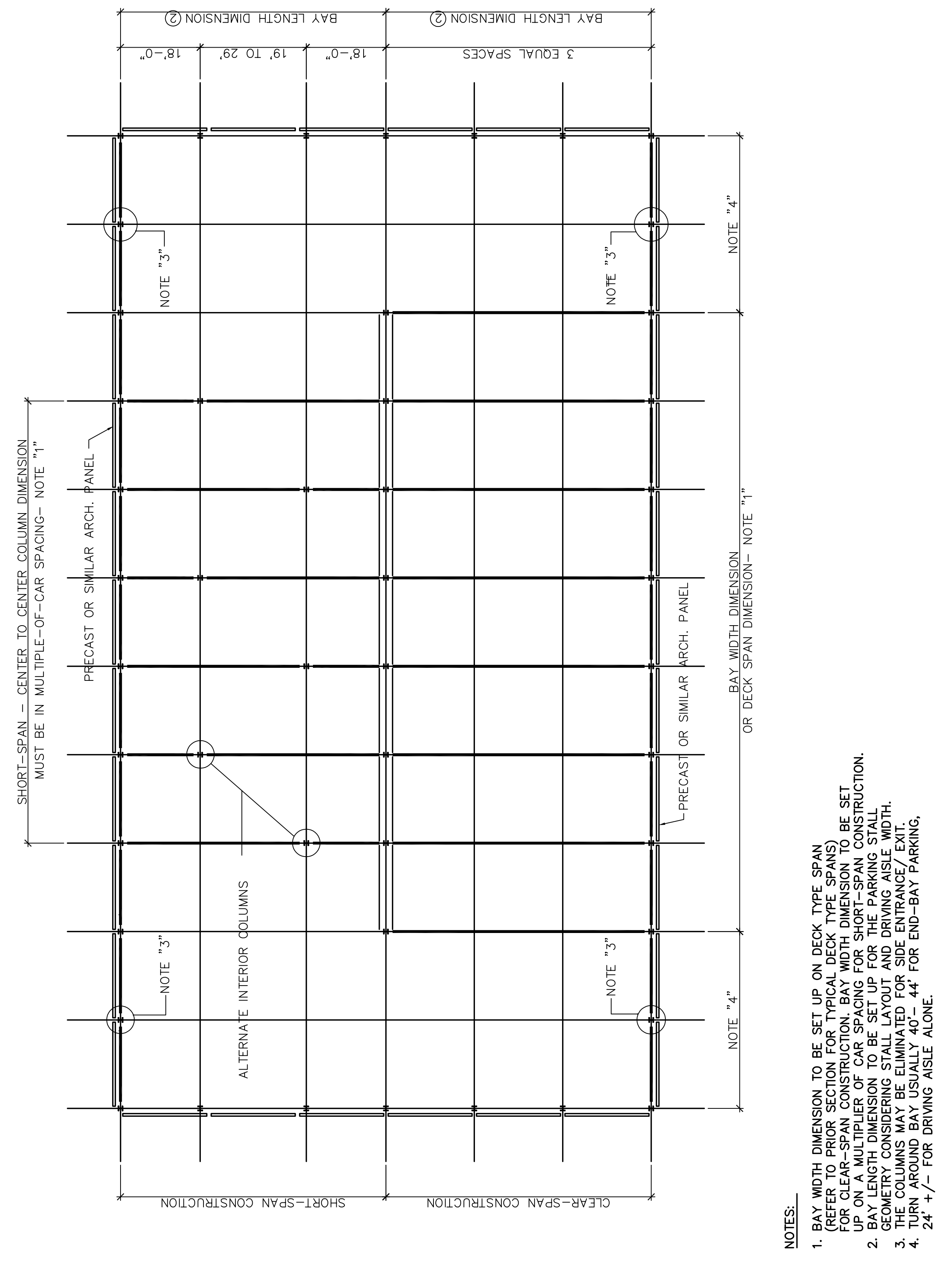


Fig. 3-1. Typical Floor Framing

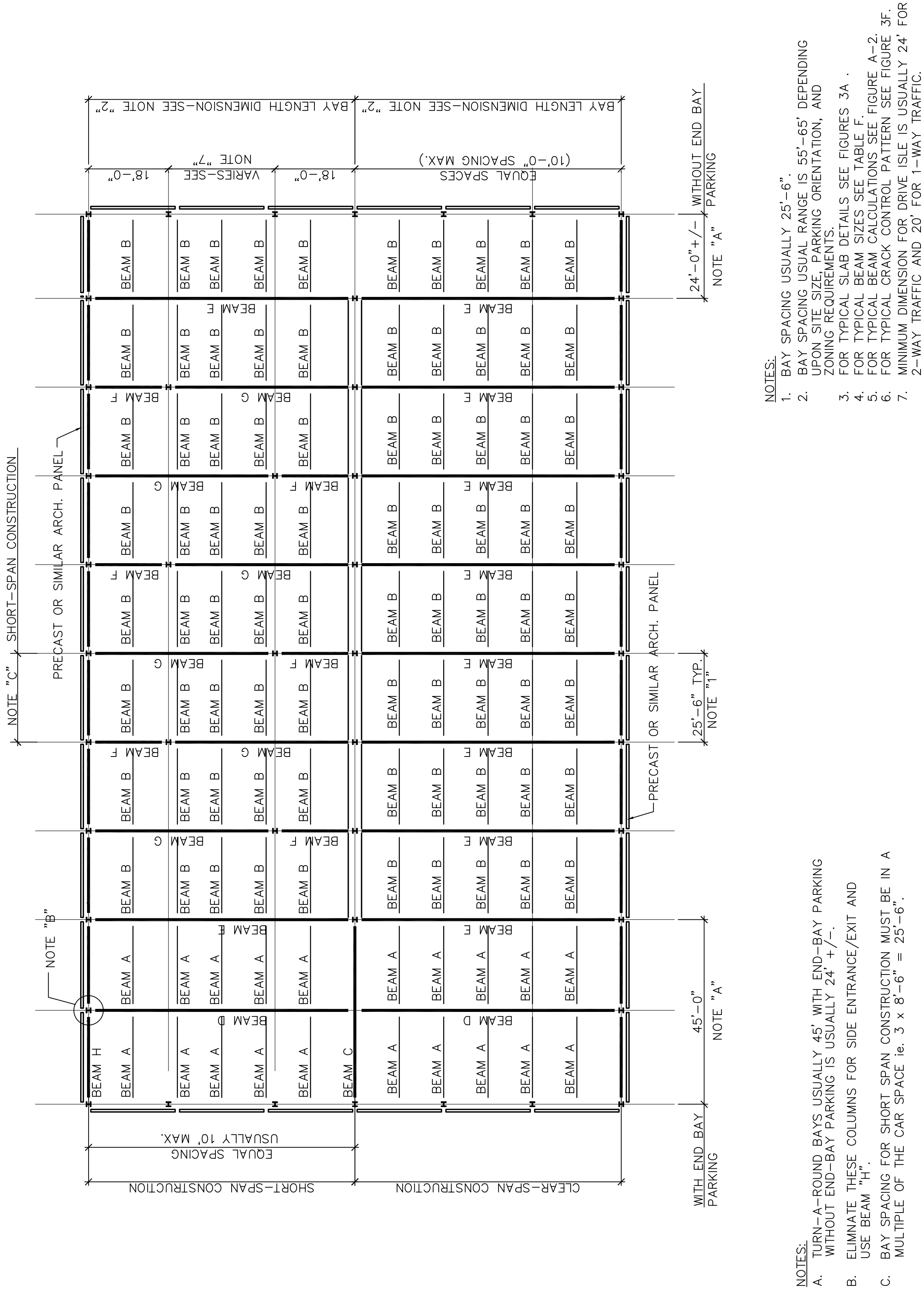


Fig. 3-2. Typical Framing Plan—Cast-in-Place Concrete Using Metal Deck—Configuration 1

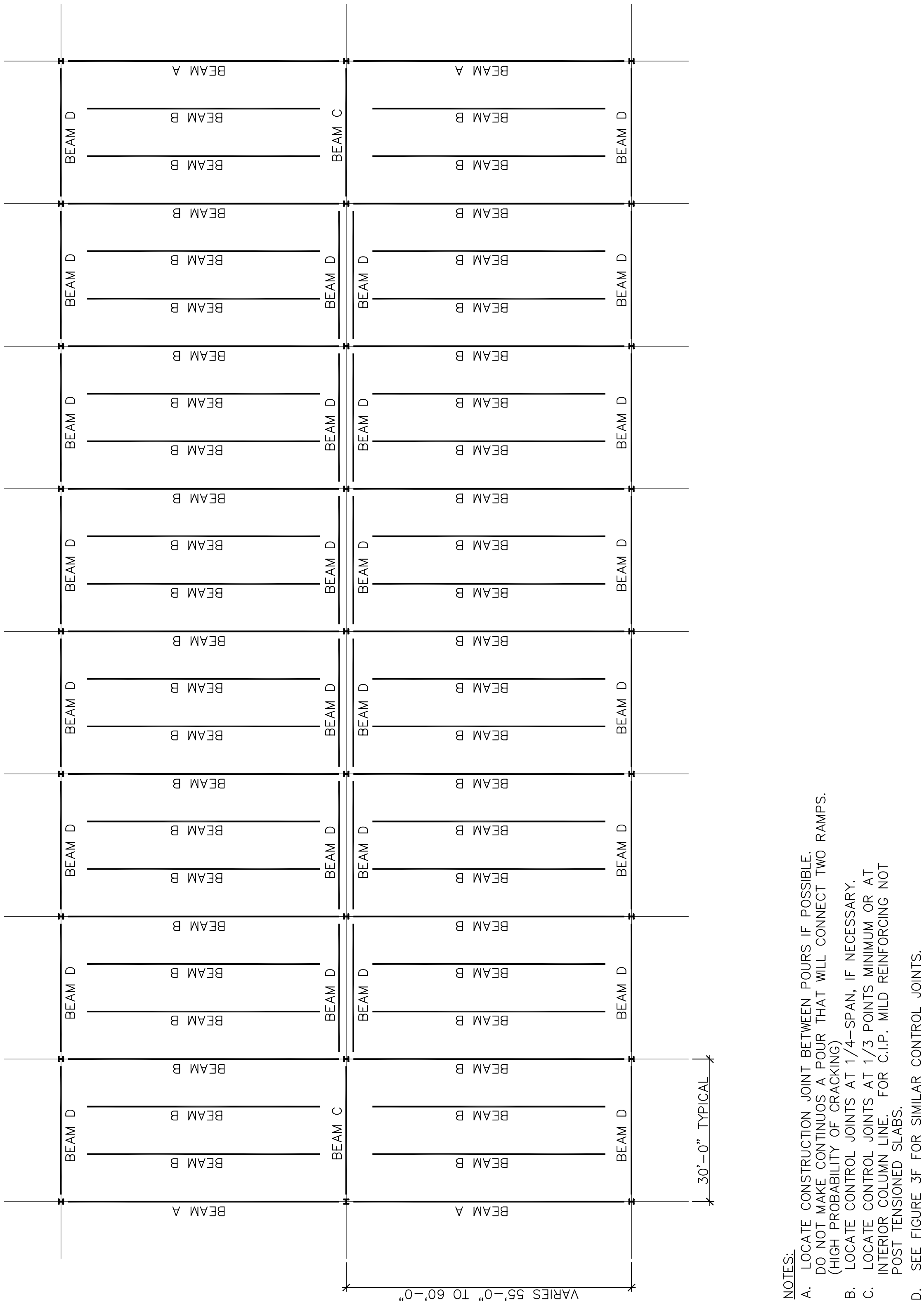


Fig. 3-3. Typical Framing Plan—Cast-in-Place Concrete Using Metal Deck—Configuration 2

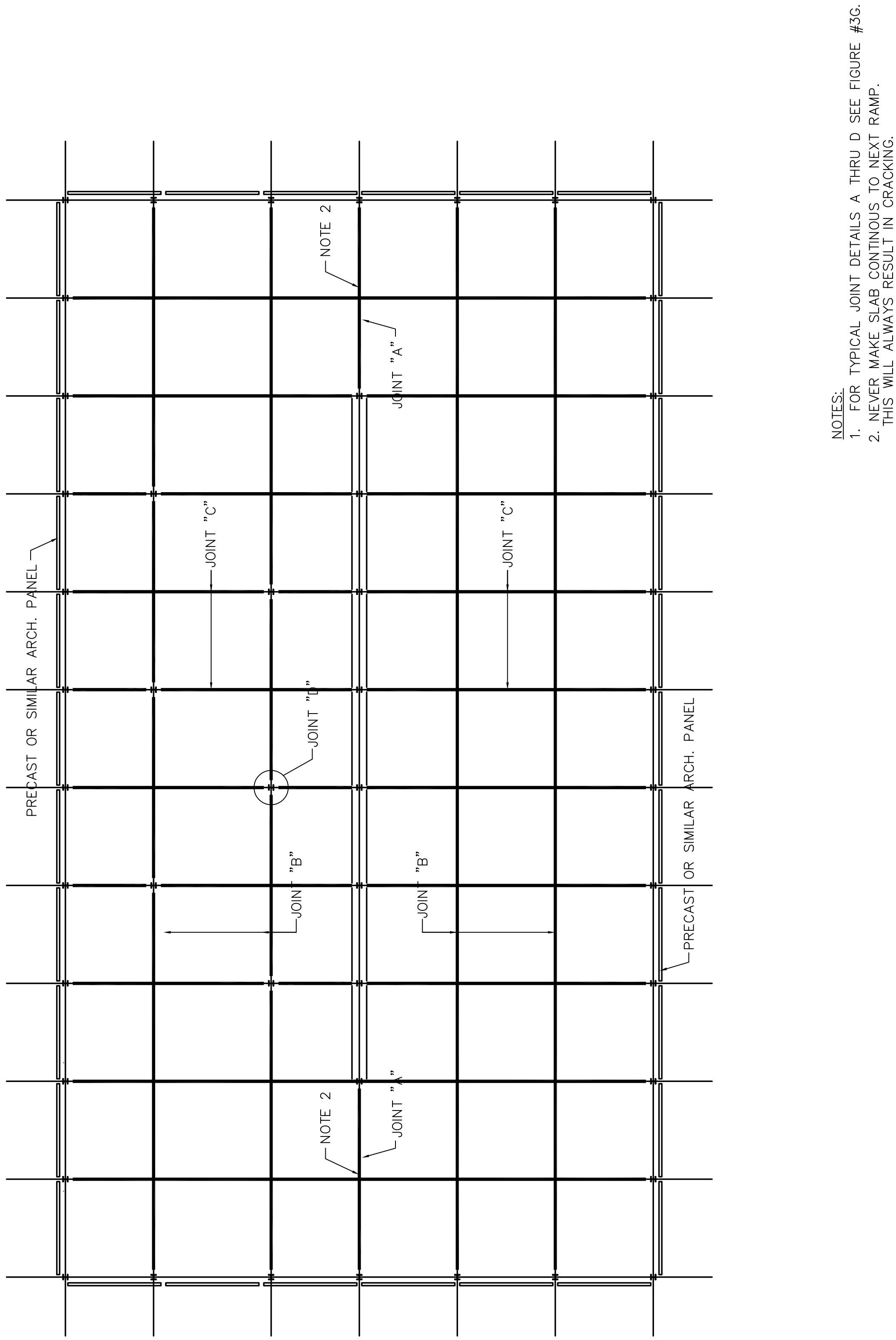


Fig. 3-4. Typical Joint Pattern

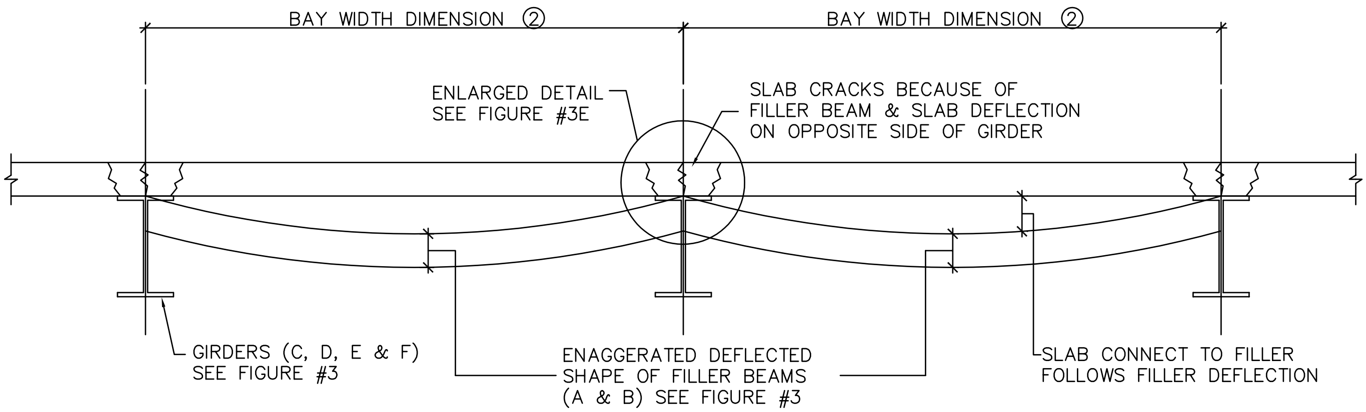


Fig. 3-5. Cross-Section Through Slab at Filler Beam and Girder

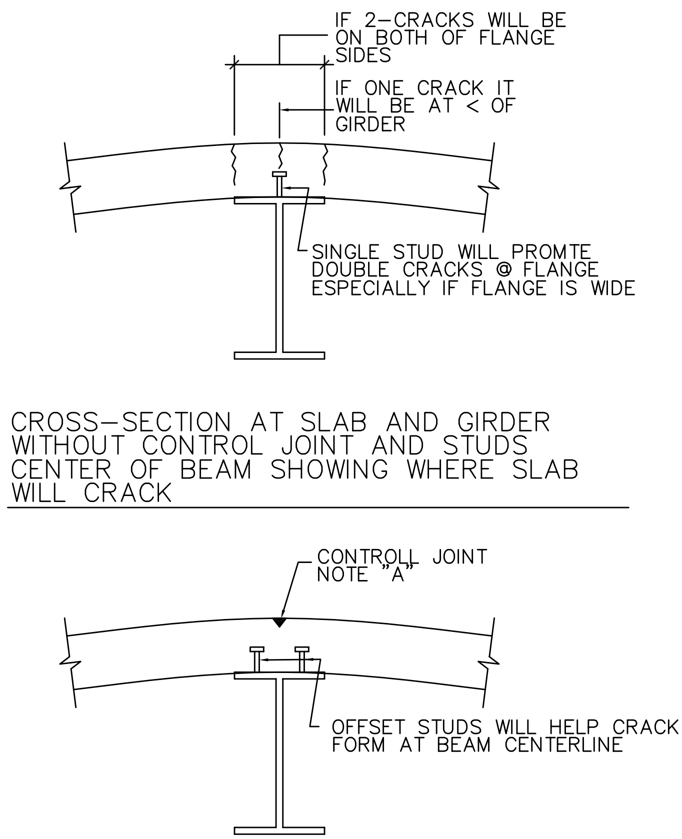


Fig. 3-6. Control Joints and Offset Studs

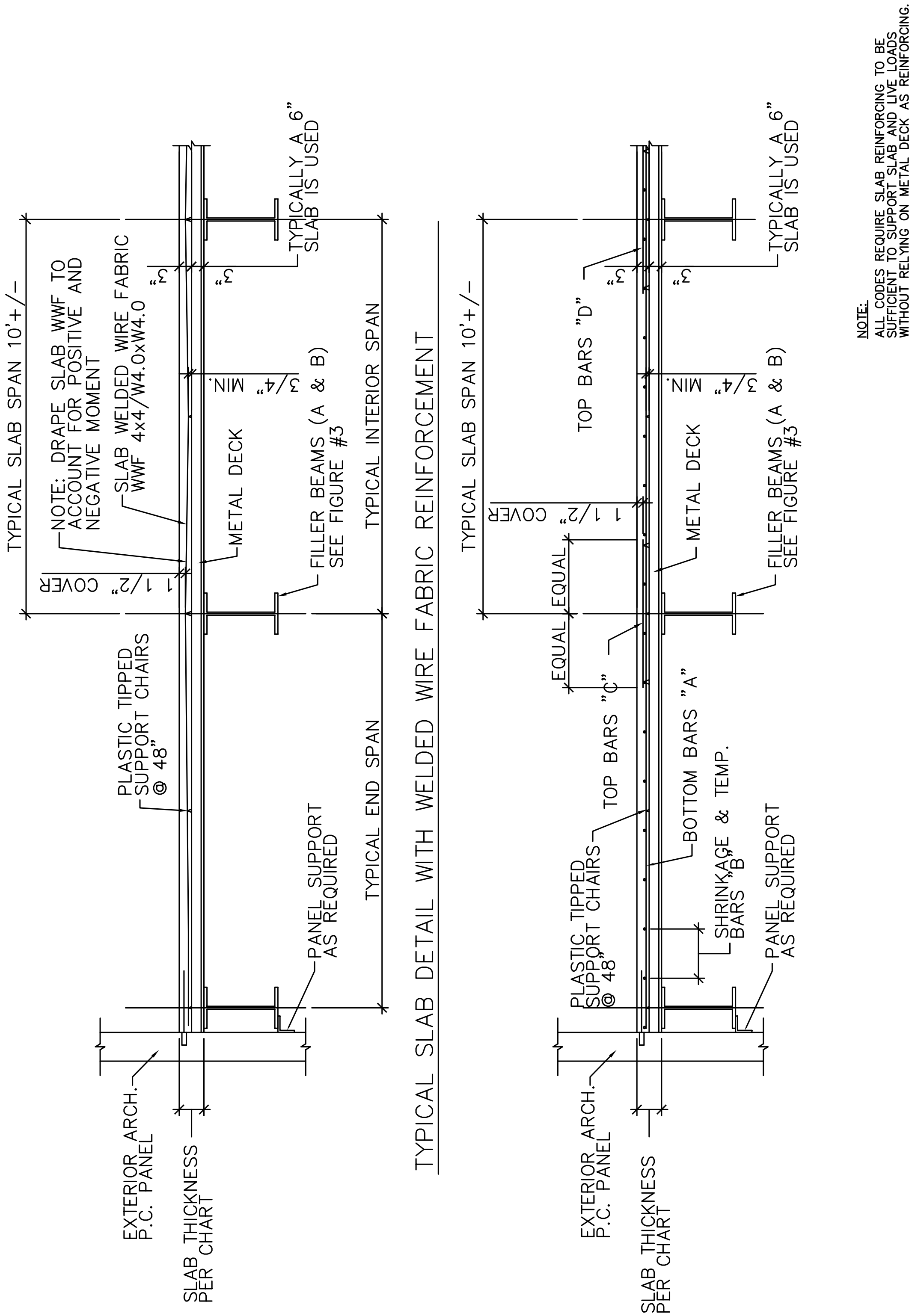
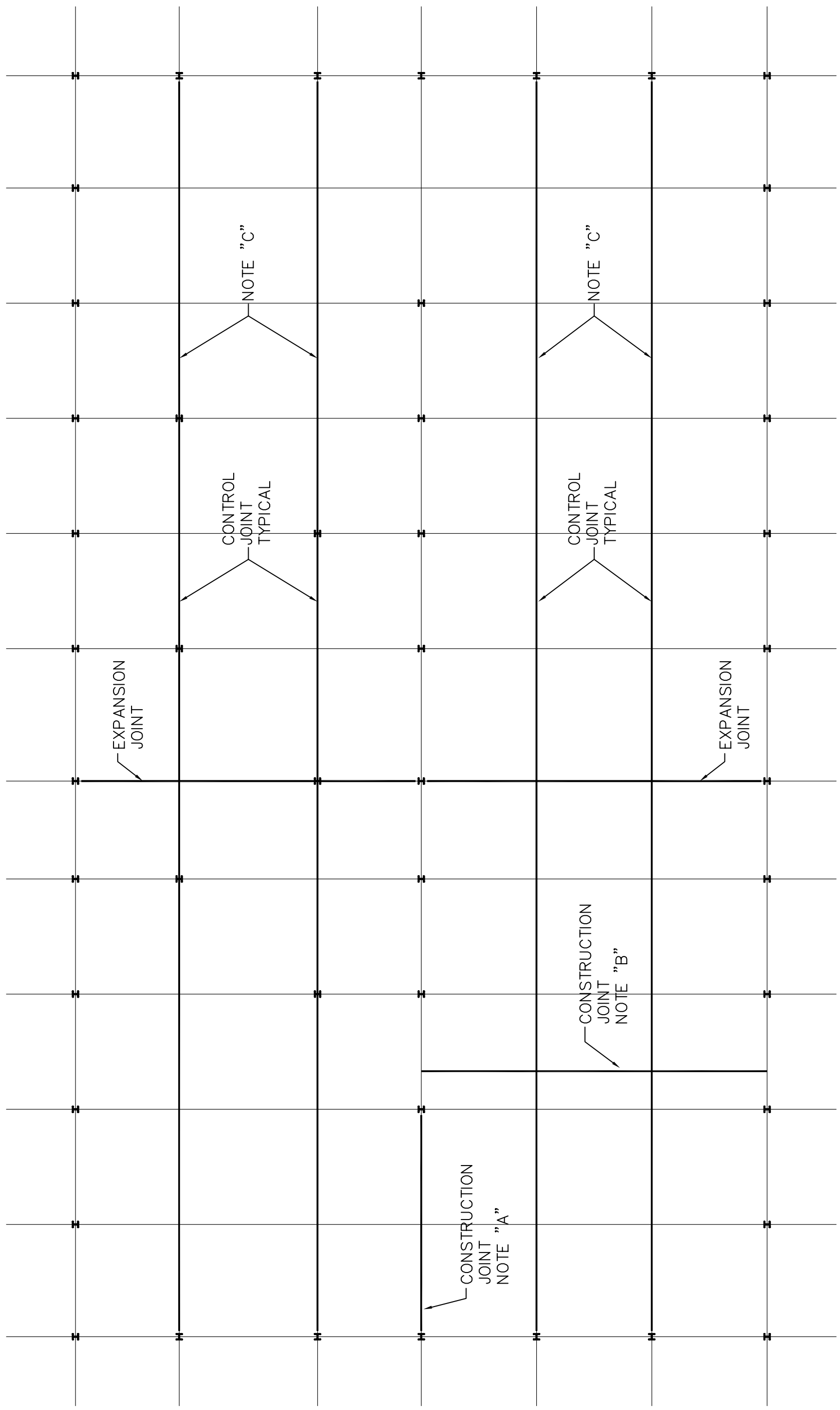


Fig. 3-7. Typical Slab Detail with Reinforcement



NOTES:

A. LOCATE CONSTRUCTION JOINT BETWEEN POURS IF POSSIBLE. DO NOT CONTINUE POUR THAT WILL CONNECT TWO RAMPS. (HIGH PROBABILITY OF CRACKING)

B. LOCATE CONTROL JOINTS AT 1/4–SPAN, IF NECESSARY.

C. LOCATE CONTROL JOINTS AT 1/3 POINTS MINIMUM OR AT INTERIOR COLUMN LINE. FOR C.I.P. MILD REINFORCING NOT POST TENSIONED SLABS.

Fig. 3-8. Typical Joint Detail

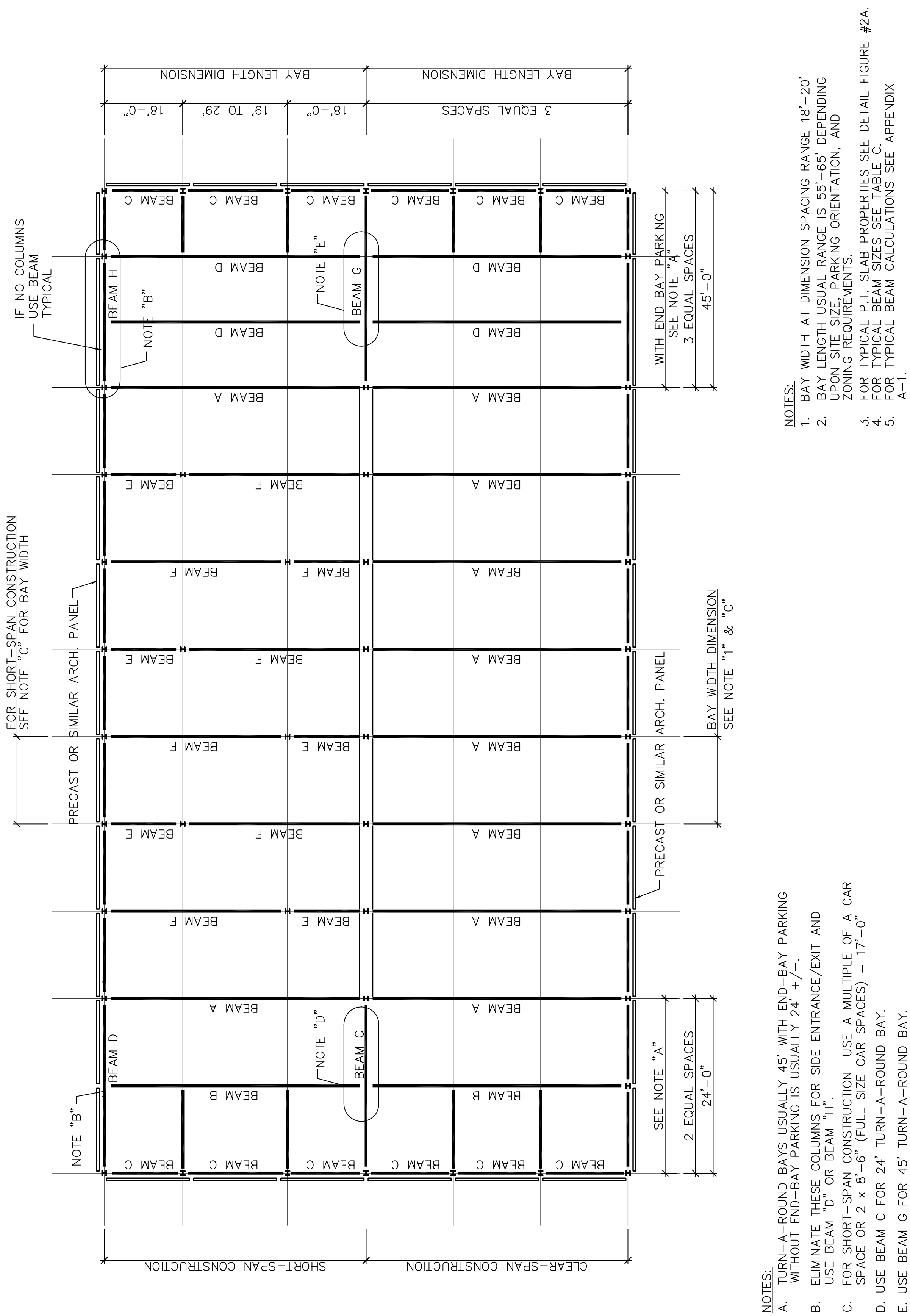


Fig. 3-9. Typical Framing Plan—CIP and PT Slab

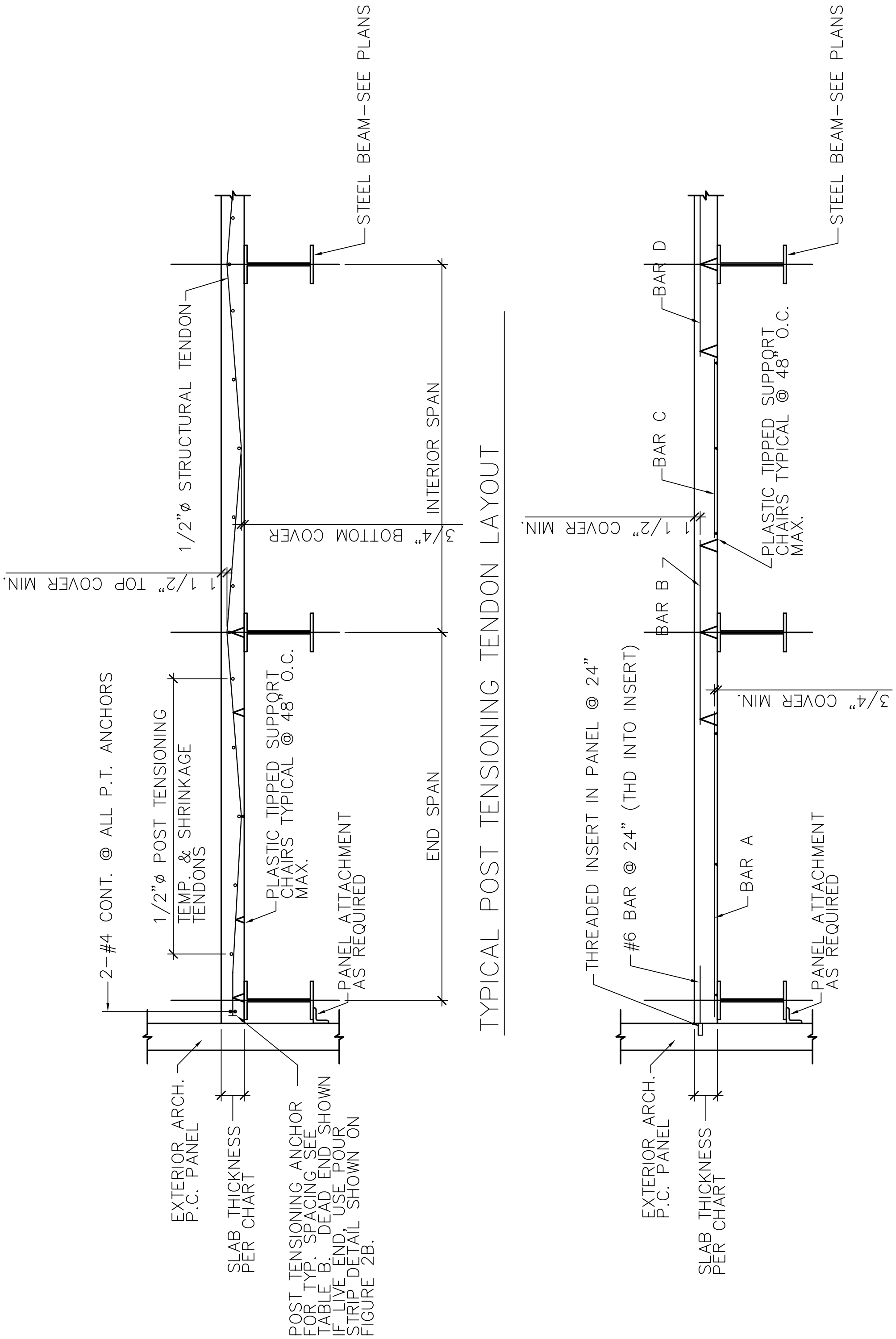


Fig. 3-10. Typical Post-Tensioned Slab Profiles

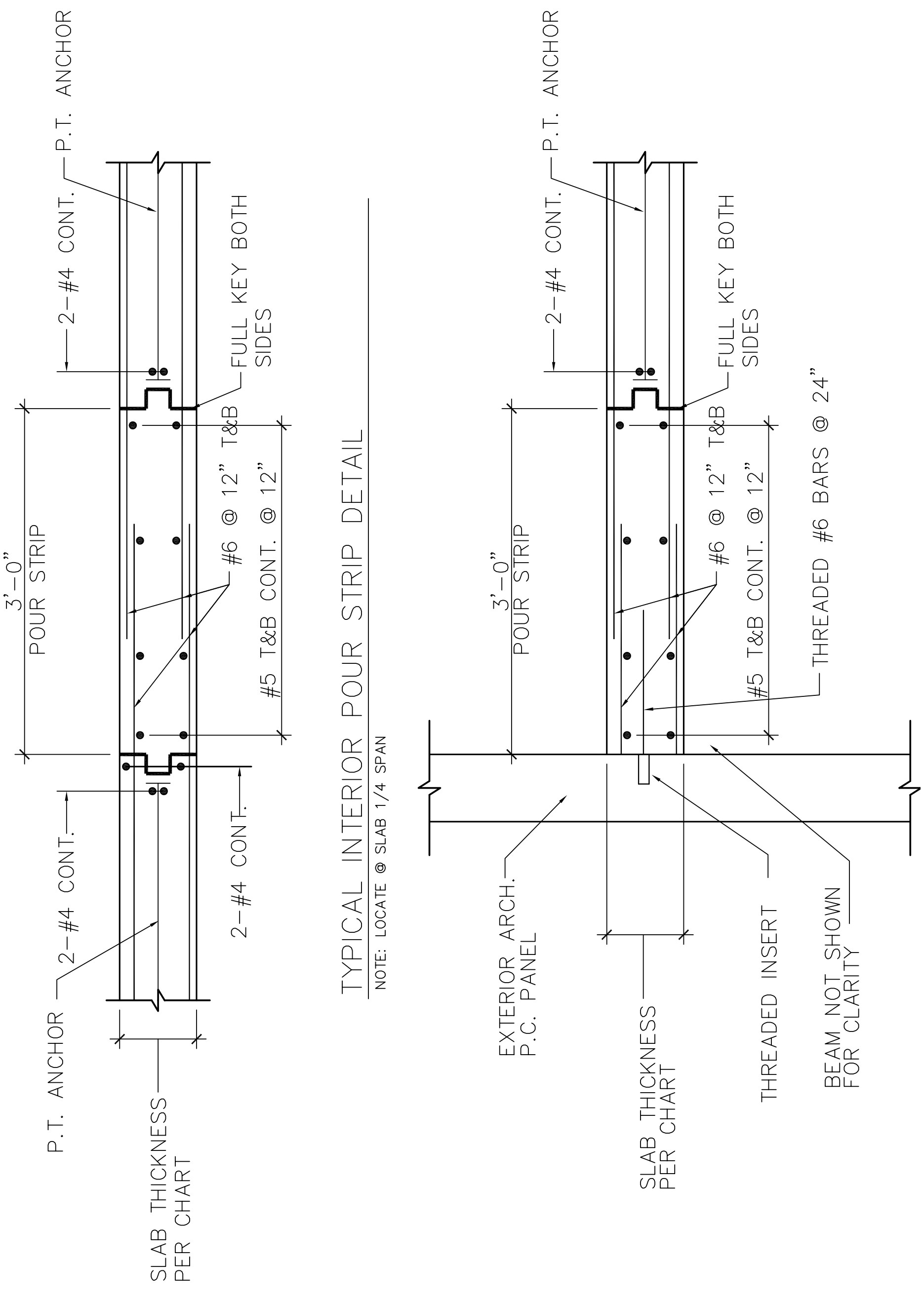


Fig. 3-11. Typical Pour Strip Details

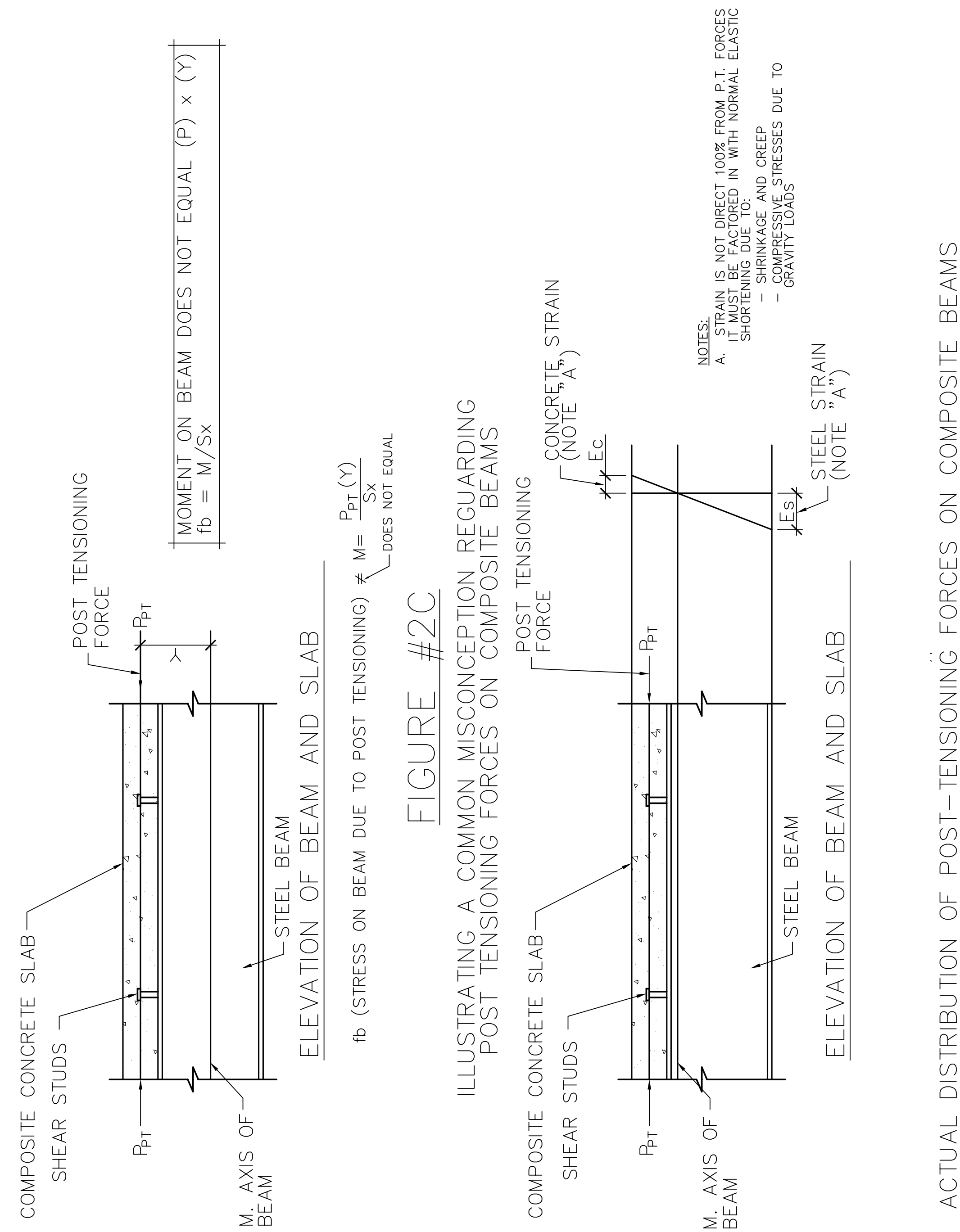


Fig. 3-12. Effect of Post-Tensioning Forces

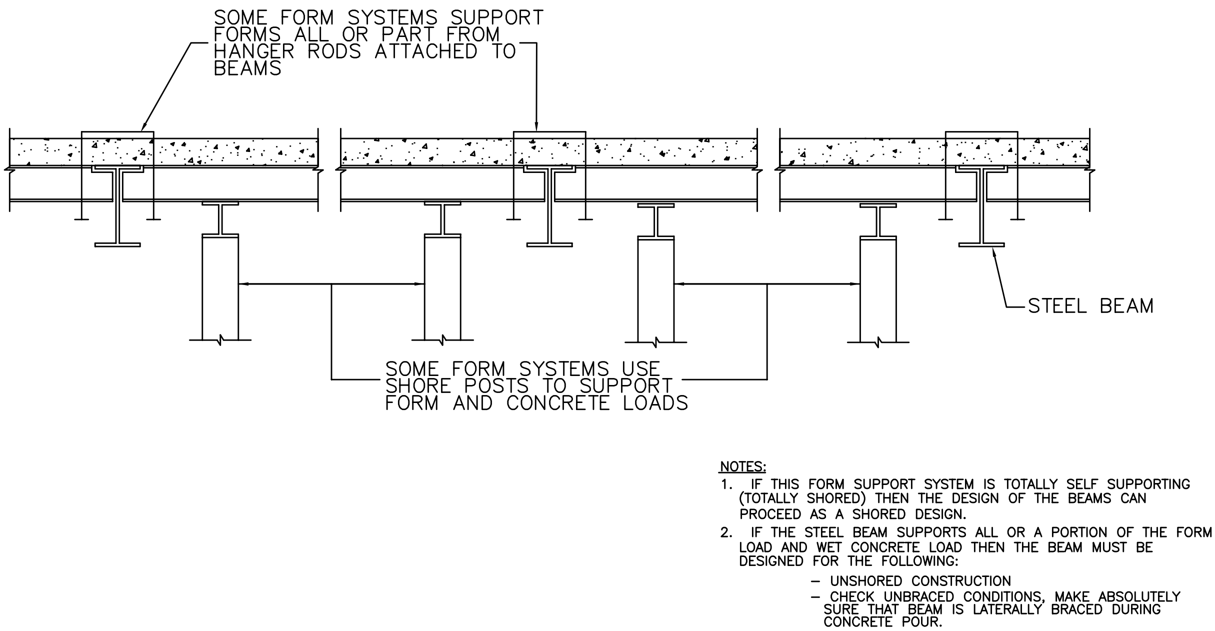


Fig. 3-13. Slab Support During Construction

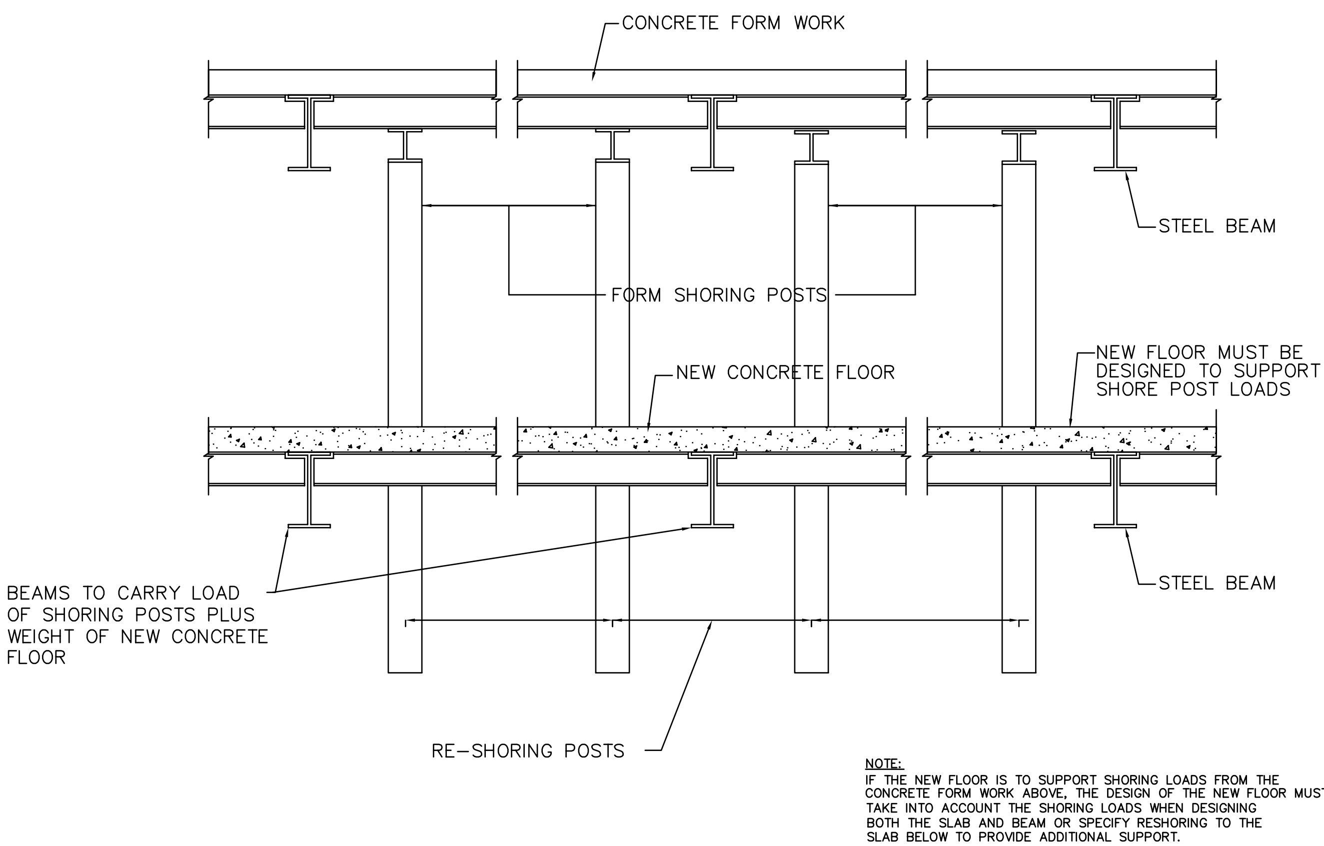


Fig. 3-14. Shoring Between Floors

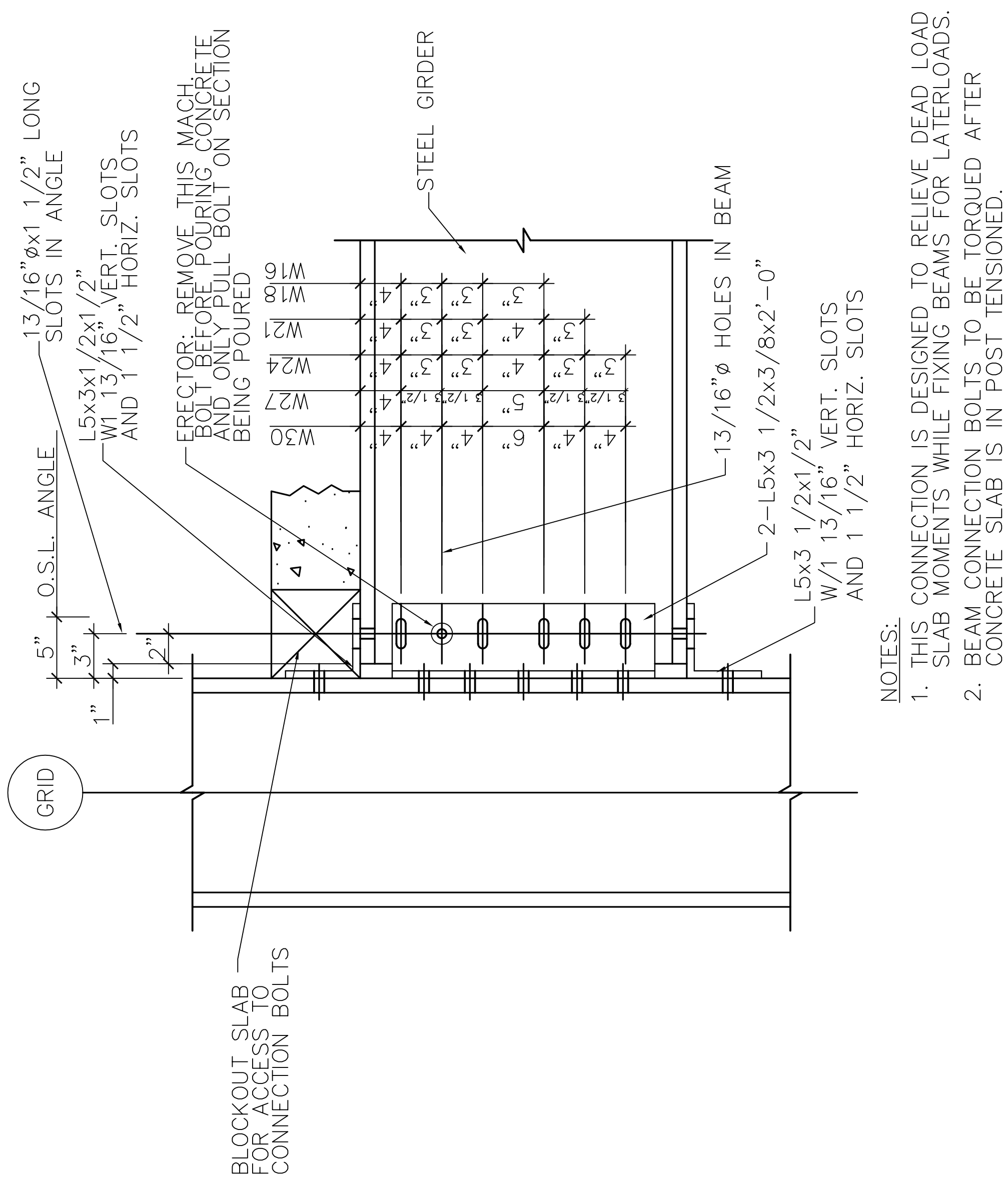


Fig. 3-15. Typical Connection

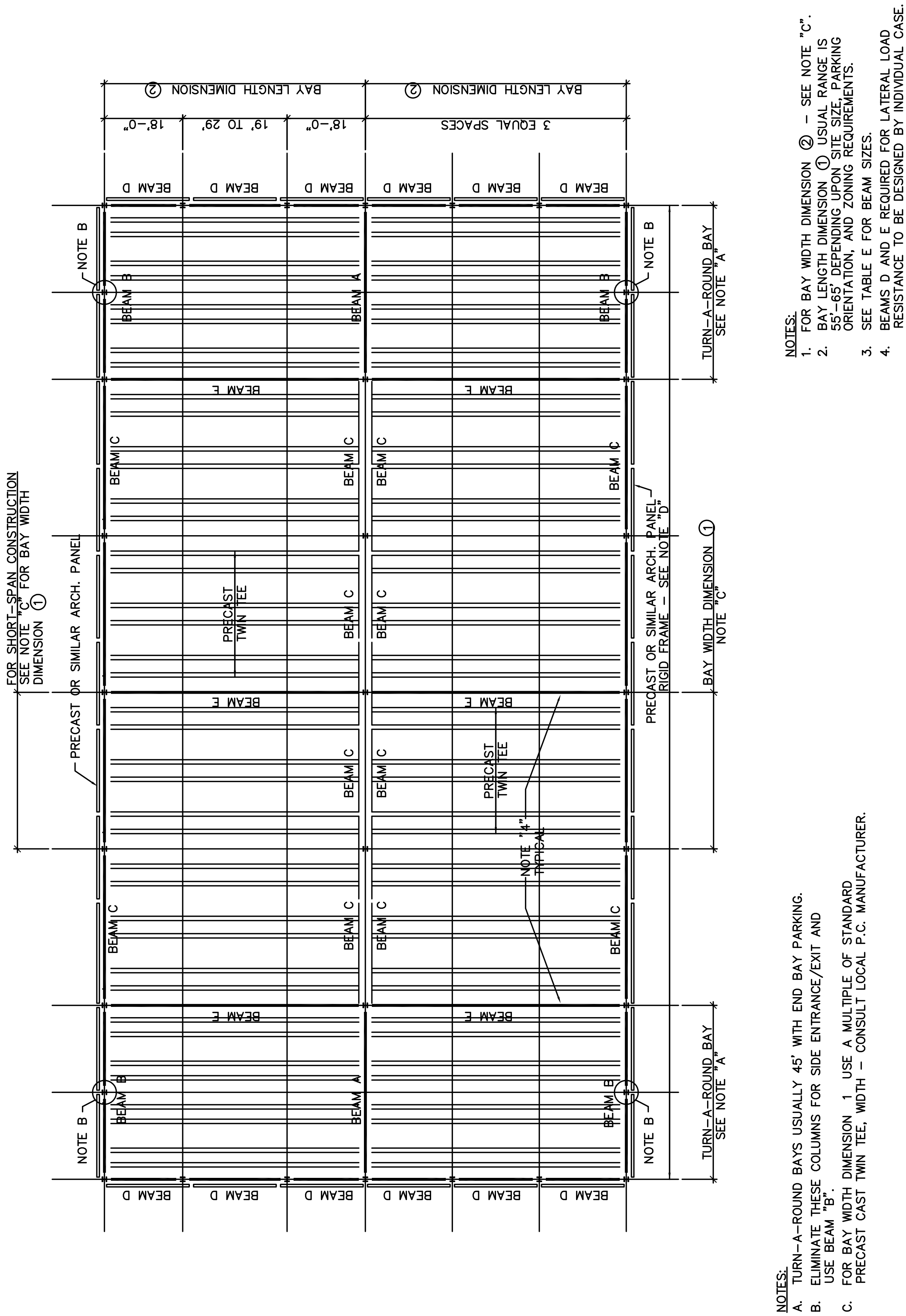


Fig. 3-16. Typical Floor Plan—Double Tee Deck

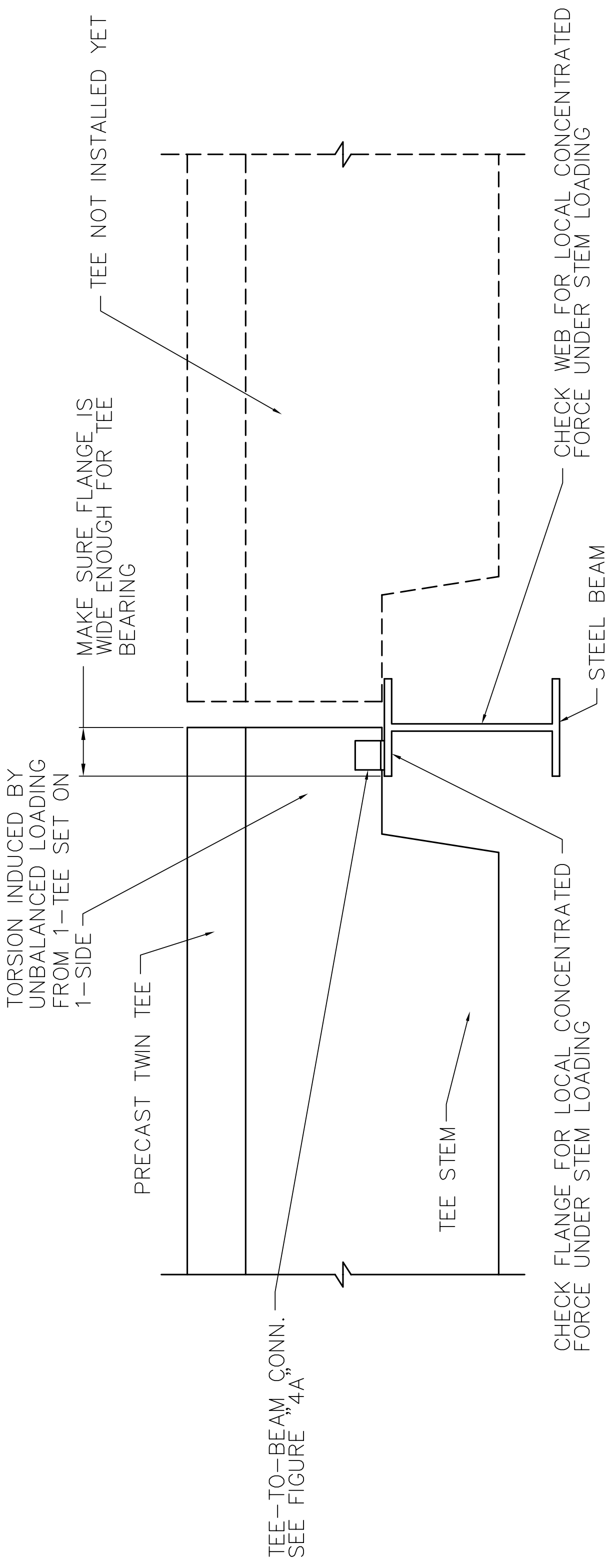


Fig. 3-17. Typical Double Tee on Beam Cross-Section

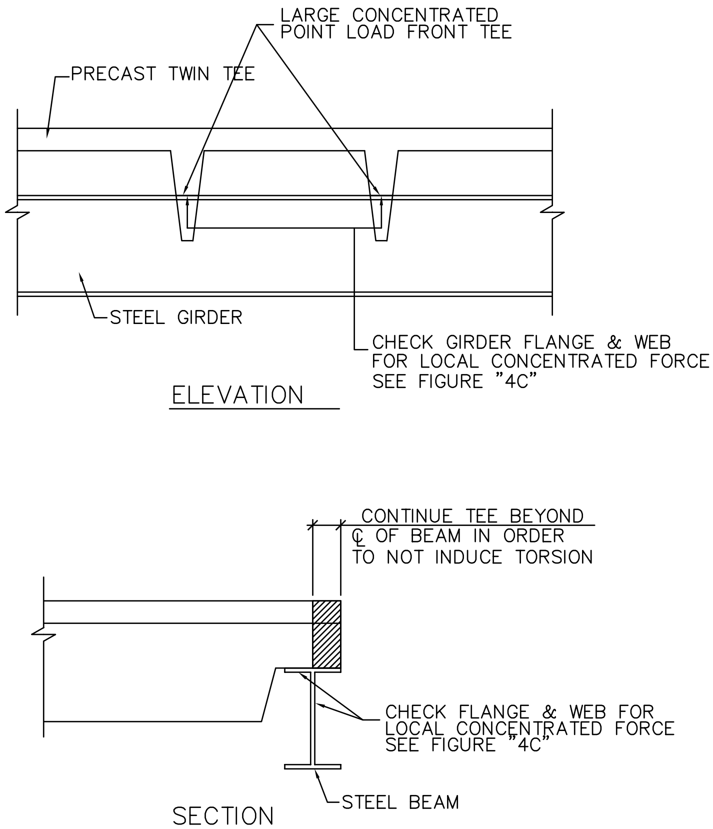


Fig. 3-18. Precast Double Tees on Girder

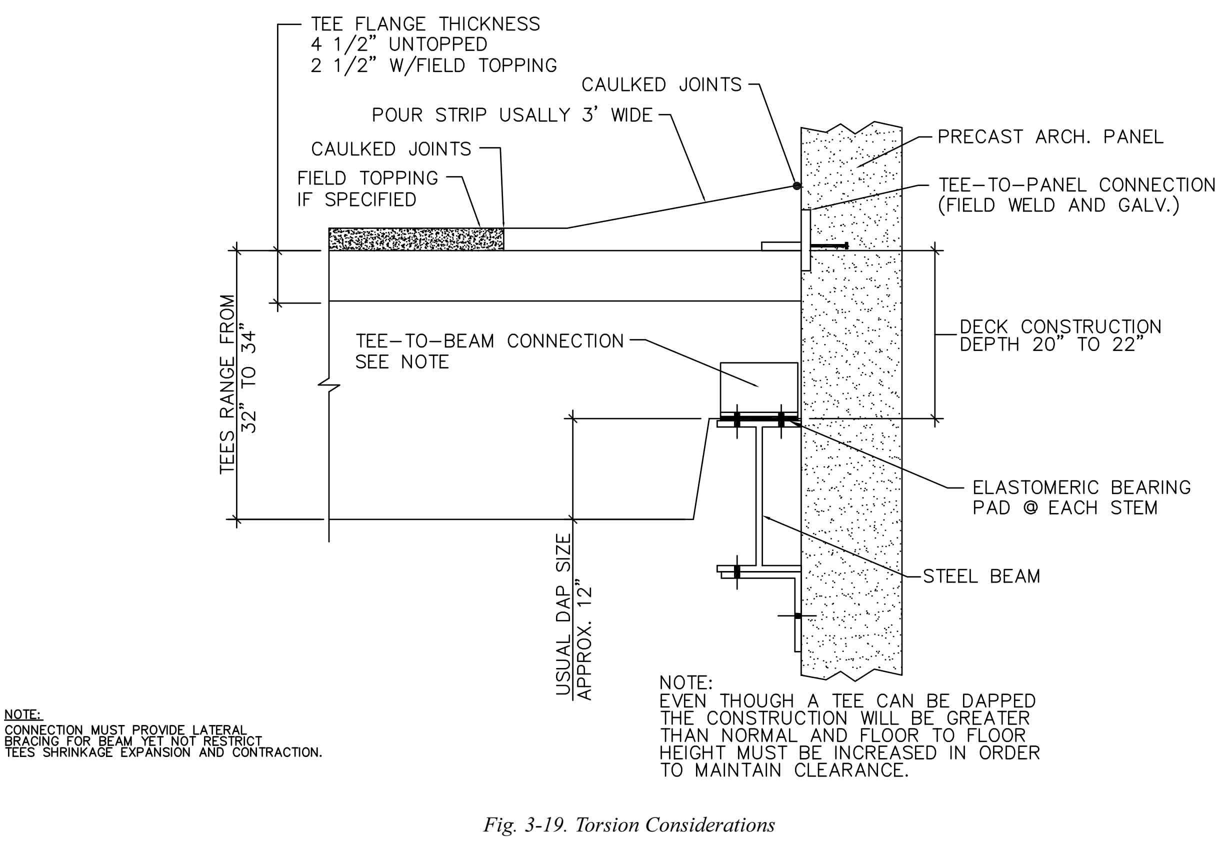


Fig. 3-19. Torsion Considerations

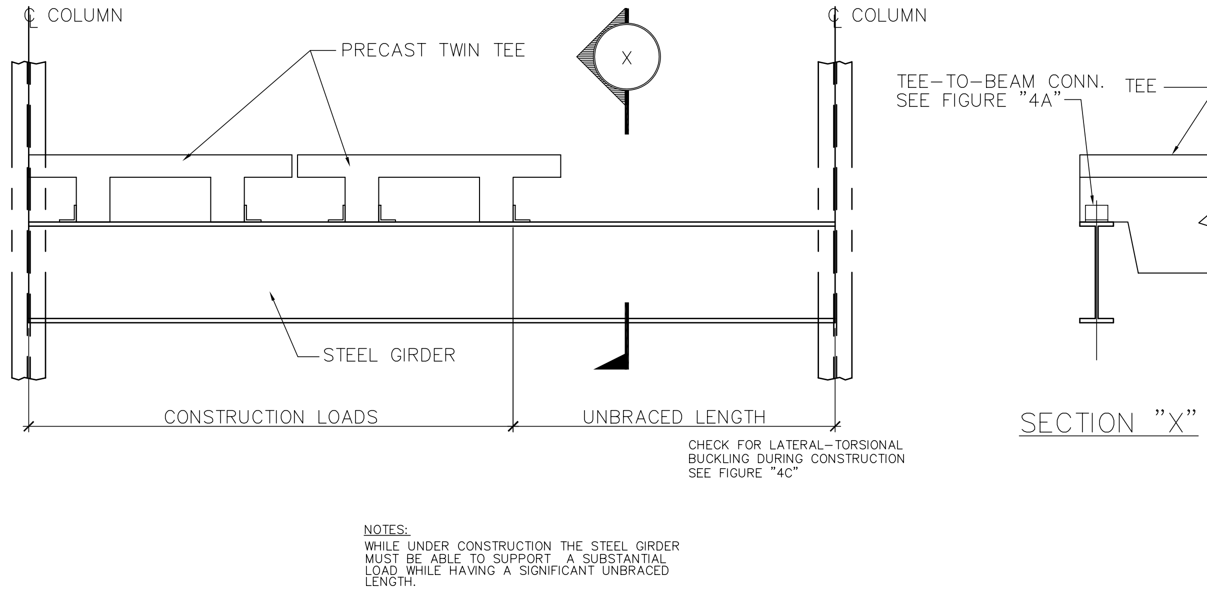


Fig. 3-20. Torsion Considerations During Construction

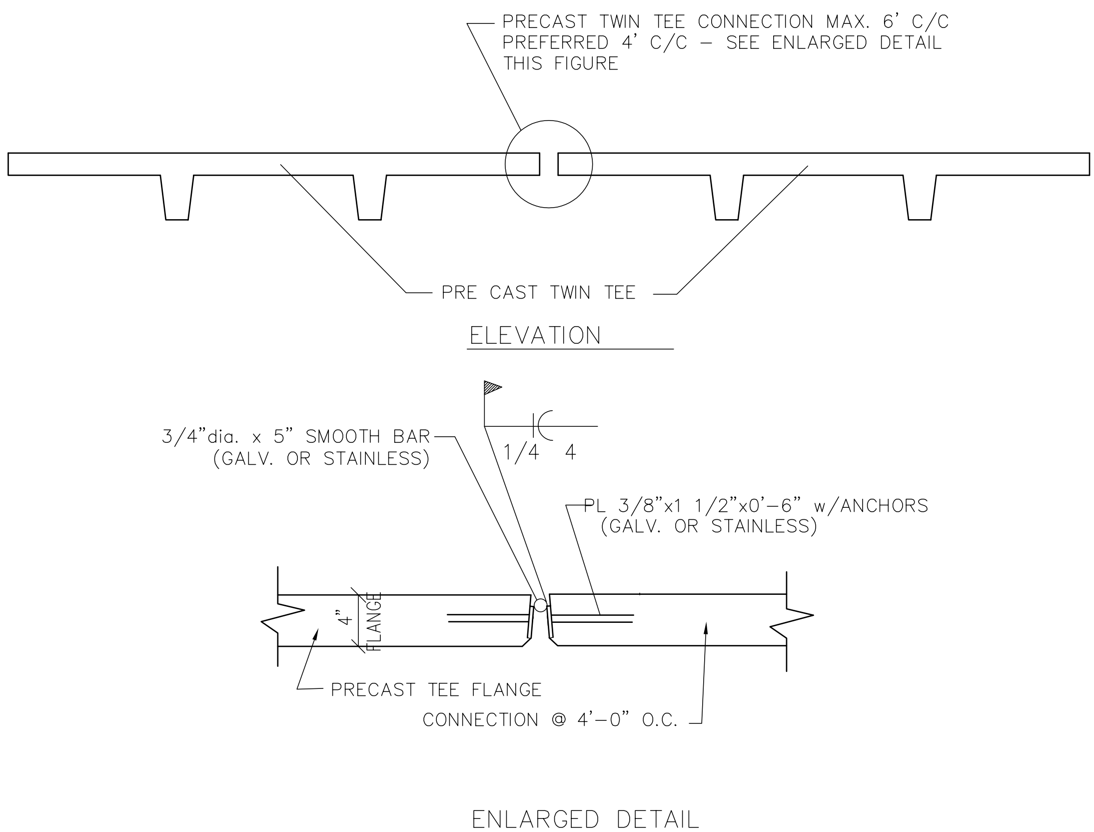


Fig. 3-21. Tee-to-Tee Connections

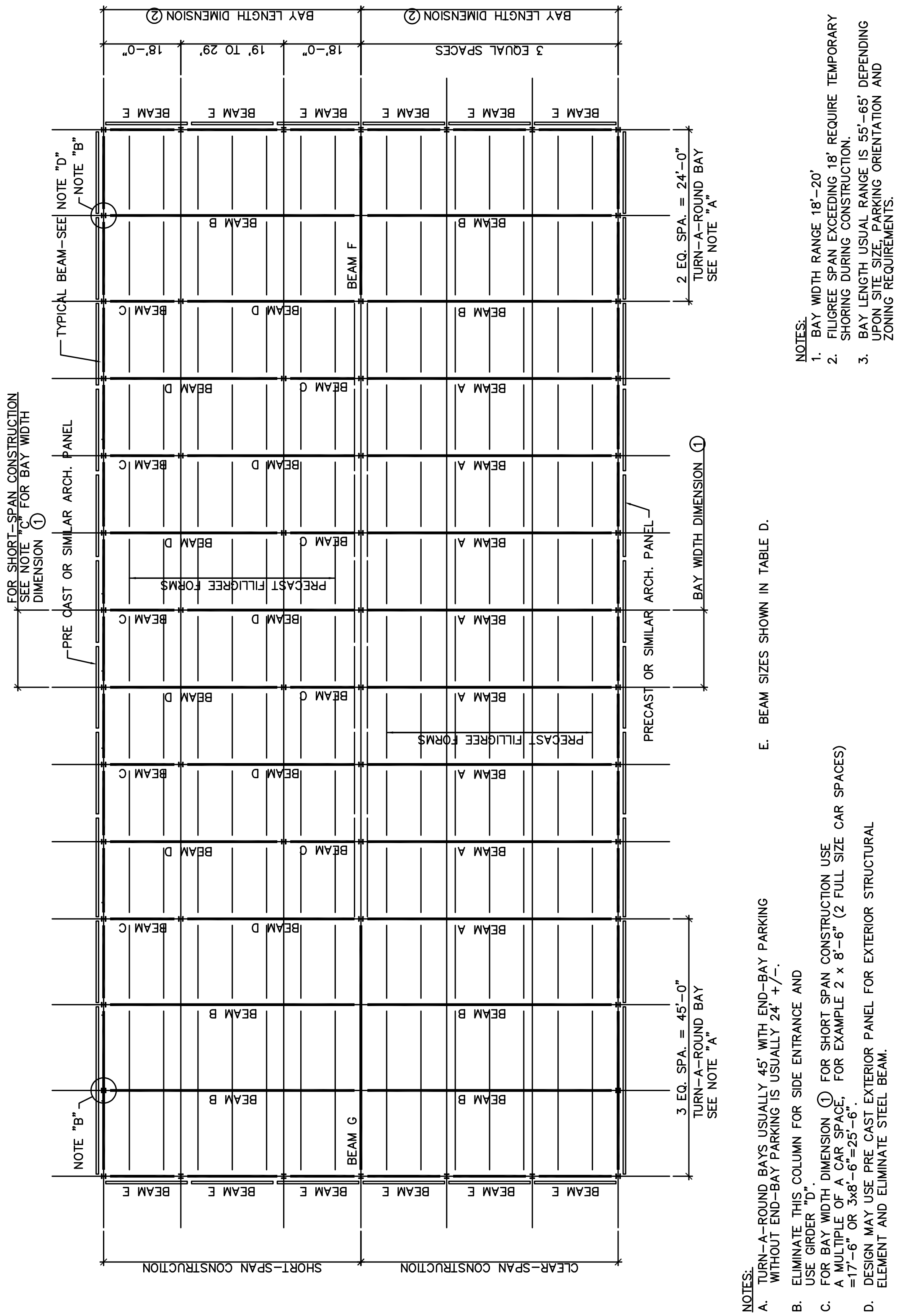


Fig. 3-22. Cast-in-Place Post-Tensioned Slab on Filigree Forms

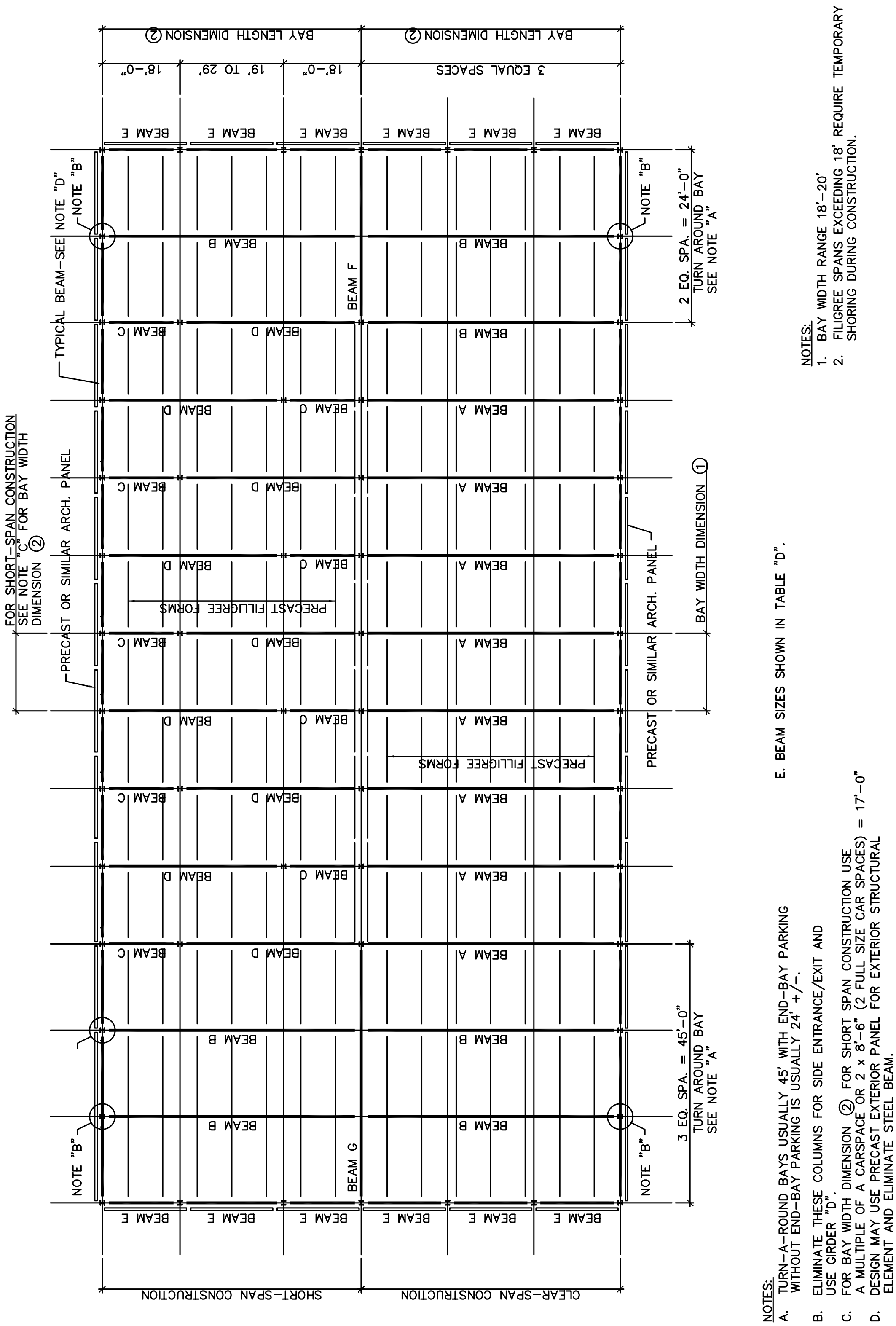


Fig. 3-23. Cast-in-Place Conventionally Reinforced Slab on Precast Forms

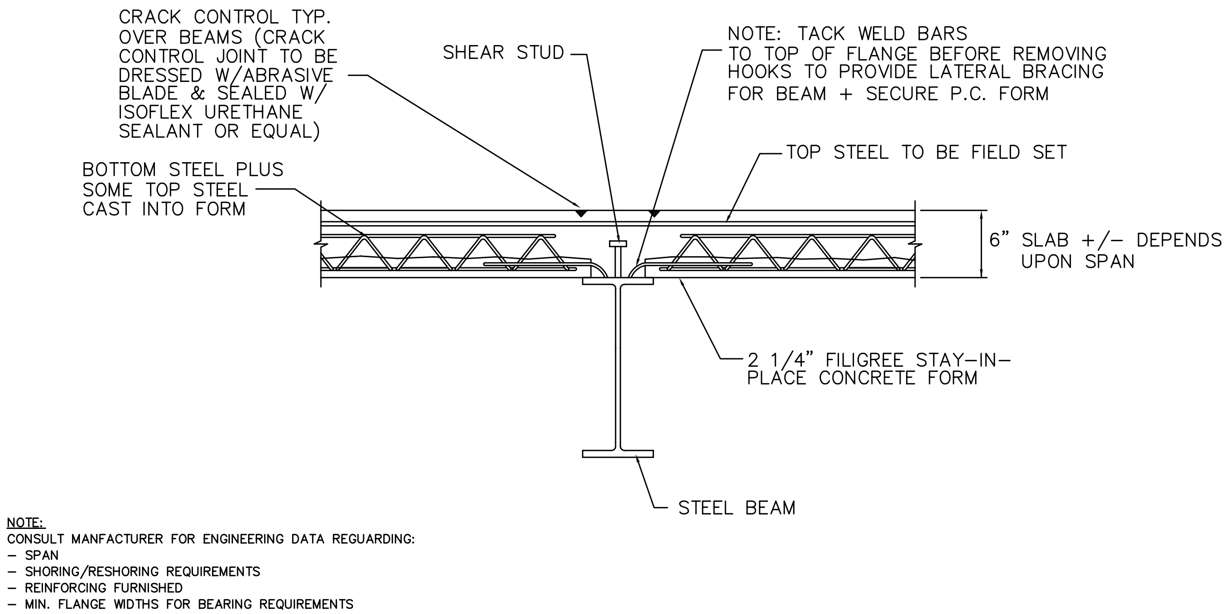


Fig. 3-24. Typical Detail at Control Joints

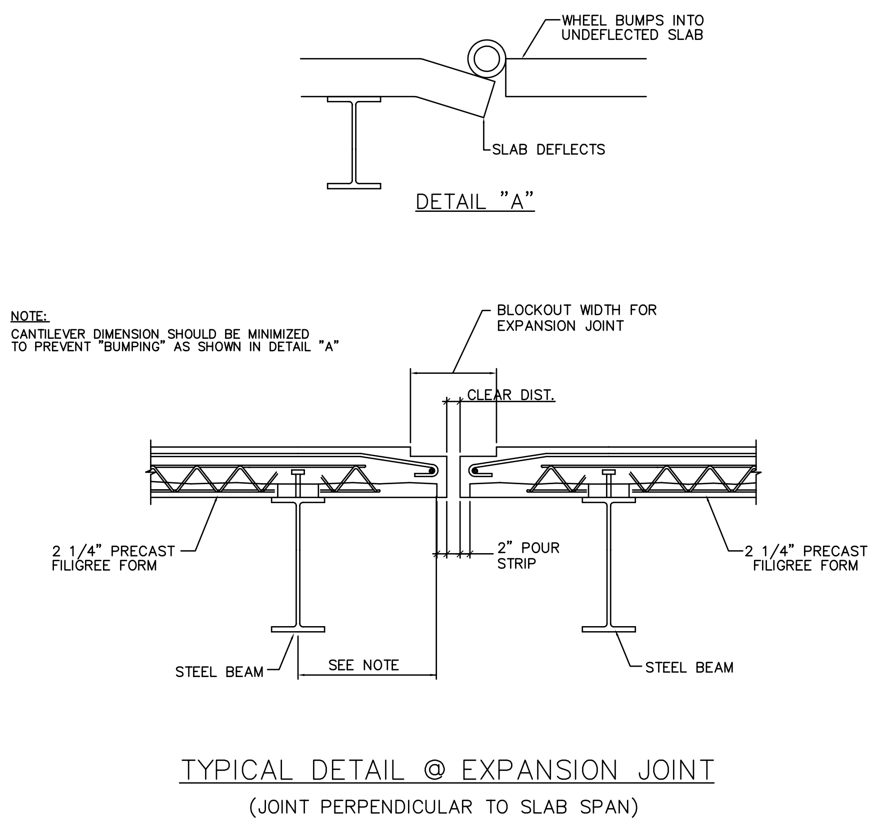


Fig. 3-25. Typical Detail at Expansion Joint—Joint Perpendicular to Slab Span

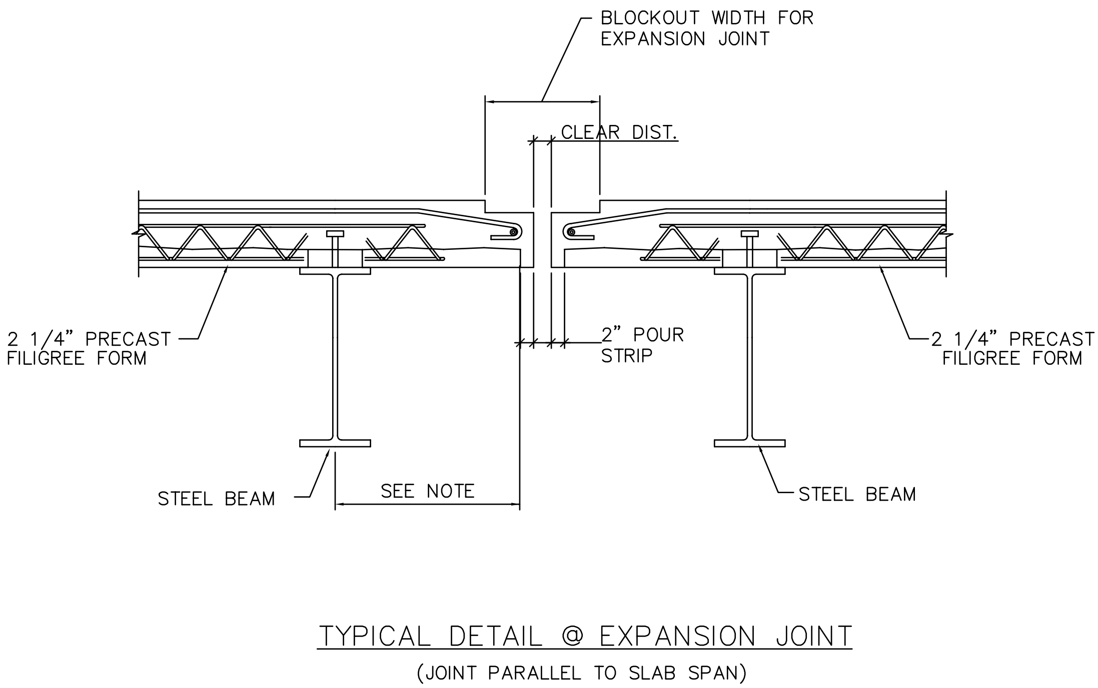


Fig. 3-26. Typical Detail at Expansion Joint—Joint Parallel to Span

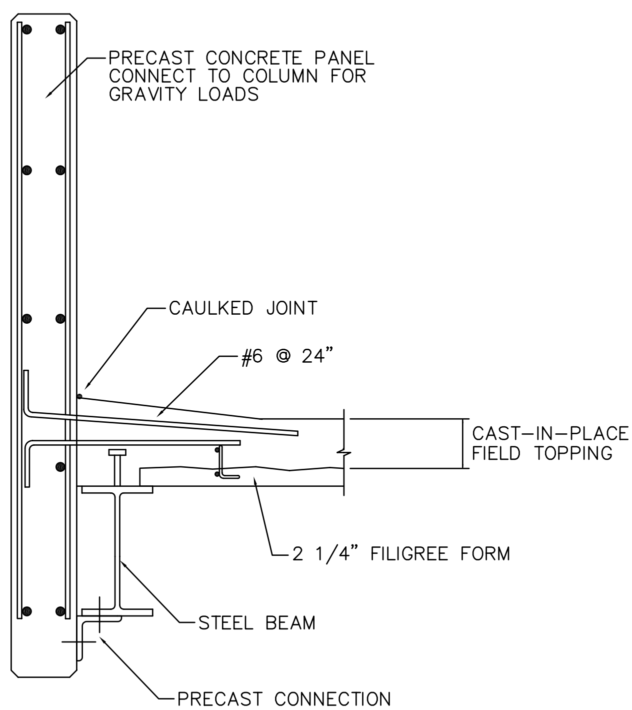


Fig. 3-27. Typical Precast Connection to Steel Beam and Slab

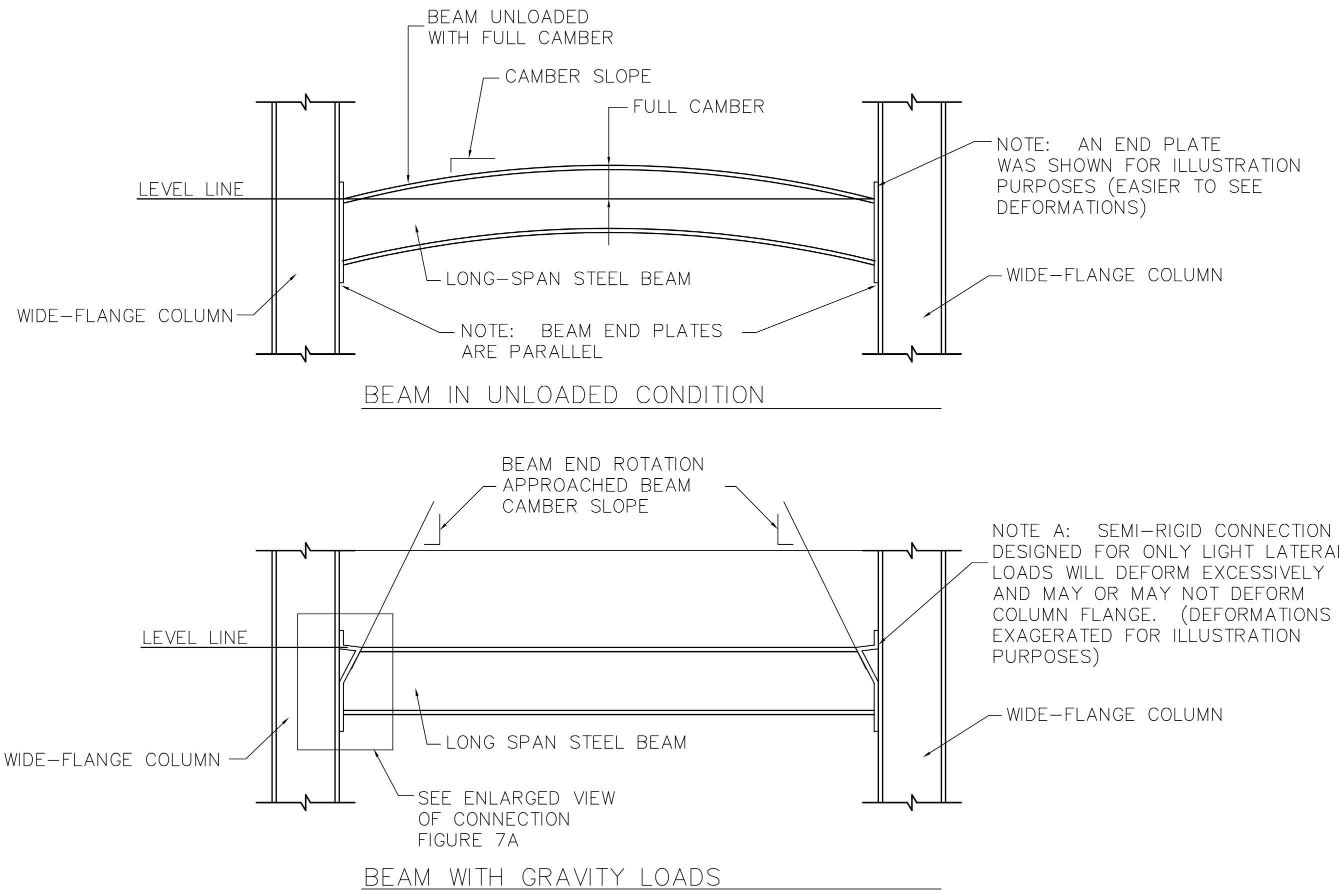


Fig. 3-28. Beam Loading Conditions

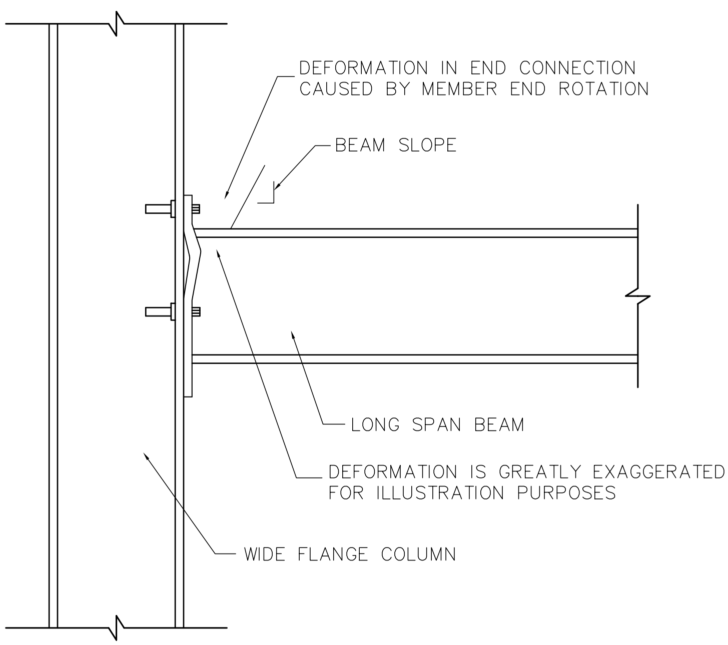
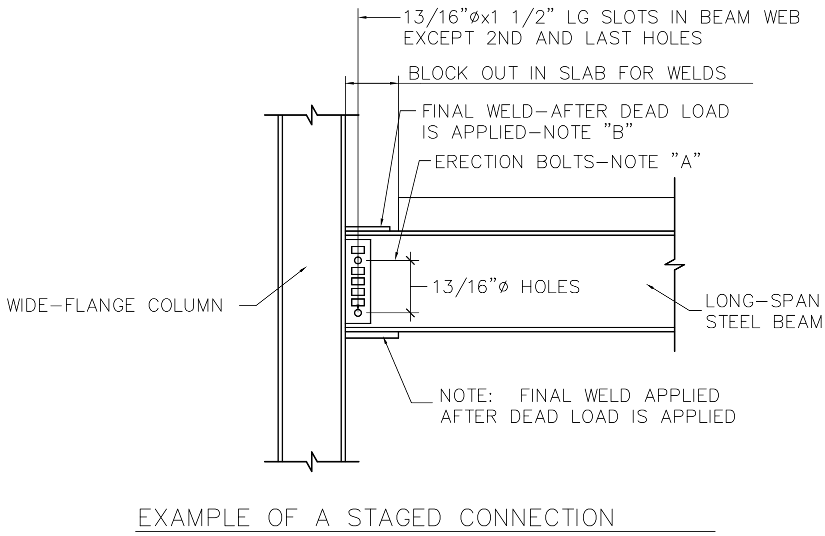


Fig. 3-29. Deformation in End Connections



NOTES:

A. THE ERECTION BOLTS ARE USED TO STABILIZE STRUCTURE DURING ERECTION AND IS REMOVED WHEN DEAD LOAD IS APPLIED (ONLY PULL BOLTS FROM BEAMS AFFECTED) THE TERM DEAD LOAD APPLIED REFERS TO ACTIVITIES SUCH AS SLABS BEING POURED, ETC.

B. AFTER DEAD LOAD IS APPLIED FINAL WELDING OF BEAM TO COLUMN CONNECTION SHOULD TAKE PLACE (OR SIMILAR DETAIL).

Fig. 3-30. Staged Connection

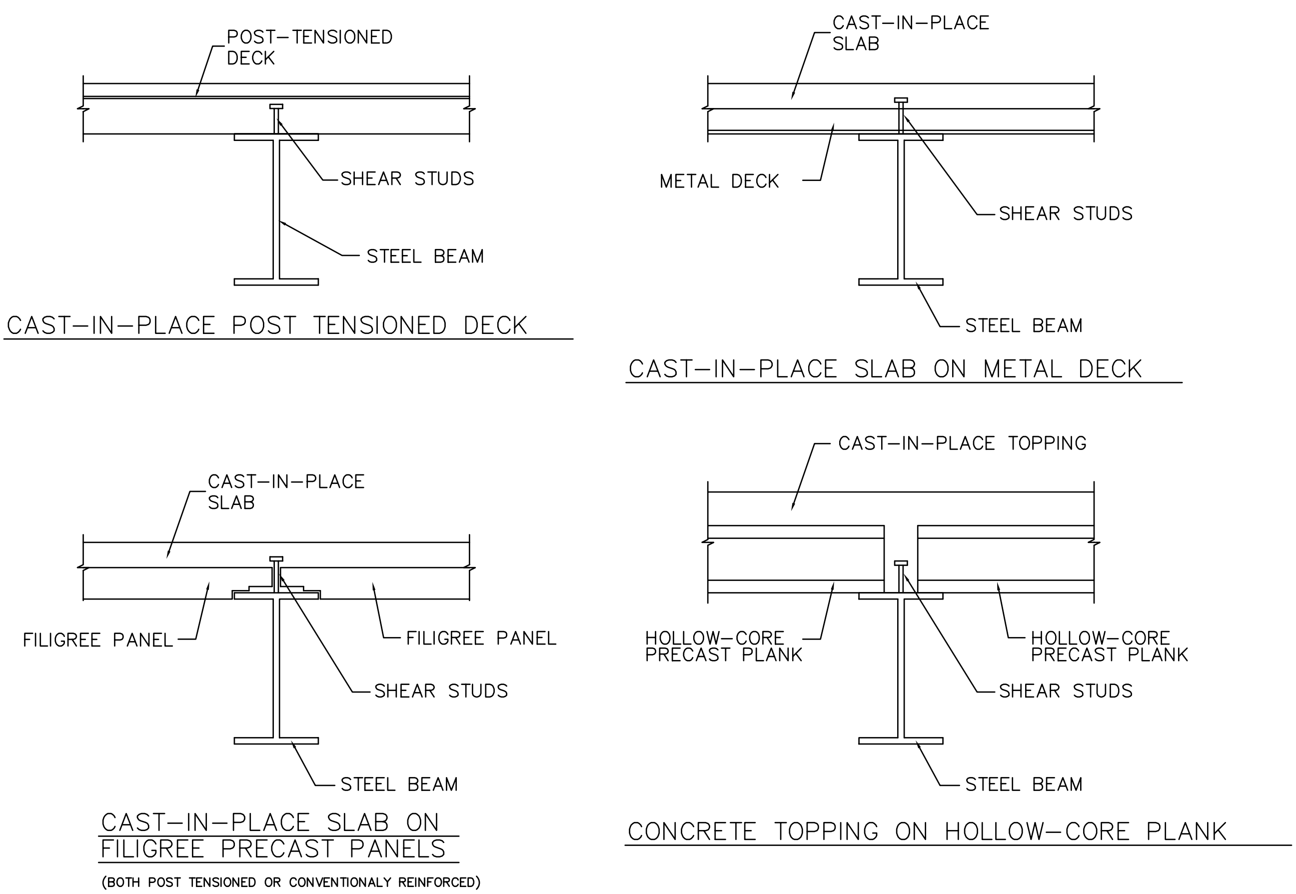


Fig. 3-31. Composite Beams

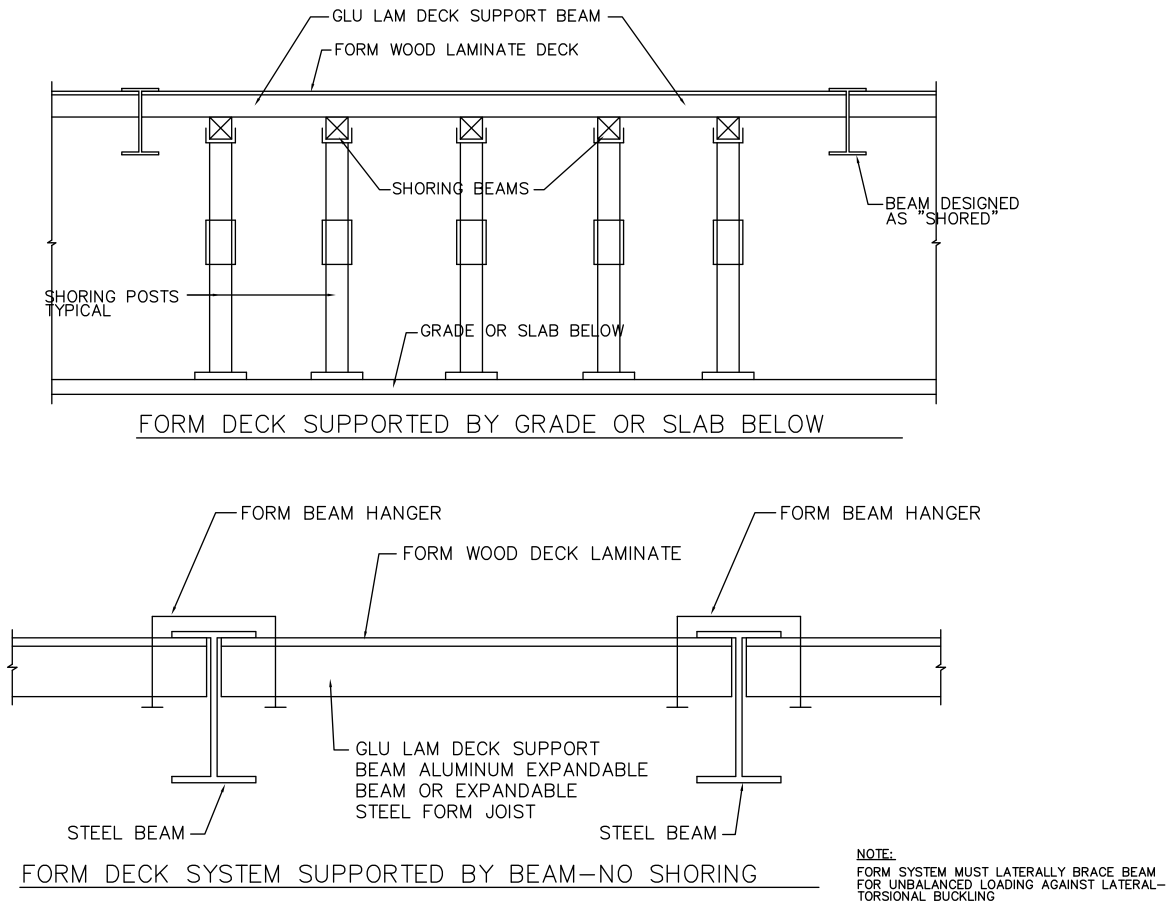


Fig. 3-32. Form Deck System Supported by Beam—No Shoring

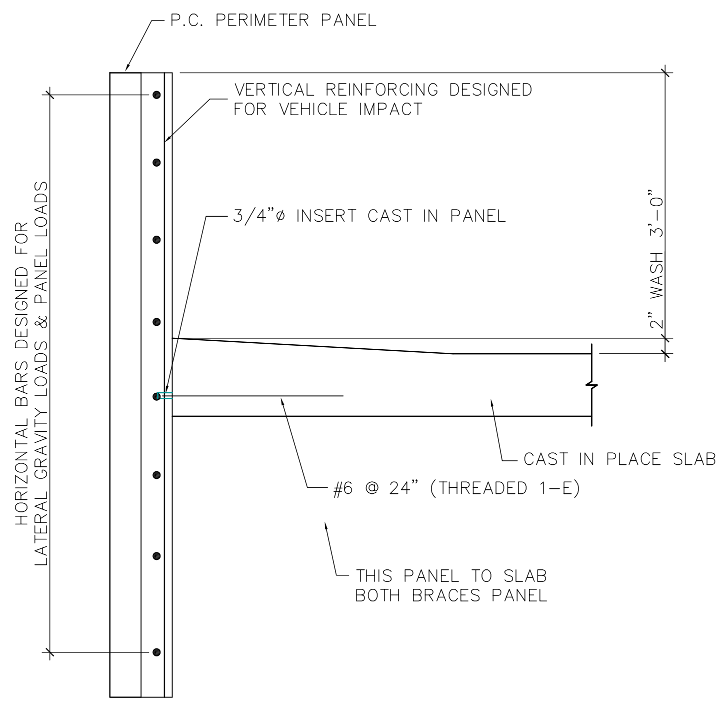


Fig. 3-33. Perimeter Panel Beams

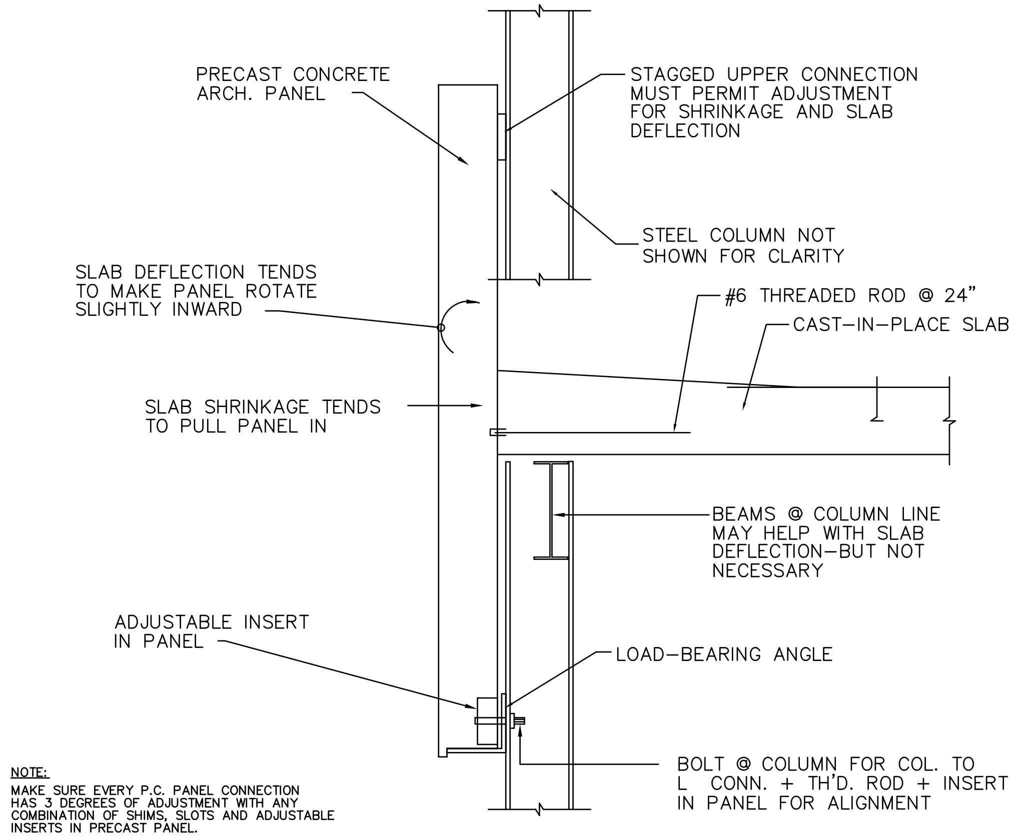


Fig. 3-34. Attachment of Precast Panels

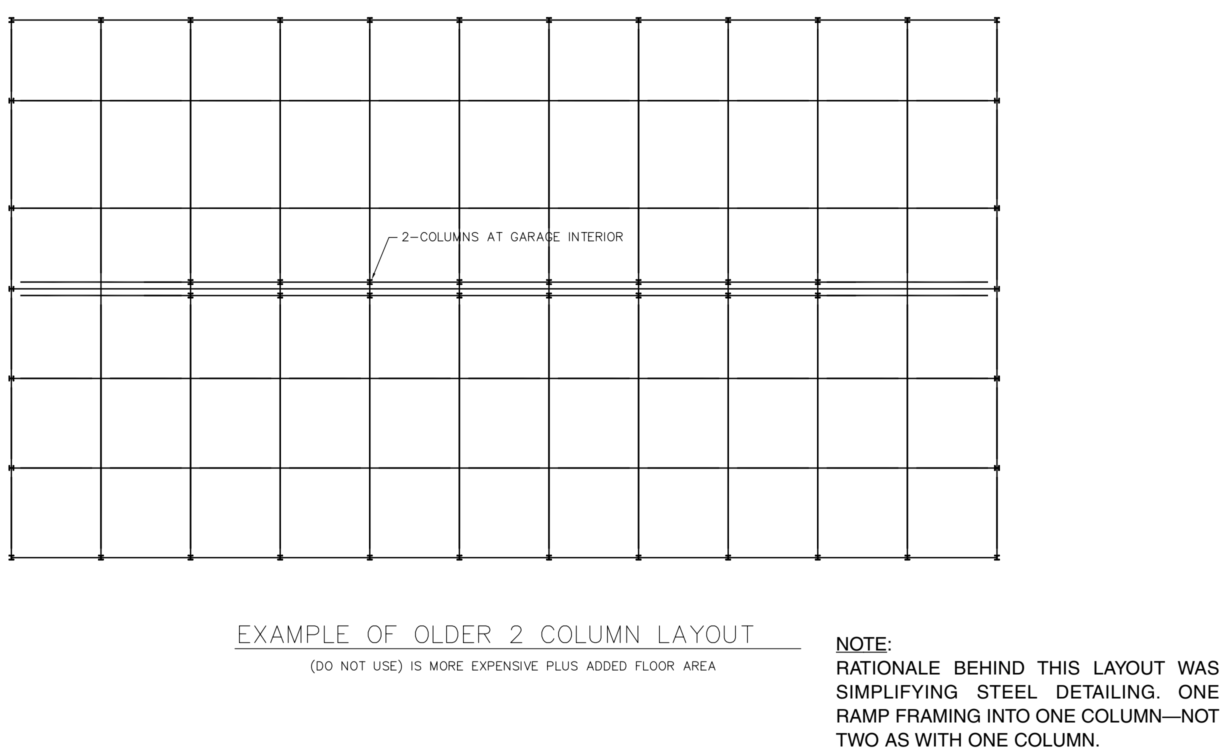


Fig. 3-35. Reinforcement of Older 2-Column Layout

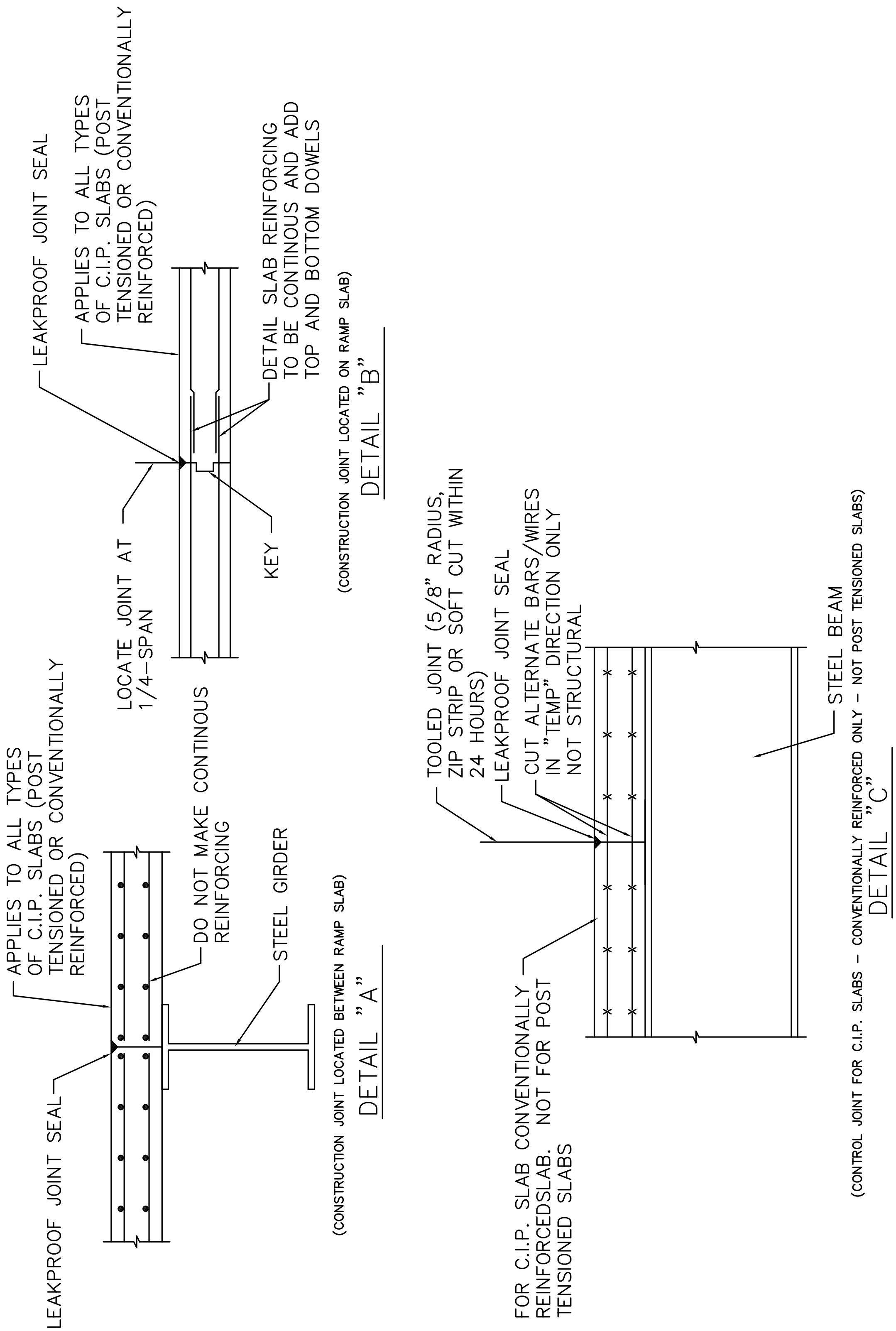


Fig. 3-36. Construction Joints

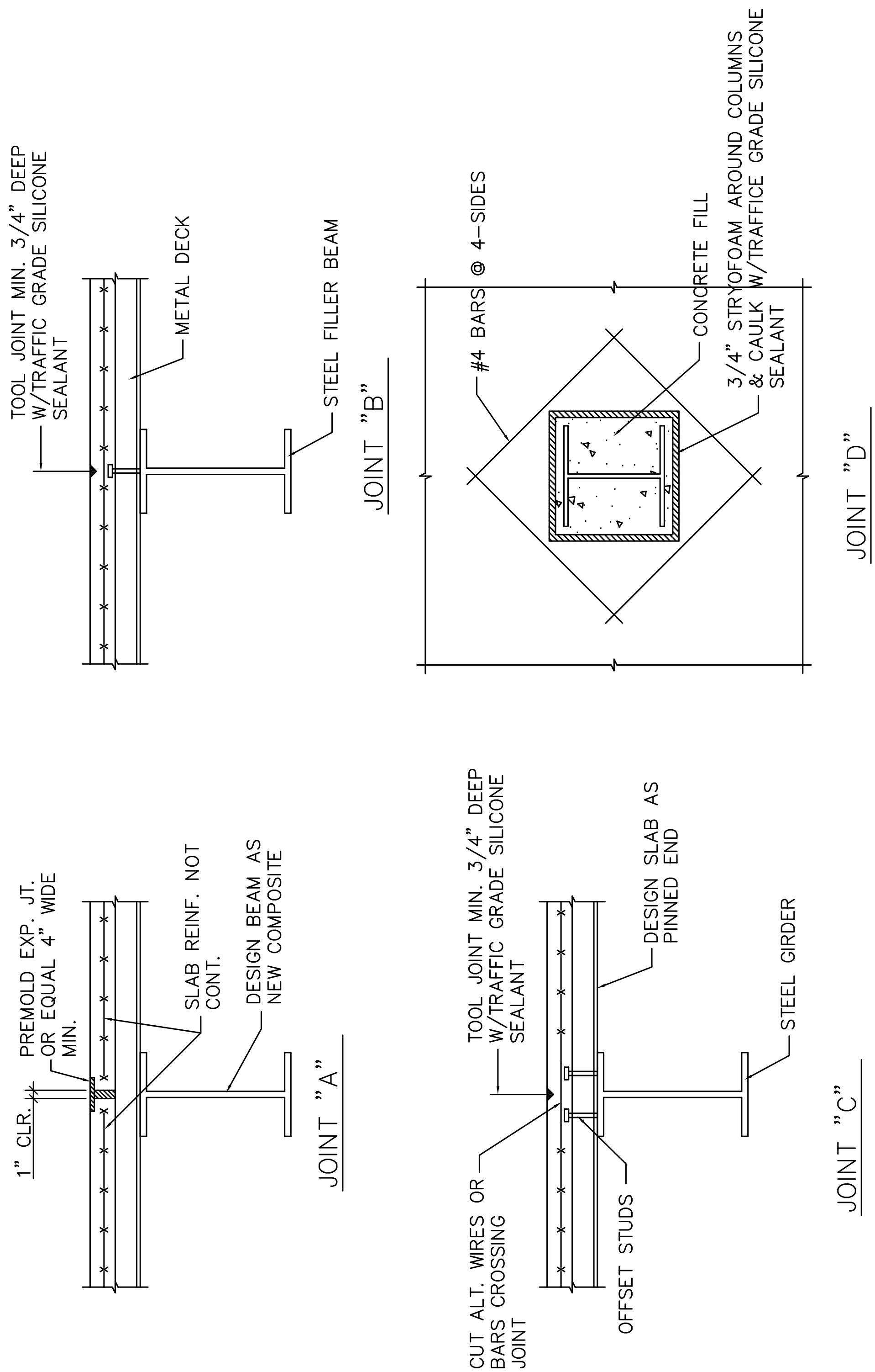


Fig. 3-37. Control Joints

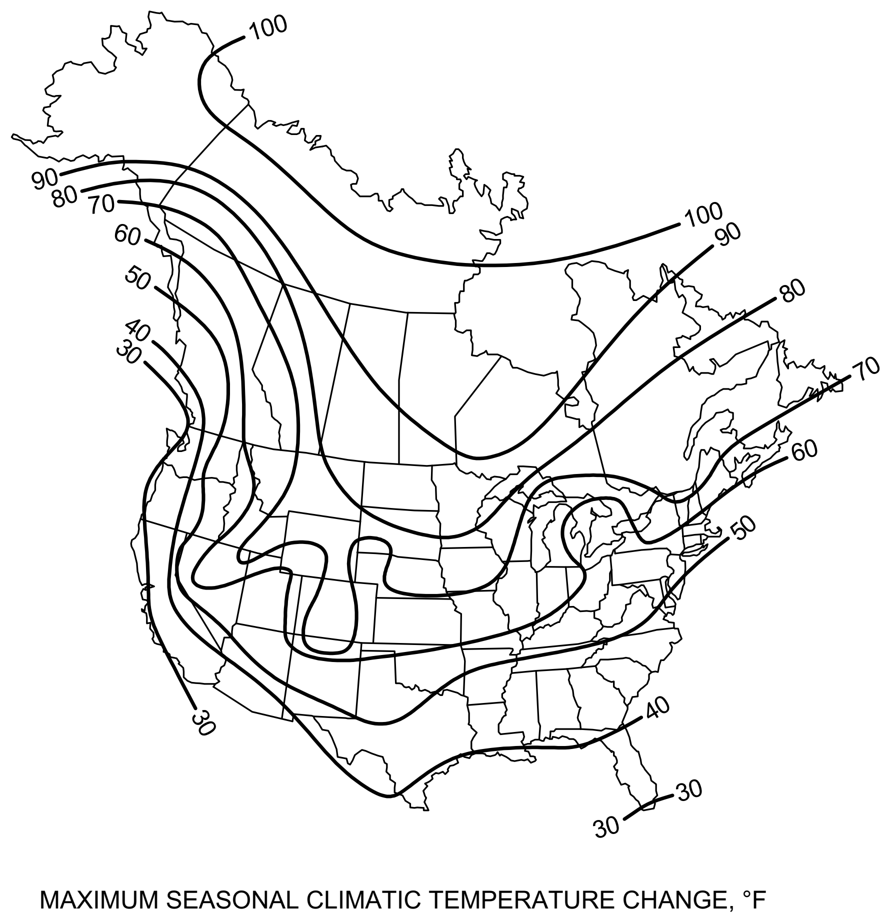


Fig. 3-38. Thermal Map of the United States

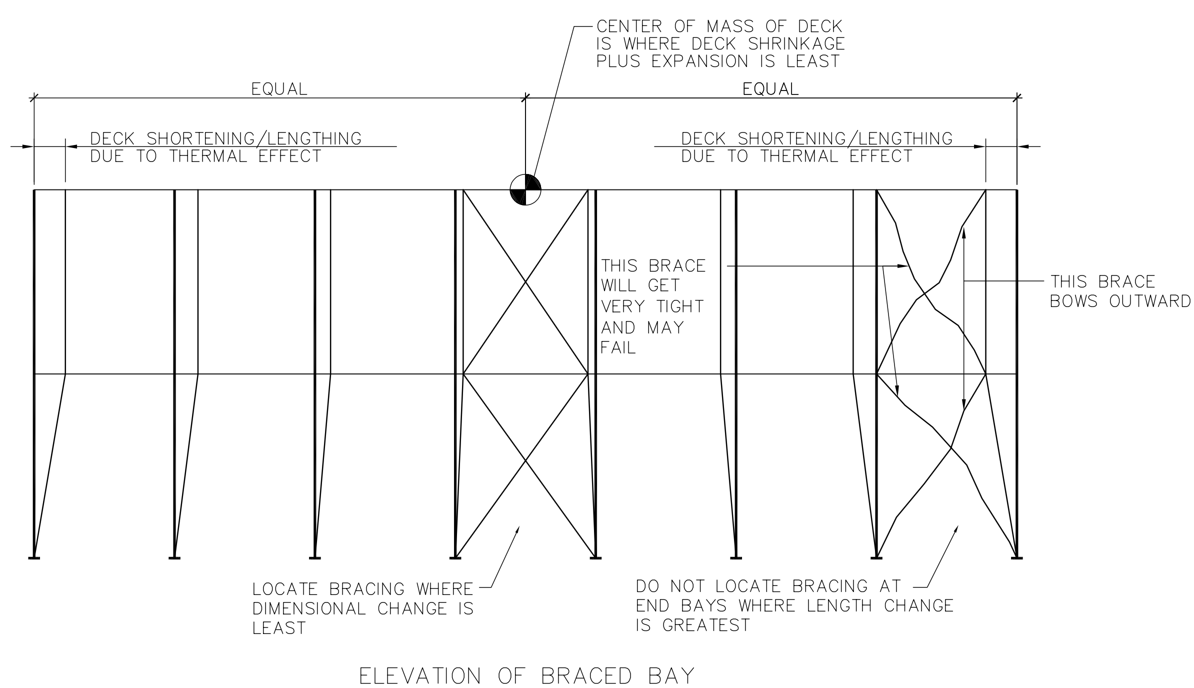


Fig. 3-39. Location of Bracing

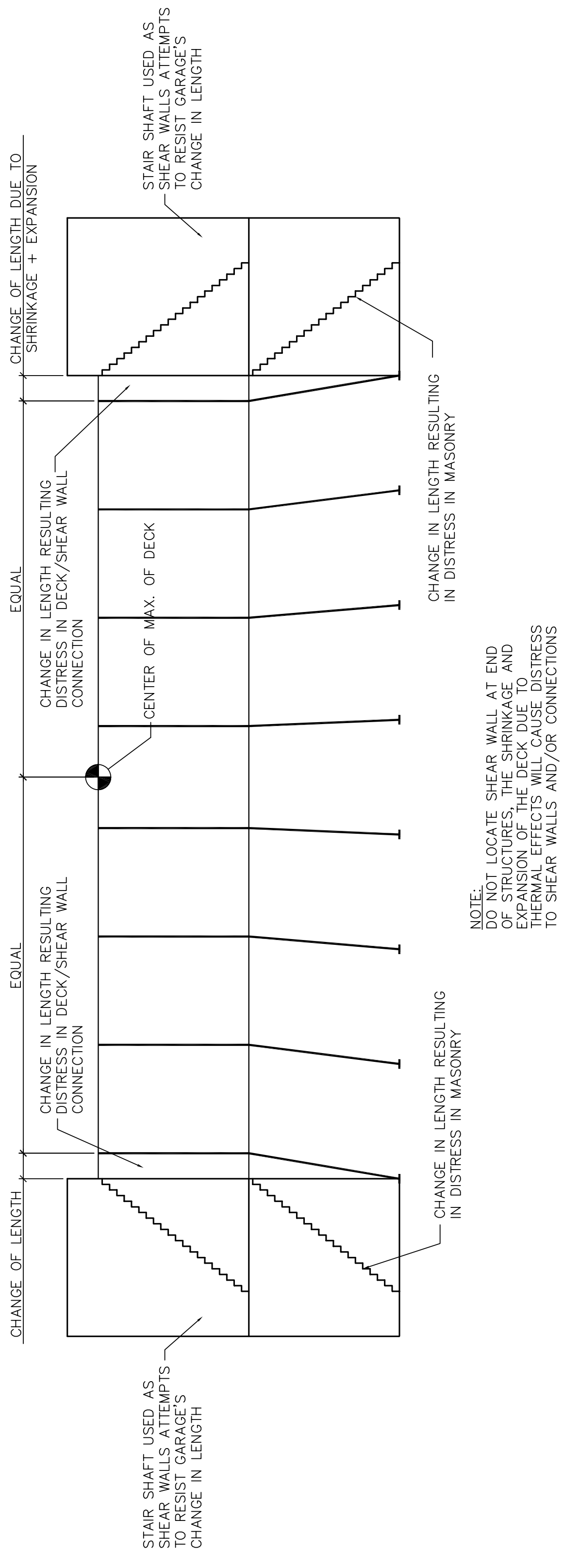


Fig. 3-40. Problems with Utilizing Stairwells for Lateral Bracing

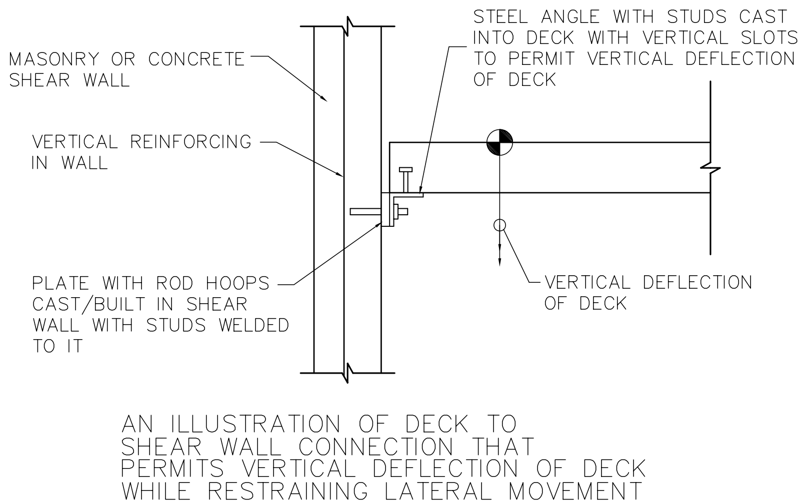


Fig. 3-41. Typical Shear Wall Connection

Chapter 4
Mixed-Use Structures

With the decrease of available property in urban areas, the availability of parking spaces takes on a new significance. Large sites on which on-grade parking could easily be provided are becoming increasingly difficult to find. Today's projects must provide a minimum amount of parking for both code and market requirements. Consequently, parking structures are being combined with or attached to business retail, commercial or residential structures.

A substantial number of new parking spaces in structured parking are now being generated below or above occupied space, within multi-story commercial, office, residential or public buildings. For buildings with a structural fire rating for the occupied portion above, building codes usually require that structural steel be fire resistant (fire rated), even if the parking levels themselves are above grade and open. (The applicable code may only require the steel framing that actually supports the occupied space above to be fire rated, if the parking garage, as an open, detached structure, could otherwise be built unrated.)

Fire tests in Australia have demonstrated that structural fire resistance in enclosed or below-grade steel parking garages may not be needed if the parking levels are provided with complete automatic sprinkler protection. In these tests, sprinklers were shown to be effective controlling fires in an enclosed parking structure by:

- Rapidly controlling the fire and confining it to the car of origin
- Maintaining both air and steel temperatures at low levels
- Reducing quantity and duration of smoke and toxic products

(Fire Safety Design Compendium - Version 1.0, October, 2001. OneSteel Market Mills; OneSteel Manufacturing, Pty. Ltd.)

Framing for parking under an occupied steel superstructure has often been built using site-cast concrete, but, if an office or residential building above parking levels is being designed in steel, there are valid reasons for using structural steel framing for the parking levels as well. First, transition from a steel superstructure to the concrete frame below is avoided. Second, in comparing total cost of steel and concrete framing for parking under mid- and high-rise buildings, owner/developers note that the basic economy of steel construction, together with income from earlier occupancy and reduced financing costs realized from speedier steel construction, can outweigh additional cost of fire protecting the steel framing in the garage levels.

For these and other reasons, taller buildings are now being designed with steel framing for parking below occupied space. Projects built with novel construction methods, such as "up-down", feature speedy erection of below-grade single-shaft steel columns and an on-grade concrete deck so that construction of the steel superstructure can proceed before the site-cast concrete parking levels below grade have hardly begun.

Excavation and foundation work increase cost of below-grade parking compared with above-grade. However, for parking up to two or three levels below grade, there is little impact on design of the steel framing itself. For deeper garages the design professional may have to account for substantial axial loads in the floor system due to lateral earth pressures. The steel beams and slab must be sized and detailed so that there is adequate strength, buckling resistance and vertical stiffness. Axial forces of 40 to 50 kips per linear ft along a basement slab are possible in deep car parks where the water table is high. In some of these cases all-concrete floor systems, with steel columns and some variation of the up-down construction method, may be the best solution. In Knuttunen, David O., and Henige, Richard A., Jr.; "Beam-Supported Slabs Subject to Edge Loading," Proceedings of the ASCE Structures Congress '89, ASCE, Knuttunen and Henige have proposed a model that allows a designer to estimate the stiffness required in the steel beam/concrete slab to resist buckling due to this magnitude of bi-axial edge loading.

An architectural trend has emerged for parking under low-rise buildings only as high as three or four stories. Prospective tenants for low-rise office, commercial and residential buildings often require bay size flexibility, spacious atriums and public spaces, and interior stairways connecting two floors for a major tenant. Intrusive bearing wall elements or closely-spaced column grids are simply unworkable or impractical for many of these projects, especially if a decent parking layout and traffic flow is to be achieved in a parking levels below.

When planning parking below occupied space, the designer must first decide upon a typical bay size that will at once satisfy the superstructure program above and the parking layout below. Column locations should be considered that allow 3 side by side parking spaces (typically a total width of 27 ft) and avoid columns in drive lanes.

Chapter 5
Fire Protection Requirements

Some projects may have special fire protection requirements associated with them as a result of mixed occupancy, exceeding height limitations, adjacency to property lines or the imposition of local requirements. As a result, the deck will need to carry a special rating. This must be considered when selecting a deck system.

Research data from actual fire occurrences over the past several decades in both the U.S. has demonstrated that for an open parking structure, non-crash vehicle fires do not result in heat build up or potential for flashover. (Denda, Dale F., Director of Research; Parking Market Research Co.; McLean, VA. "Parking Garage Fires—A Statistical Analysis of Parking Garage Fires in the United States: 1986–1988", April 1992.) The small percentage of area and volume of the parking structure involved in a vehicle fire (typically less than 2 percent of the area) allows adequate air volume in the uninvolved portion of the garage to mitigate the temperature and flashover potential of the fire. While no evidence exists of heat build up or flashover, the ventilation provided by the wall openings provides redundant protection for those concerns and tenable conditions for egress. Vehicle fires in parking structures are localized events. Further, research indicates that personal injuries in parking structure fires are rare and when they occur are generally unrelated to smoke or the fire itself.

Damage to the structural systems of parking garages as a result of vehicle fires has also been shown to be minimal. Data published in 1992 collected from 404 fire events reflected an average cost of structural damage of $131 or a total cost for all fires of $53,265. The evaluation of more recent fire events is consistent with the earlier findings.

Full scale fires tests, such as the Scranton Fire Test of 1972 and the Australia Test of 1985, conducted in open-deck parking structures have indicated that temperatures reached in the structure do not approach the critical temperature of steel even in the unlikely event of multiple vehicles becoming involved in the fire.

However, application of both model and local building codes require a fire rating for steel framing systems in certain applications. Table 1-2 in Section 1.3.4 of this guide outlines the criteria for determining fire protection requirements for various configurations of open-deck parking structures under the model building codes. A general summary is that:

- A two-hour rating is required if the structure is greater than 75 ft in height or the shortest distance to a 40 percent open side from any point on the deck is greater than 200 ft
- Some local codes may require fire protection of all steel parking structures
- Some limited extension of the 75 ft height restriction is possible for structures less than a total of 400,000 ft^2 or if the structure is open on more than two sides.

Application of spray on fire protection material on the steel frame will increase the project cost by approximately 10 percent. Intumescent paints are the most expensive form of fire protection but provide the most attractive appearance within the structure. As intumescent paint becomes more popular, manufactures suggest the cost of this product will decrease.

Table 5-1 provides a summary of various approaches to and recommendations concerning fire protection materials. In addition to these cementitious materials and intumescent paint systems, fire protection requirements may be addressed by encasing columns in concrete. Sprinkler systems, although considered by some authorities to be ill advised in a parking structure because of their propensity to create a fog cloud hindering fire fighter visibility, also can be used in some jurisdictions to provide a portion of the fire rating.

It should be noted that none of these fire protection systems provide corrosion protection for the steel. To control corrosion, fire protection and should be applied over either a zinc-rich primer or a galvanized surface. Coordination should take place between the coating manufacturer or galvanizer and the fire protection material manufacturer to verify the adhesion between the corrosion protection and fire protection material.

All systems are field applied except for intumescent paint, which may be shop or field applied. If shop applied, field touch up will be required. Design consideration should be given to the overall increase in structure weight by 1 to 3 pounds per ft^2 for the fire protecting material. The actual weight increase should be confirmed with the manufacturer.

Consideration during the design and selection of framing members can reduce the cost of the required fire protection. By maximizing the massivity of the steel member (W/D = (lbs/ft)/cross-section perimeter) the amount of required fire protection will be lessened.

Table 5-1 Fire Protection Materials

Fireproofing Material	Low Density	Medium Density	High Density	Intumescent Paint
Density	15-19 pcf	22-25 pcf	39-50 pcf	
Application	Spray – wet or dry	Spray – wet or dry	Spray – wet and/or trowel	Spray
Primer	Optional*	Optional*	Optional*	Yes, if required
Corrosion Protection Afforded	None unless galvanized or HPC	None unless galvanized or HPC	None unless galvanized or HPC	Yes, with compatible primer
Topcoat paintable	Yes	Yes	Yes	Yes
Impact Durability	Poor - concealed	Low	Good	Good
Weather exposure	None	Limited	Exterior	Exterior –mastic or solvent based only
Use	Concealed	Exposed inaccessible	Exposed	Exposed

* adherence between fireproofing and paint must be confirmed by manufacturer

Recommended systems if required in a Parking Structure:

System
Intumescent Paint over zinc rich primer or galvanizing
High Density trowel over zinc rich primer or galvanizing
High Density spray over zinc rich primer or galvanizing
High Density 1/8” topcoat spray over low density spray over zinc rich primer or galvanizing

Chapter 6
Barriers and Facades

6.1 Impact Requirements

Barrier rails are required at the perimeter of a parking structure to provide adequate restraint to prevent a vehicle from breaking through the barrier and dropping the ground surface below. The National Parking Association recommends using an ultimate point load of 10,000 pounds applied on a one sq. ft area at a distance of 18 in. above the riding surface, located at any point along the length of the riding surface.

6.2 Railing Code Requirements

Openings in railings should be designed to prohibit individuals from being able to scale the railing and should prevent young children from either passing through the barrier or becoming lodged in gaps within the barriers. Openings in the railings and the spacing of the components should conform to the applicable sections of the local governing building code.

6.3 Facade Options

The selection of the facade treatment on a parking structure is important with respect to security, aesthetics and cost. Parking garage owners and developers are now realizing that a visitor's first impression of their organization is conveyed by the parking structure. Many municipalities are now requiring that parking structures blend into the architectural aesthetics of the neighborhoods in which they are located. For these reasons the choice of a facade treatment is a critical choice for the garage.

A variety of options for facades are available. The use of steel railing or cable systems provide a low cost solution that enhances the openness of the garage and the perception of the security of those using the garage. Precast concrete panels are available with a variety of surface treatments including exposed aggregates and brick inlays. Composite sandwich panels, which combine a variety of architectural options similar to those available with precast panels with a reduced structural weight, are available for a reasonable cost. Some parking structures utilize brick or masonry exterior treatments. All of these façade systems can be easily attached to a steel framing system.

Care must be exercised in the selection of a facade system to ensure that the required openness to maintain a rating as an "open-deck parking structure" is maintained. Failure to maintain proper openness may result in mechanical ventilation and fire protection being required. If a required facade system results in less than the required amount of openness and sufficient space is available, it may be possible to offset the facade away from the face of the structure to maintain the required openness.

It is important to recognize that these facade treatments may serve a structural function in addition to the architectural presentation. The ability of the facade to carry live, dead, bumper, wind or seismic loads must be considered in the design process. It is also important to resolve the possible conflict between the requirement for a stiff structure to support a rigid façade and the need to maintain flexibility in the main structure.

6.4 Perimeter Protection

Perimeter protection or vehicle and pedestrian barriers are required in all locations where a differential in elevation exists. Perimeter protection for both pedestrians and vehicles almost always occurs in the same location; therefore the requirements are blended together resulting in a minimum height requirements of 42 in. above finished floor (above concrete wash. See Figure 6-1.), a maximum opening of 4 in., and a capability to withstand a force of 10,000 pounds at a distance of 18 in. above the finished floor.

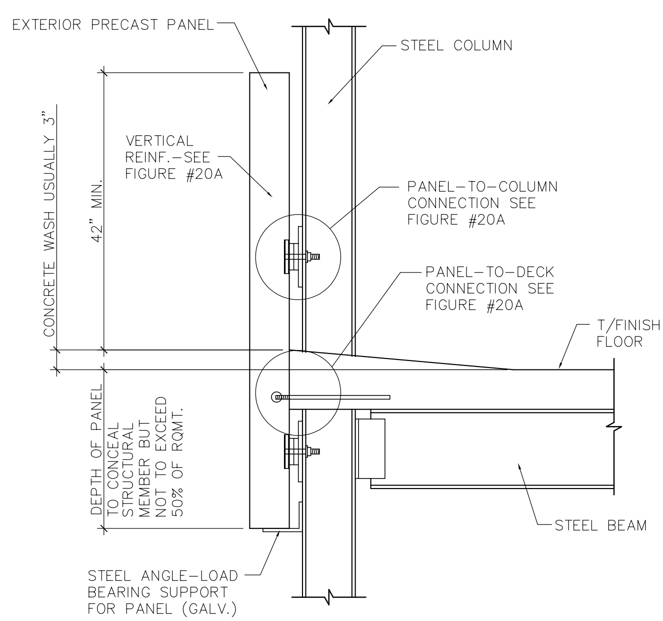


Fig. 6-1. Typical Precast Panel to Steel Column Connection

In a steel structure these requirements are usually met by using precast architectural panels or open steel members.

6.4.1 Precast Architectural Panels

Most parking structures utilize exterior precast architectural panels for perimeter protection. The design of these panels and their connections is straightforward, and follows a few simple rules:

1. Panels should span column to column.
2. Panels are usually 6- to 7-in. thick depending on span, lateral loading and architectural requirements.
3. Panel connections must incorporate support from the deck slab for vehicle impact loads.
4. The panel to deck connections must accommodate vertical deck deflection. See Figure 6-2.
5. Panel connections must incorporate the difference in erection tolerance between AISC steel erection and the architectural exposed precast panels.
6. A bearing seat must be provided as a part of the precast panel connection to take the weight off of the panel.

6.4.2 Open Steel Member Design

The open steel member design is usually used in the interior of the garage because it is less costly while providing a greater degree of openness. Listed below are the main points for consideration as shown in Figure 6-3 are:

1. The impact member, usually a tube or cable, must absorb the 10,000-pound impact force.
2. The perimeter protection cannot have an opening through which a 4-in. sphere could pass.
3. Some local codes do not permit the use of ½ in. diameter prestressed cables at four inches center to center because of the possibility of the ladder effect.

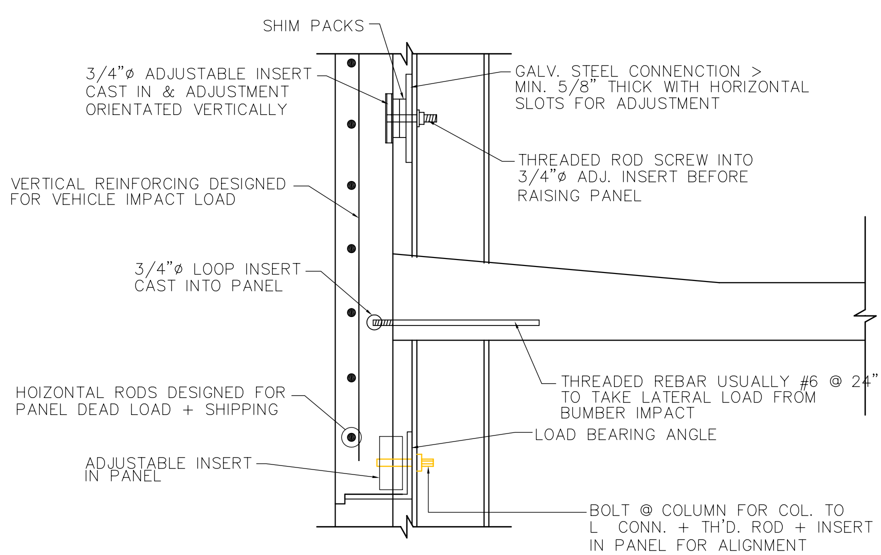
6.4.3 Cable Barrier Design Calculations

Barrier cable design for a parking structure is unique because it utilizes a “cable” as it’s design element, not a beam, column, slab, or panel, as is popular with most designers.

Why are cables so effective as a restraining device? Cables are extremely efficient members for restraint because they convert a concentrated or uniform force applied on an axis normal to their span into a tension load, which requires very little material to resist. This section will provide a few design considerations and simple equations that will be helpful in arriving at a design solution.

LOAD

As previously noted, there are no code prescribed loads for parking structure design, nor are there any code prescribed



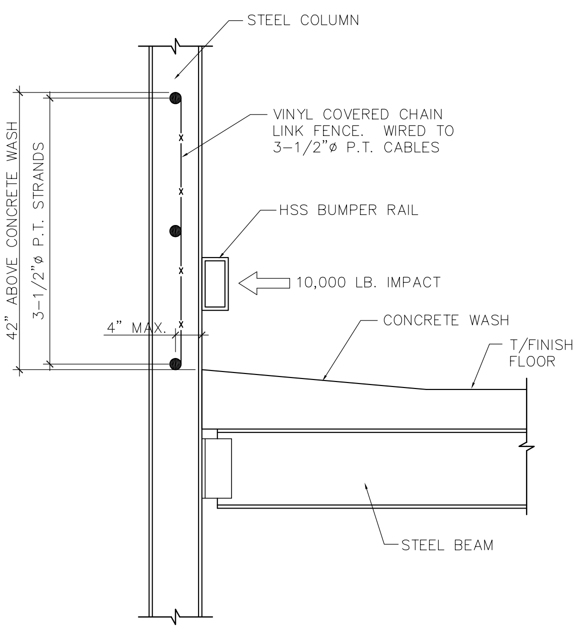
NOTE:
MAKE SURE EVERY P.C. PANEL CONNECTION HAS 3 DEGREES OF ADJUSTMENT WITH ANY COMBINATION OF SHIMS, SLOTS AND ADJUSTABLE INSERTS IN PRECAST PANEL.

Fig. 6-2, Typical Precast Panel to Steel Column Connection analytical methods. A 10,000 pound concentrated load applied 18 in. off the deck is the generally accepted design loading. A cable analysis has more than one variable, so it is an iterative solution. Cable equations describe the displacement of a cable in terms of its area length, applied load, and pre-tensioned load. The larger the pre-tensioning force, the more taught the cable, the less the cable is able to deflect. These cables cannot be tightened without taking into account the forces they would put on the columns. Some of these forces could be quite large, up to 27 kips per in. each.

Several of these cables could easily overload a small column that supports a small vertical load such as a corner column supporting only one level.

DESIGN EQUATIONS

There are two equations required for the design of a barrier cable. Equation numbers Eq(1) and Eq(3) are used to solve for the deflection. Then the value of the deflection is used in equations Eq(2) and Eq(4) to solve for the tensioning forces in the cable due to load. There are two sets of equations listed; one for a uniform load case and one for a concentrated load. The pre-tension forces must be added to the tension forces calculated from the load and compared with the cables ultimate capacity force or factor of safety.



NOTE:
MANY OLDER GARAGES USED 1/2" DIA. CABLES @ 4 1/2" C/C FOR PEDESTRIAN BARRIER. THIS IS NO LONGER PERMITTED BY SOME CODES BECAUSE IT CAN BE CLIMBED LIKE A LADDER.

Fig. 6-3. Typical Open Steel Member Design

UNIFORM LOAD CASE

1. Solve for deflection noted as a.

$$a = \left[\frac{3wl}{64EA}\right]^{\frac{1}{3}} l \tag{1}$$

where:

w = Uniform load
l = Overall length of the cable
E = Elasticity (29,000 ksi)
A = Area of cable (0.153 in.^{2} is area of a ½-in. diameter, seven strand cable, which is the most commonly used)
a = Deflection

2. Using value calculated for deflection a solve for pre-tensioning forces noted as T.

$$T = \frac{wl^2}{8a} \tag{2}$$

where:

T = Tension force in the cable
l = Cable span
a = Cable deflection from Equation (1)

CONCENTRATED LOAD CASE

1. Solving for cable deflection a.

$$a = \left[\frac{Pl^2 L}{8EA}\right]^{\frac{1}{3}} \tag{3}$$

where:

P = Applied load (10 kips in this case)
l = Span of cable
L = Overall length of cable
E = Elasticity (29,000 ksi)
A = Area of cable (usually ½-in. diameter cable area or 0.153 in.^{2})

2. Using value calculated for deflection a solve for pre-tensioning forces noted as T.

$$T = \frac{Pl}{4a} \tag{4}$$

where:

P = Applied load
l = Cable span
a = Cable deflection from Equation (3)

To complete the design, Equation (5) adds the tension forces and compares them to the cable's ultimate strength.

$$Factor\ of\ Safety = \frac{T + T'}{T_{ult}} \tag{5}$$

where:

T = Calculated tension force due to applied load
T' = Pre-tensioned force
T_{ult} = Ultimate capacity of the cable

Chapter 7
Stairs and Elevators

7.1 Stair Locations and Requirements

Stair Location

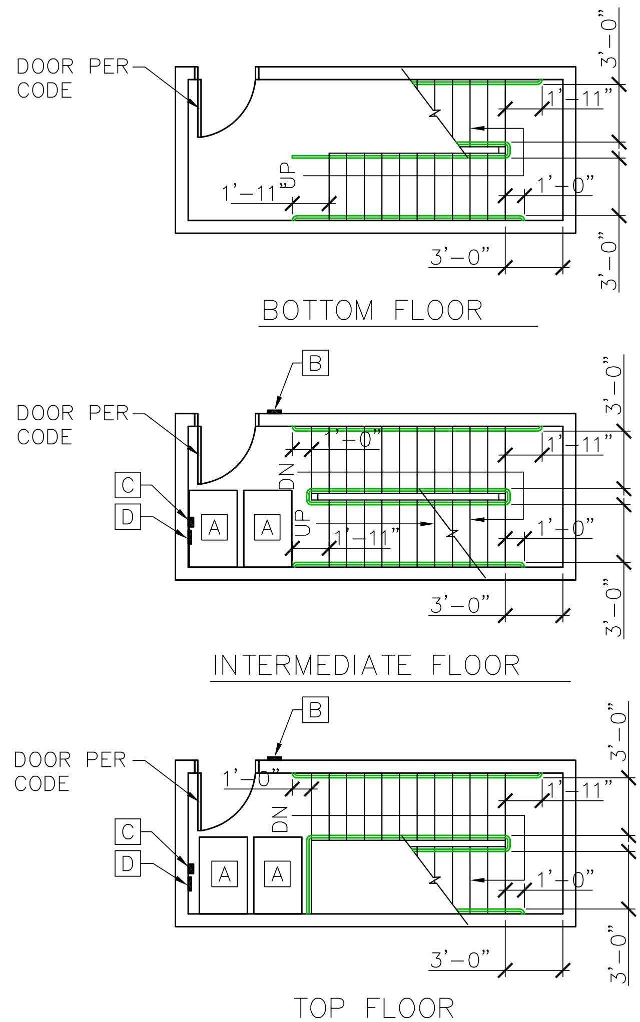
Every parking structure is required to have a minimum of two means of egress (emergency exits, i.e., stairs) that are remote from each other. The maximum travel distance determines the numbers and locations of the stairs. Travel distance is defined as the distance an occupant must travel from any point in the structure to the closest stair. In open structures this distance is 300 ft.

Specific Recommendations:

1. The minimum width of a stairway is 36 in.
2. Vertical rise shall not be more than 12 ft between landings.
3. Minimum tread width should be 11 in.
4. Maximum riser height should be 7 in.
5. Handrails are required on both sides of the stairs
6. Handrails can be between 34 in. and 38 in height. 34 in. is the standard height.
7. Handrails must extend 12 in. beyond the top riser and 12 in. plus one tread width at the bottom of the stair run.
8. The handrail diameter is 1¼ in. minimum to 2 in. maximum. 1½ in. diameter is standard.
9. Minimum headroom requirements are 80 in. clear.
10. Open railings must have baluster or be solid such that a 4 in. sphere cannot pass through any opening.
11. An area of refuge must be provided at each exit and at each level above and below grade. This area must not interfere with the path of travel and shall be no less than 6 ft^2.
12. Stairs must be well lit and, if enclosed, glass should be considered for the enclosure.

7.2 Elevators

Elevators have become an essential component of parking structure design and construction for both user convenience and Americans with Disabilities Act (ADA) requirements.



LEGEND

A WHEELCHAIR SPACE: 30 X 48 in. MINIMUM OF TWO REQUIRED WITH ONE PER 200 OCCUPANTS BY THE AREA.

B "AREA OF RESCUE ASSISTANCE" SIGN WITH SYMBOL OF ACCESSIBILITY ILLUMINATED IF REQUIRED FOR EXIT SIGNS. DIRECTIONAL SIGNS ARE ALSO REQUIRED.

C AUDIBLE AND VISUAL TWO-WAY COMMUNICATION UNIT.

D INSTRUCTION ON USE OF SPACE.

Fig. 7-1. Plan View—Stairs

Elevators

The number of elevators required will vary with garage usage. For an employee garage, there will be peak high demand periods separated by low demand periods. In contrast, garages in shopping malls will have a more even demand. The designer should consult with an elevator specialist or manufacturer's representative to determine optimum design. There are also a few good rules of thumb to observe when considering elevators.

- Garages with capacities up to 200 cars with less than three supported levels need only one elevator. Consideration must be given to patrons' use of available stairs, should the elevator be out of operation.
- Garages with a capacity up to 500 cars should have at least two elevators.
- Garages with capacities over 500 cars should have two elevators for the first 500 cars and one elevator additional for each additional 500 cars.

Type of Elevators

Elevators come in two typical types, hydraulic and traction. Hydraulic elevators are usually slower, less costly, and have a vertical travel limit of approximately 60 ft. Hydraulic elevators require a small machine room at grade level. They function like a large telescoping lift, as shown in Figure 7-4. Hydraulic elevators do not require a large overhead machine room but only overhead clearing required by code. Traction elevators are moved by electric motors that are above the elevator cabs. Traction elevators require a machine room above the elevator shaft that is always larger than the shaft. Traction elevators cost more than hydraulic elevators, and require a large elevator tower/machine room, however, their vertical speed and capacity is much greater that the hydraulic elevators. Always consult with an elevator manufacturer for all specification requirements.

Size of Elevators

Parking garages usually only require a standard 2,500 pound capacity elevator. Designers, however, should review for unusual applications such as:

- Health Care Services—elevators equipped to carry wheel chairs and gurneys
- Sports Facilities
- Maintenance Facilities

All elevators require pit ladders, sill angles, and hoist beams.

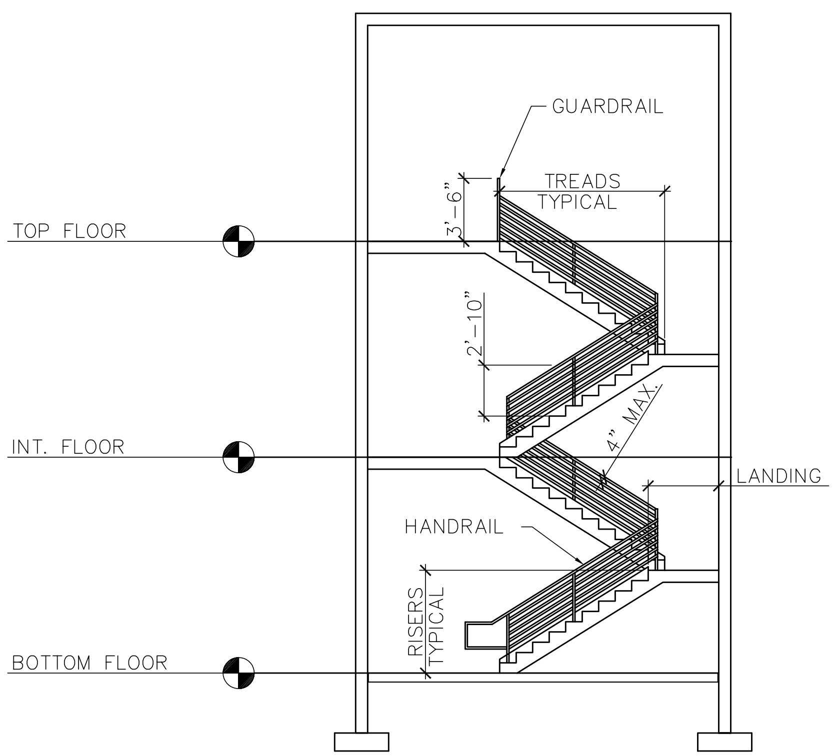


Fig. 7-2. Typical Stair Section

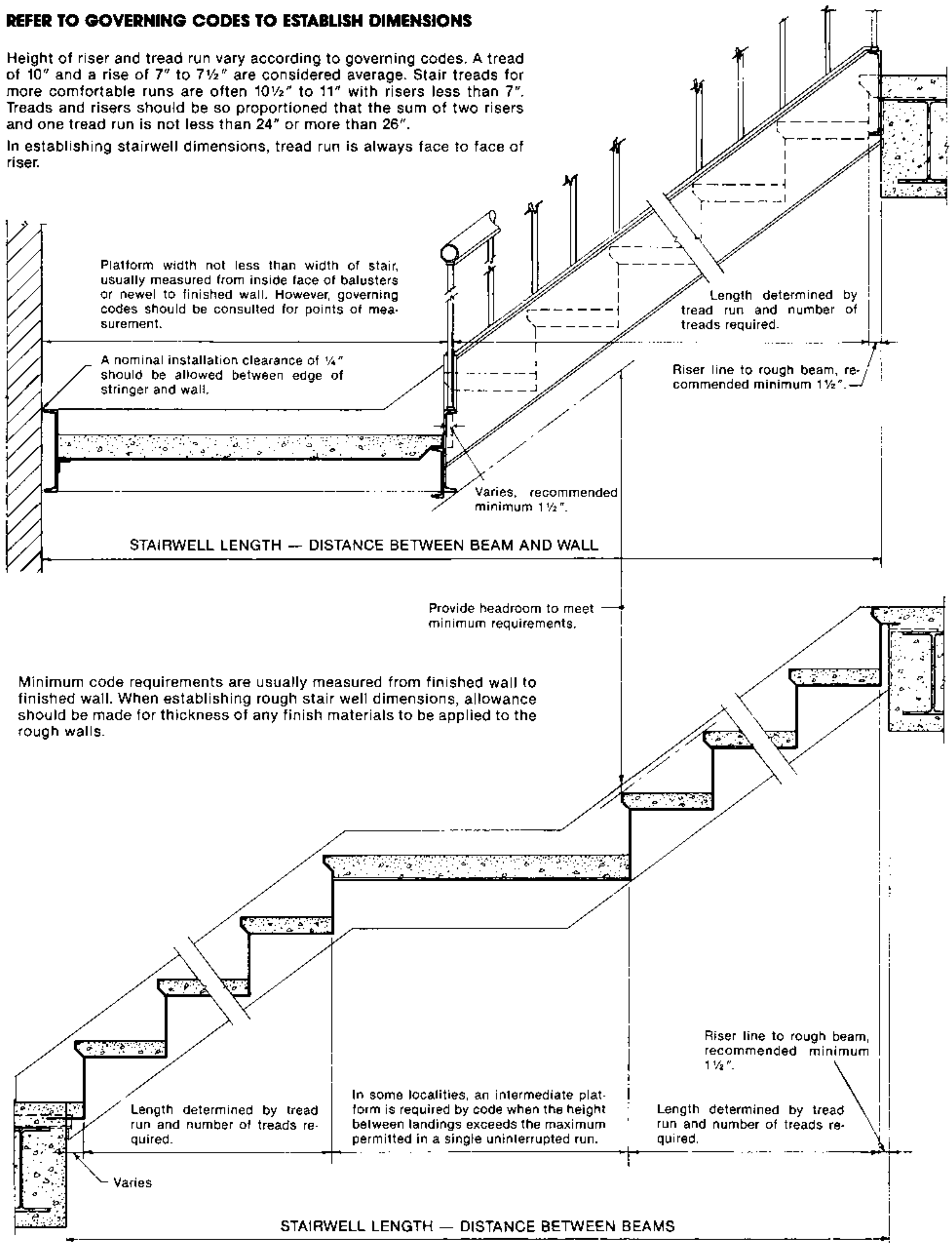


Fig. 7-3a. Tread Riser Stair Chart

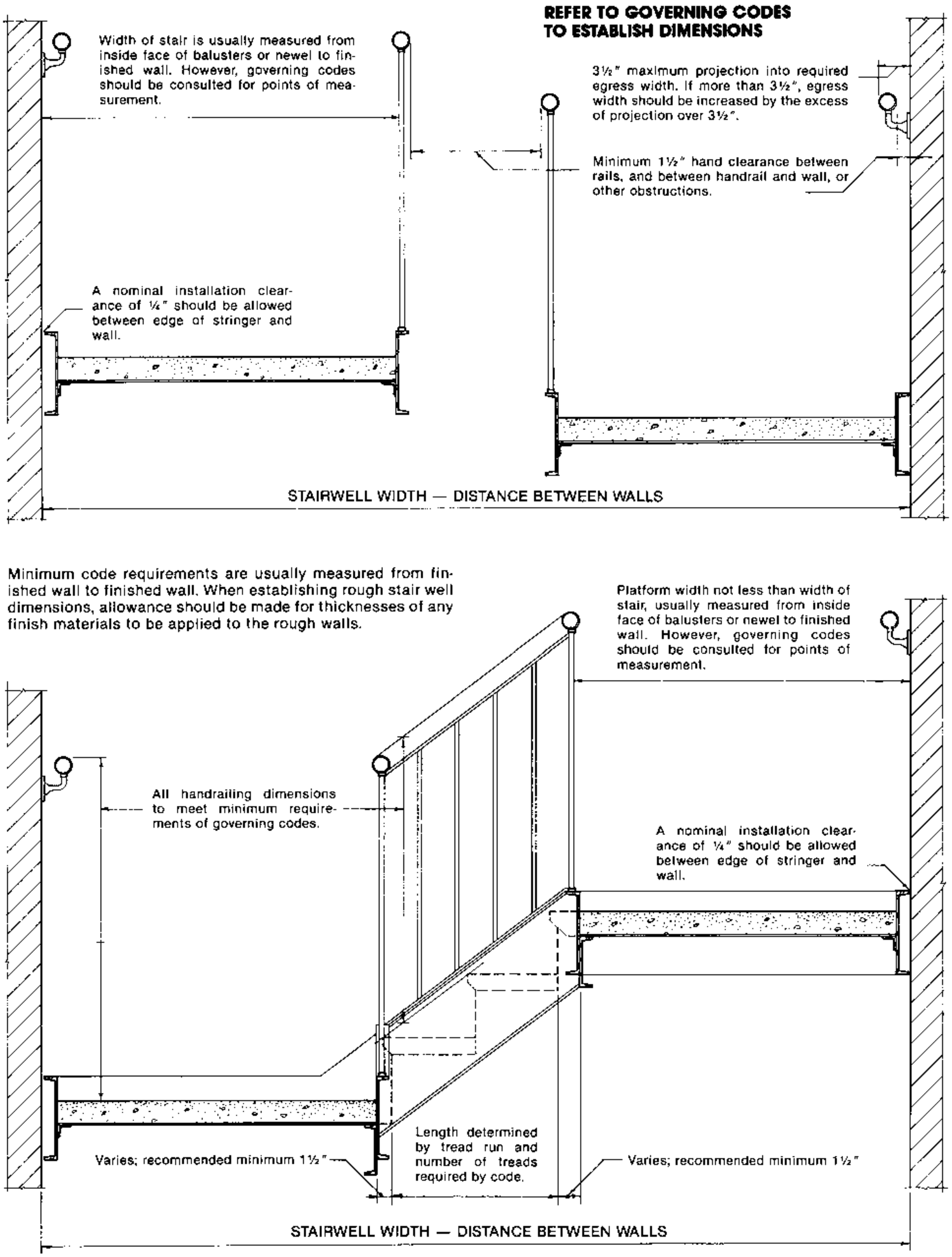


Fig. 7-3b. Tread Riser Stair Chart

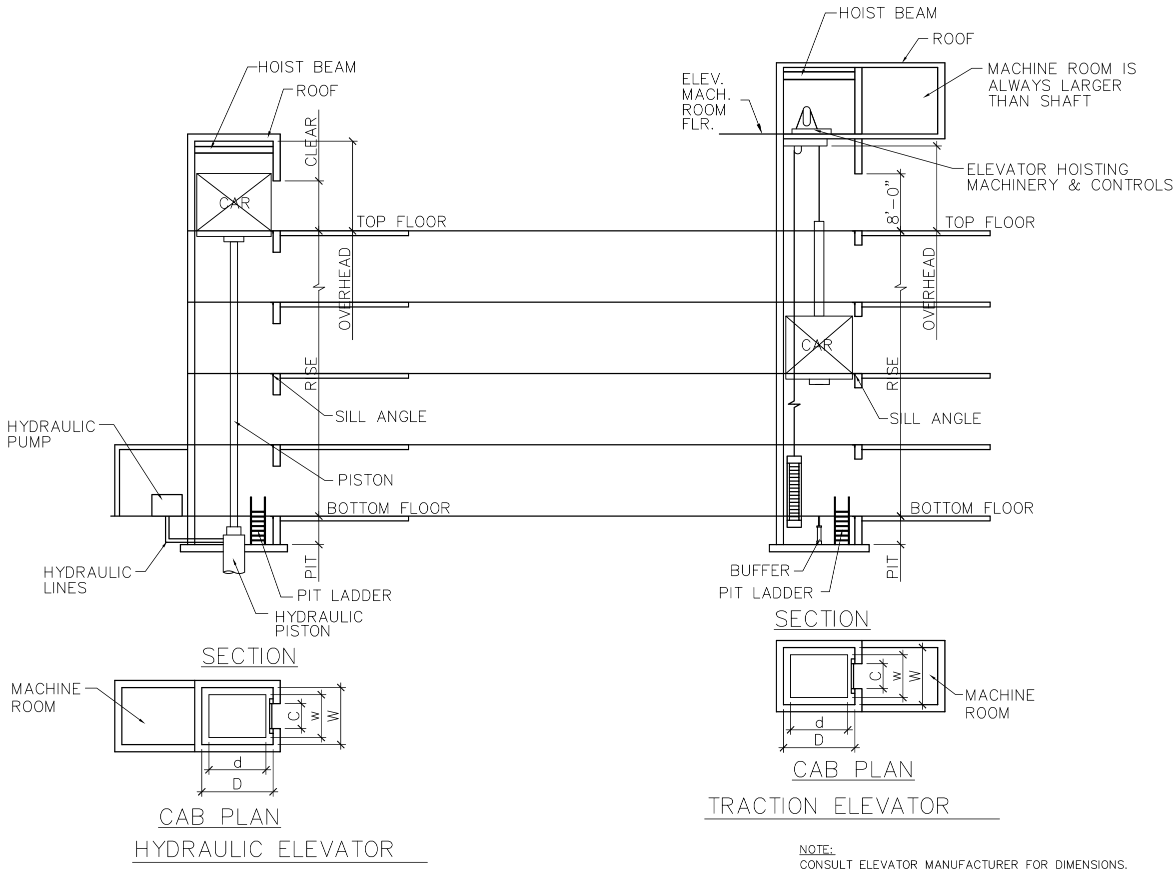


Fig. 7-5. Typical Cab Plans

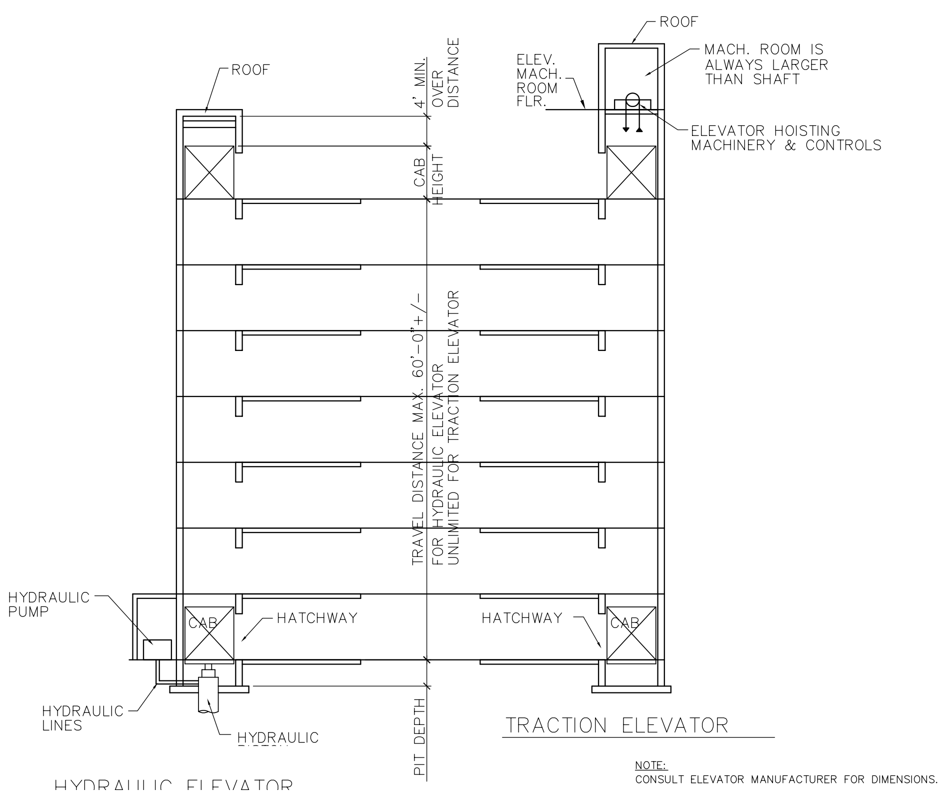


Fig. 7-4. Elevator Detail

Chapter 8
Corrosion Protection for Exposed Steel in Open-Deck Parking Structures

8.1 General Overview

The development and application of high-performance coating systems have contributed to increased use of structural steel framing for parking garages. Owners concerned with maintenance and life-cycle cost now have the option of providing steel framing with a variety of corrosion protection systems that offer basic life spans of three decades or more. Ultimately, the long-term performance of most corrosion protection systems for the structural steel depends on the overall durability of the concrete deck above and its resistance to leakage.

High-performance corrosion protection may be accomplished using painting or galvanizing systems. The alternative to a coating system for corrosion resistance, ASTM A588 or "weathering" steel, is not recommended for use in parking garages. The main drawbacks to uncoated A588 structural steel are:

- Good performance in a normal atmospheric environment, but not in a corrosive exposure that may develop if chlorides leak through joints or cracks in the concrete deck.

- The steel surface weathers to a very dark reddish brown, not an appropriate interior color if a bright finish is desired.

- During the initial weathering process (usually one to two years) the products of weathering that run off or get blown off the steel at the exterior may stain adjacent finishes, including concrete and parked vehicles.

- Interior surfaces not exposed to direct weathering may not develop a proper patina to protect the steel from deterioration.

The discussion that follows is intended to provide the designer with general direction for selecting a corrosion protection system for structural steel framing in parking garages. The design professional should become familiar with guidelines issued by organizations such as the Society of Protective Coatings (SSPC), the American Galvanizers Association (AGA) and the National Association of Corrosion Engineers (NACE). The corrosion protection system suppliers are a good source for technical information and assistance, and they can recommend proper systems for any application.

8.2 Environmental Factors

One classification of exposures for structural steel is published by the SSPC. The open parking garage exposure in middle and northern tier states probably falls between Zone I-B ("Exterior normally dry") and Zone 2-B ("Frequently wet by salt water - involves condensation, splash, spray or frequent immersion").

In open parking structures, not located within a corrosive atmosphere such as coastal fog, exposure for structural steel framing will be Zone 1-B for most of the steel surfaces, but more like Zone 2-B in the vicinity of any leakage around vertical drain lines, deck joints, and at cracks that fully penetrate the deck. Rate of breakdown of the coating system at these locations will depend on width of the deck penetration, drainage characteristics of the deck surface and intensity of the de-icing program involving the use of chlorides in the community.

8.3 High-Performance Coating Systems

8.3.1 Overview

Paint systems have been the most popular choice for corrosion protection of structural steel in parking garages. The paint system consists of a prescribed surface preparation for the steel usually followed by a two- or three-coat application of two or three paints. Standards and methods for surface preparation are published by the SSPC. Two standards should be noted: SSPC-SP 2, "Power Tool Cleaning to Bare Metal," and SSPC-SP 12, "Industrial Blast Cleaning," which is an intermediate level of cleaning between Commercial (SSPC-SP 6) and Near White Metal (SSPC-SP 10).

The cleaner the steel surface at the time of priming, the better the paint system will perform over the long term. The Commercial Blast Clean (SSPC-SP 6) is the most common surface preparation specified for parking garages. However, certain paints may have a low tolerance to variation in the surface condition. A Near White Metal Blast clean, SSPC-SP 10, is recommended for surfaces to receive an inorganic zinc-rich primer. Paint systems can be applied to steel surfaces cleaned by manual power tool (SSPC-SP3), but it is not recommended unless a limited performance, non-zinc primer rated as "moderate duty, durable" is utilized. It does not make much sense to specify this lower quality cleaning method together with a high-performance paint system for a potentially corrosive environment. Most structural steel fabricators involved in parking garage work have access to blast cleaning equipment.

The purpose of surface preparation, essential for any paint system, is to clean and roughen the steel surface. Loose or foreign matter on steel at the time of painting can cause premature failure of the coating system. A surface profile of $1\frac{1}{2}$ to $2\frac{1}{2}$ mils is an adequate roughness for just about any coating. A good steel surface is one that is rough and clean of loose matter but easily wetted by the paint, a combination that provides good coating adhesion.

Zinc-rich paints should have a minimum 80 percent zinc dust by weight. As noted earlier, inorganic zinc-rich paint should be applied only to the cleanest of steel surfaces. Compared to organic zinc-rich paint, however, inorganic zinc-rich paint has a lower minimum cure temperature (O °F to 40 °F vs. 35 °F to 50 °F) and better resistance to salt. For industrial applications, inorganic zinc-rich paint is often used with no topcoat. As part of a multi-coat paint system, inorganic zinc-rich paints have been the most popular primer for parking garage applications, especially in combination with the epoxy top coat and urethane finish coat. This three-coat system has been the high-performance paint system of choice for steel parking structures.

Environmental regulations restrict the level of solvent emissions from painting operations in some areas. Many coating types are formulated with low solvent volatile organic compounds (VOC). Coating specifications should include requirements for coatings to be "VOC compliant" for the area in which they are applied.

Coating removal processes are also regulated by the EPA. Removal of lead based paint and the generation and control of dust from abrasive blasting is strictly regulated. High performance zinc rich primers are recommended because they are permanent for the life of the structure. Future maintenance is performed on the coatings rather than the substrate.

It has been well established that structural steel to be encased in concrete does not have to be primed or otherwise coated. Encased iron and steel in place for 75 years and longer, when exposed during demolition or recycling work, is almost always found to be in good condition. There are, of course, exceptions. Concrete whose ingredients are high in chlorides can cause corrosion of the encased structural steel (and any reinforcing steel present).

Another oversight noted in some steel specifications is the assignment of responsibility for field painting. The structural steel fabricator is not a field painter. Normal field touch-up of shop painting should be assigned by contract to the field painting subcontractor.

Quality assurance of shop painting is best achieved by timely shop inspection of both surface preparation and the applied wet film. Considering that a high-performance paint system for a parking garage may comprise 10 to 20 percent of the erected steel cost, the owner/developer has a strong incentive for assuring that the coating system specified and purchased is, in fact, supplied and properly applied and inspected.

The paint specifier has a choice of specifying a particular brand of paint system (no substitutions allowed), issuing a performance specification, or specifying a brand with an "or equal" clause. The burden of proof for the "or equal" is on the steel fabricator and/or paint supplier to assure the specifier that a given substitution is, indeed, equal and acceptable. One method is to require performance verification from an independent testing laboratory. A sample of a generic performance specification is presented in Appendix C. Permitting acceptable substitutions gives the steel fabricator some flexibility while assuring the specifier that a paint system of sufficient quality will be supplied. For a multi-coat, high-performance system, all paints should be supplied by the same manufacturer, if possible. The concept of "shared responsibility" will only complicate the resolution of any subsequent question or dispute regarding paint performance.

How often must steel be touched up or repainted? Life-cycle cost may actually be decreased if maintenance is performed on a schedule such that the primer remains intact and only the top and/or finish coats need be replenished. Because the high performance zinc rich primers practically eliminate undercutting corrosion, the period of addressing issues through visible touch-up spans many years. The life of the paint system will depend on the quality of the total coating system, on the exposure, on the quality of the concrete deck and on the adherence of the owner to a prescribed maintenance program. Designers should consult the major paint suppliers, many of whom have information on life-cycle costs.

8.3.2 Selection

8.3.2.1 Factors That Affect Cost and Performance

A coating system should provide maximum performance at the lowest cost. In making the most proper choice, a number of performance factors should be considered; among them are the following:

1. Functional Requirements—In most environments, coatings are a requisite for the protection of steel from corrosion. Exposed steel in parking garages is often visible to the public, making maintenance of its appearance (the gloss and color retention) an important issue.

2. Service Life of Both Coatings and Structures—The service life of a high performance coating system, properly applied and with periodic maintenance, can be expected

to provide suitable protection over the life of the structure. Optimum service life is best achieved through conformance with acceptable specifications that prescribe surface preparation and coating application parameters.

3. Coating System Quality—As previously noted, the type of coating selected is an important factor for both its performance and cost. Normally, coating material accounts for 10 percent to 15 percent of the system's total cost. So, sacrificing quality for cost on the coating system is not a wise decision. In all cases, a paint system of highest performance should be specified. The performance characteristics of the selected primer should be evaluated against alternative primer systems. Also, coatings performance must be specified. Do not assume that the color and gloss retention of all polyurethane coatings are equal. Weatherability can vary across a wide range depending on how a coating is formulated. The Society for Protective Coatings (SSPC) Paint Specification No. 36—Two-Component Weatherable Aliphatic Polyurethane Topcoat, Performance Based is included in its entirety as Appendix B as a guide for topcoat performance.

4. Quality of Surface Preparation and Application—In virtually all systems that use high-performance coatings (e.g., ethyl silicate zinc-rich, epoxy polyamides, polyurethanes), their most costly component is surface preparation. The degree of surface preparation achieved is a major determinant of the ultimate performance of the coating system.

Initially investing in a superior surface preparation will almost always result in an increased service life. Either an SSPC-SP6 commercial blast cleaning, or an SSPC-SP10 near white metal blast cleaning, is recommended for use in parking structures.

a) SSPC-SP6 "Commercial Blast Cleaning"—Commercial blast cleaning defines a more thorough, but not quite perfect, degree of blast cleaning. It is a minimum specification that is used with coating systems of higher performance, yet less forgiving of surface imperfections. During cleaning, all rust, mill scale, and other detrimental matter is removed. Staining that resulted from previously existing rust and mill scale is limited to no more than 33 percent of each unit area of surface, as defined under SSPC-SP6, Section 2.6 of SSPC Painting Manual Volume 2.

   The advantage of commercial blast cleaning is generally lower cost than SSPC-SP10.

b) SSPC-SP10 "Near-White Metal Blast Cleaning"—This specification limits random staining to no more than 5 percent of each unit area of surface. This cleanliness level is generally used when the expense is justified by the severity of the anticipated service environment. Near-White Metal Blast Cleaning is frequently specified in combination with inorganic zinc-rich coatings.

   Parking structures are not anticipated to will require the use of a surface preparation that is more stringent than the "Near-White Blast Cleaning."

5. Maintenance Program—A well-established maintenance program will create a substantial increase in the service life of the parking structure. This is a "common sense" approach to asset management that holds true for all components of a parking structure. The magnitude of maintenance expenditure and the interval between such expenditures, depends on the initial coating choice and the established type of maintenance program.

6. Determining Coating Costs—To assist in making an informed decision, designers, specifiers, and owners of garages, require information on comparative costs and the expected service life of alternative coating systems. It is relatively easy to compute initial costs. Shop-application coating costs are normally include: material, surface preparation, application, inspection and overhead. For more precise estimates, individual shops can, determine the costs of labor, materials, and other items with greater precision. Maintenance painting and touch-up requires a case-by-case evaluation to determine painting costs.

When specifying coating system performance, performance factors such as gloss, weatherability, graffiti resistance (chemical resistance), abrasion resistance, low temperature cure, etc., may increase coating material costs. When comparing alternative coating systems, compare the cost of comparably performing coatings.

8.3.2.2 Recommended Coating Systems

1. a) SSPC-SP 6 or SSPC-SP 10
   b) moisture cure urethane(mcu) or epoxy zinc-rich primer
   c) high build aliphatic polyurethane finish coat (two component)

2. a) SSPC-SP 6 or SSPC-SP 10
   b) mcu or epoxy zinc rich primer
   c) mcu intermediate coat containing micaceous iron oxide
   d) aliphatic polyurethane finish coat (one or two component)

3. a) SSPC-SP6 or SSPC-SP 10

Table 8-1 Recommended High Performance Coating Systems

COATING SYSTEM DESCRIPTION AND APPLICATION PARAMETERS

System	1	2	3	4
Description	MOISTURE CURE URETHANE OR EPOXY ZINC-RICH PRIMER/ HIGH BUILD ALIPHATIC POLYURETHANE FINISH COAT (TWO COMPONENT)	MOISTURE CURE URETHANE OR EPOXY ZINC-RICH PRIMER/ MOISTURE CURE URETHANE INTERMEDIATE COAT/ ALIPHATIC POLYURTHANE FINISH COAT (ONE OR TWO COMPONENT)	MOISTURE CURE URETHANE OR EPOSY ZINC-RICH PRIMER/ HIGH BUILD EPOXY INTERMEDIATE COAT/ ALIPHATIC POLYURETHANE FINISH COAT (TWO COMPONENTS)	ETHYL SILICATE INORGANIC ZINC-RICH PRIMER/ HIGH BUILD EPOXY INTERMEDIATE COAT/ ALIPHATIC POLYURETHANE FINISH COAT (TWO COMPONENT)
Benefit	This two-coat system will provide application cost savings with the elimination of an intermediate coat and will provide acceptable corrosion resistance properties through (1) galvanic protection to the substrate and (2) application of a durable high build color retentive finish coat.	This three-coat system will provide long term protection to the structure through (1) galvanic protection to the substrate, (2) encapsulation with a moisture cure urethane intermediate coat and (3) application of a durable color retentive finish coat.	This three coat system will provide long term protection to the structure through (1) galvanic protection to the substrate, (2) encapsulation with a high build epoxy intermediate coat and (3) application of a durable color retentive finish coat.	This three coat system will provide long term protection to the structure through (1) galvanic protection to the substrate with outstanding abrasion resistance prior to subsequent overcoating applications, (2) encapsulation with a high build epoxy intermediate coat and (3) application of a durable color retentive finish coat.
Surface Preparation	SSPC-SP 6 or SSPC-SP 10. The selection of either cleanliness level should be made after an analysis of cost vs. anticipated service severity.	SSPC-SP 6 or SSPC-SP10. The selection of either cleanliness level should be made after an analysis of cost vs. anticipated service severity.	SSPC-SP 6 or SSPC-SP 10. The selection of either cleanliness level should be made after an analysis of cost vs. anticipated service severity.	SSPC-SP 10
Primer	Moisture Cure Urethane Zinc-Rich Primer applied at 3.0-4.0 mils dry film thickness (dft).or epoxy zinc rich	Moisture Cure Urethane Zinc-Rich Primer applied at 3.0-4.0 mils dft or epoxy zinc rich	Moisture Cure Urethane applied at 3.0-4.0 mils dft or epoxy zinc rich	Ethyl Silicate Inorganic Zinc-Rich applied at 3.0-4.0 mils dft
Intermediate Coat		Moisture Cure Urethane containing micaceous iron oxide applied at 4.0-6.0 mils dft.	High Build Epoxy applied to 4.0-6.0 mils dft.	High Build Epoxy applied at 4.0-6.0 mils dft
Topcoat	High Build Aliphatic Polyurethane (two-component) applied at 4.0-6.0 mils dft.	Aliphatic Polyurethane finish coat applied at 2.0-4.0 mils dft. The topcoat can be a one-component aliphatic Moisture Cure Urethane or a two-component aliphatic polyurethane.	Aliphatic Polyurethane finish coat (two-components) applied at 2.0-4.0 mils dft.	Aliphatic Polyurethane finish coat (two-component) applied at 2.0-4.0 mils dft

b) mcu or epoxy zinc-rich primer
c) high build epoxy intermediate coat
d) aliphatic polyurethane finish coat (two component)

4. a) SSPC-SP 10
b) ethyl silicate inorganic zinc-rich primer
c) high build epoxy intermediate coat
d) aliphatic polyurethane finish coat (two component)

NOTE: There are coatings systems of varying generic types that may also be considered for use in parking structure applications. An analysis of service life history, application properties and conformance to environmental regulations is a prerequisite of any product or system usage.

In garages that utilize interior revenue collection, the lower flange of wide span girders in the collection area should receive a second color coat to protect the bottom flange from sulfuric attack from exhaust gasses of idling engines.

8.3.2.3 Moderate Performance Coating Systems

Long-term corrosion protection is a critical component in the construction of a parking structure. Most parking structure owners desire a 30-year minimum performance life for a coating system. Therefore, high performance coating systems are generally recommended for parking structure applications. However, situations do exist where a limited structure life or the necessity of minimizing initial construction costs mandates utilizing a lower cost moderate performance coating system. In such cases a two-coat paint system utilizing a mastic primer over wire brush cleaned steel (SSPC SP-3) and a polyurethane top coat may be specified. A mastic primer provides only barrier protection to corrosion as opposed to the sacrificial protection of a zinc-rich primer and allows for corrosion undercutting of the paint system. The selection of a moderate performance system should be made with the full knowledge and understanding of the owner and architect as to the increase in future maintenance costs.

8.3.2.4 Low-VOC Alternatives

To meet future VOC requirements, the systems 1 through 4 listed in Section 8.3.2.2 are available for commercial use at lower VOC levels, as needed.

While low VOC systems have demonstrated good long-term service life, the manufacturer/supplier must demonstrate the suitability of shop application properties, as well as citing the products specific field usage.

Notes on Coating Specifications:

1. Thickness recommendations are typical of many coating suppliers and are presented here as minimums. Maximum allowable or specified coating thickness should not exceed the manufacturer's recommendations.

2. Gusset plates and faying surfaces are best protected by the shop applied primer. Most zinc-rich primers are rated for friction connections in accordance to the Research Council for Structural Connections (RCSC) as Class A or Class B. Should friction connections be utilized in the construction confirm the rating of the specified primer.

3. Field touch-up of erection damage, block-ours, primed gusset plates and fasteners should utilize system 1, the moderate duty surface tolerant system.

4. Mechanically galvanized fasteners are recommended

8.4 Galvanizing

All-galvanized steel frame parking structures made their first appearance during the early 1980's. Hot-dip galvanized structural steel can be competitive with steel protected by a high-performance paint system.

Hot-dip galvanizing is a shop-applied coating that provides a unique combination of properties. Galvanizing is different from painting in that a progression of zinc alloy layers are metallurgically bonded to the base metal. Penetration of contaminants through this type of coating and the resulting under film corrosion are less likely. Galvanizing protects steel in two ways:

1. As a barrier coating which seals the base metal from the corrosive action of the environment; and

2. By sacrificial action of the zinc, which tends to "repair itself" when damage or minor discontinuities occur in the coating.

Galvanized coatings have a hardness greater than the steel itself and thus have exceptional resistance to damage from impact and abrasion. Also, the coating tends to be thickest at comers and edges, often locations of minimum thickness of paint systems.

Because hot-dip galvanizing is accomplished by total immersion (when maximum member size of the dipping tank is not exceeded), all surfaces of the steel assembly become coated and protected. Since the zinc will not metallurgically bond to unclean steel, poor quality galvanizing is immediately apparent as the work is withdrawn from the molten zinc bath and adjustments can be made on the spot.

Galvanizing tanks located in many regions of the United States are now capable of handling members exceeding 60 ft in length in a single dip. In a galvanizing dip tank as short as 35 ft, beam sections up to 63 ft long can be galvanized by "double dipping". For sections exceeding 63 ft, surfaces in the center area not coated by double dipping can be metallized. Design professionals should consult with local galvanizers when developing costs of galvanized coatings, especially if beams are longer than available dipping tanks. Designers should also bear in mind that galvanizing of ASTM A490 structural bolts is not permitted, therefore ASTM A325 bolts should be specified.

Hot-dipped galvanized steel is initially shiny, but a natural "weathering" process, which does not reduce the protective capacity of the coating, tends to slowly transform the shiny surface of a duller brownish-gray tint. If a bright uniform finish or color is desired, a compatible topcoat of paint will have to be applied, an additional cost for the galvanized steel option for corrosion protection. The combination of composite, galvanized wide flange beams and galvanized

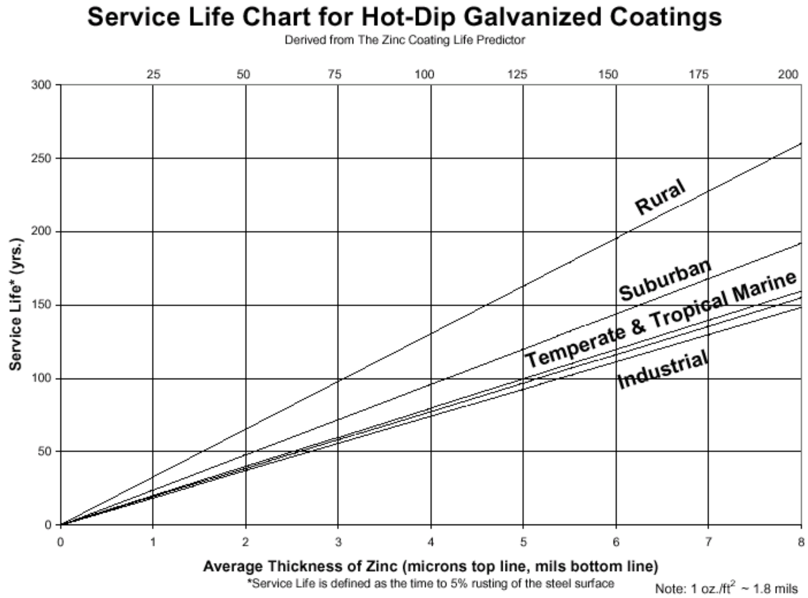


Fig. 8-1. Service Life for Galvanized Coatings (Source: American Galvanizing Association)

metal deck requires the removal of the coating on the top flange at the location of shear studs. The galvanizer, at the fabricator's request, however, can easily mask off areas where shear studs are to be welded. Research is currently being conducted to develop a method of "shooting" shear studs directly into the galvanized beam without the need to remove the galvanized surface.

The level of galvanizing specified for structural members is typically a minimum thickness of 3.9 mils. A galvanizing performance model available through the web site of the American Galvanizing Association (www.galvanizeit.org) is designed to predict, for a given thickness of galvanizing, the time that will pass before the surface corrosion reaches 5 percent, using meteorological data specific to any location. The model will also predict the thickness of galvanizing for a desired performance life. Figure 8-1 presents the service life in various types of environments. In selecting a corrosion protection system, the structural engineer is encouraged to compare the price, performance and aesthetic appeal of various options.

Chapter 9
Life-Cycle Costs of Steel-Framed Parking Structures

Investigation of the life-cycle cost of steel-framed parking structures in comparison to other framing materials indicates that significant life-cycle cost savings may be possible through the use of a steel framing system for an open-deck parking structure. These results correlate to an independent study of a steel-framed parking structure that was performed by Hill International for the Port Authority of New York and New Jersey. That study is documented in the April 2000 issue of AISC's *Modern Steel Construction* magazine (reprints available through AISC).

Life-cycle costs are a function of the initial construction cost, routine maintenance, and any future restoration costs of the structure. For more information and guidance on determining on Life Cycle Costs for parking structures, contact the AISC Steel Solutions Center at solutions@aisc.org.

Chapter 10

Checklist for Structural Inspection of Parking Structures

Decks

❒ Are there any cracks? Do they leak?

❒ Is the surface sound, or are there areas of surface scaling?

❒ Does a chain-dragging test reveal a hollow sound in any areas?

❒ Is there any evidence of concrete delamination?

❒ Is there any evidence of corrosion of reinforcing steel or surface spalling?

❒ Are there any signs of leakage? Describe conditions and note locations.

❒ If there is a traffic bearing membrane, does it have any tears, cracks or loss of adhesion?

❒ Are there low spots where water ponding occurs?

❒ Are there water stains on the underside (soffit) of the deck?

❒ Has the concrete been tested for chloride-ion content? When was it last tested?

❒ Are records of previous inspections available?

Steel Beams and Columns

❒ Are there any signs of corrosion on the beams or columns? Is the corrosion a surface effect or is there a significant loss of section?

❒ Is repainting required?

❒ What is the condition of the interface or attachment point between the steel members and the concrete deck?

❒ Is there any staining that would indicate deck leakage adjacent to the steel member?

Stair and Elevator Towers

❒ Are there any signs of a leaking roof?

❒ Are there any cracks in the exterior finish?

❒ Are there any signs of corrosion-related deterioration of stairs or railings?

❒ Are any other corrective actions required?

Expansion Joints

❒ Are there leaks through isolation-joint seals?

❒ Are leaks related to failure of the seals or the adjacent concrete?

❒ Could the cause be snowplows?

❒ What type of isolation joint/expansion joint seal is installed?

❒ Who is the manufacturer?

❒ Is there a warranty in force?

❒ Consult the manufacturer for repair recommendations if applicable.

Joint Sealants

❒ Are there any signs of leakage, loss of elastic properties, separation from adjacent substrates or cohesive failure of the sealant?

❒ Are there failures of the concrete behind the sealant (edge spalls)?

Exposed Steel

❒ Is there any exposed steel (structural beams, handrails, door frames, barriers, cable, exposed structural connections)?

❒ Is corrosion visible? Is it surface corrosion or is there significant loss of section?

❒ Is repainting required?

❒ What is the condition of attachment point and surrounding concrete?

Drains

❒ Are drains functioning properly? When were they last cleaned?

❒ Are the drains properly located so that they receive the runoff intended?

❒ Are seals around the drain bases in good condition?

Previous Repairs

❒ Are previous repairs performing satisfactorily?

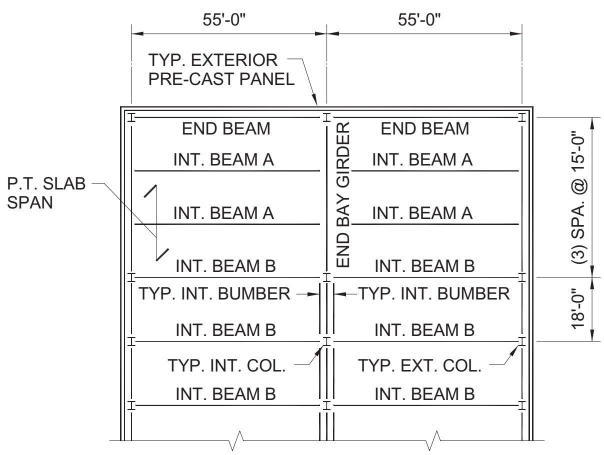
❒ Are the edges of previous patches tight?

❒ Do the patches sound solid when tapped?

Source: Mr. David Monroe, President, Carl Walker Construction. Originally published in Parking, November, 2001

Appendix A1
Example: Post-Tensioned Deck Parking Garage

This exercise is an illustration of typical calculations to be performed in the design of a post-tensioned deck parking structure. The design examples are presented using both LRFD and ASD design procedures.



PARTIAL PLAN - POST-TENSIONED DECK GARAGE

NOTE: THIS PARTIAL PLAN ILLUSTRATES THE TYPICAL COMPONENTS TO BE DESIGNED FOR A POST-TEMSIONED DECK

Geometry

Ramp Width = 55 ft

Beam Spacing:

- 15 ft c/c for end bays. Note that this garage has end bay parking requiring a 45 ft total end bay.
- 18 ft c/c for typical beam spacing.

Design Loading

Dead Loads:

5 in. post-tensioned slab	63 psf
structural steel	<u>10 psf</u>
	73 psf

Note: The slab thickness selection will be shown in the slab design.

Live Loads:

Gravity	50 psf
Wind	Note "A"
Seismic	Note "A"

Note "A" refers to local codes for wind + seismic loads.

Component Design

Post-Tensioned Slab:

The thickness of a post-tensioned slab is a function of its span. A span to depth ratio of 45 for parking structures almost always satisfies both structural and serviceability requirements.

Span = 18 ft

Required slab thickness = $\dfrac{18 \text{ ft } (12 \text{ in./ft})}{45} = 4.8 \text{ in.}$

Use 5 in. thickness.

(Refer to Table 3.5 for minimum slab thickness vs. span.)

Reinforcing Requirements:

The design of a post-tensioned slab is complex and is beyond the scope of this design guide. However there are many software packages available that will simplify the design process. Also refer to Table 3.5 for typical reinforcing sizes and details. For this example the following reinforcing should be used:

Post-Tensioning Tendon Spacing:

From Table 3-5 using the clear span of slab = 18 ft and slab thickness of 5 in.

Spacing of structural tendons = 24 in.

Since the width of the slab is 55 ft the required number of tendons is:

$$No. = \frac{55\text{ft} \times 12}{24} + 1 = 28.5\text{ft}$$

Use 29 tendons.

Temperature Tendons:

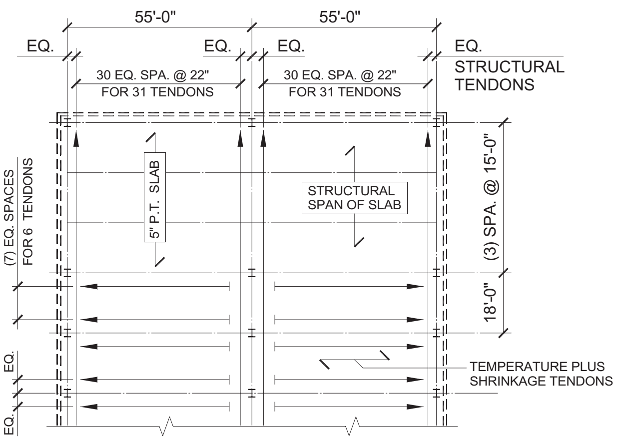
From Table 3-5 the maximum temp. spacing = 33 in. Therefore the minimum required number of tendons is:

$$No. = \frac{18\text{ft} \times 12}{33} = 6.5$$

Use 7 temp. tendons.

Mild Reinforcing Steel

Once again referring to Table 3-5



PARTIAL PLAN SHOWING POST-TENSIONING TENDONS
LAYOUT FOR POST-TENSIONING SLAB DECK

THIS PARTIAL PLAN ILLUSTRATES A TYPICAL TENSIONING LAYOUT
SLAB IS DESCRIBED AS 2-WAY POST-TENSIONED (BOTH STRUCTURAL
SPAN DIRECTIONS AND TEMPERATURE PLUS SHRINKAGE DIRECTION)

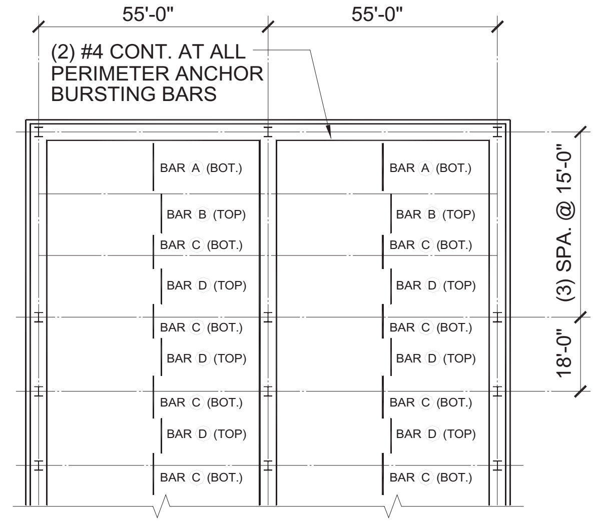
DENOTES STRESSING END OR LIVE LOAD
DENOTES DEAD END

The four typical bars required are:

Bar "A" #4 × 13 ft-4 in. @16 in.
Bar "B" #5 × 9 ft-6 in. @16 in.
Bar "C" #4 × 9 ft-6 in. @16 in.
Bar "D" #4 × 9 ft-6 in. @16 in.

Refer to Figure 3-10, which illustrates placement and location of reinforcing bars.

Please also note: Always include two #4 bars continuous at all anchorages.



PARTIAL PLAN - MILD REINFORCING STEEL
LAYOUT FOR POST-TENSIONING SLAB DECK

THIS PARTIAL PLAN ILLUSTRATES THE TYPICAL MILD
REINFORCING STEEL LAYOUT.

BAR	SIZE	LENGTH	SPAN
A	#4	13'-4"	16"
B	#5	9'-6"	16"
C	#4	9'-6"	16"
D	#4	9'-6"	16"

Span length, L = 55 ft
Beam spacing, s = 15 ft
Slab thickness, t_o = 5 in.
Concrete, f_c' = 5 ksi
n = 7
Steel, F_y = 50 ksi
Studs, ¾ in. dia. × 3 in.

Loading—Service

Dead Loads:

5 in. post-tensioned slab 5/12 (150) = 63 psf
self-weight of beam—assume 94 lb/ft

Uniform Dead Load:
w = 15(0.063) + 0.094 = 1.04 k/ft

Live Load:

code: 50 psf
uniform live load = 18(0.05 ksf) = 0.9 k/ft

BEAM DESIGN

Beam design is presented for both the LRFD and ASD design procedures.

LRFD DESIGN PROCEDURE

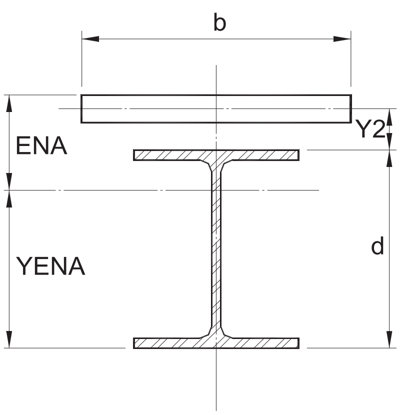
INTERIOR BEAM A

Design Loads

Load Cases (only relevant cases listed)
$1.4D = 1.4(1.04\text{ k/ft}) = 1.46\text{ k/ft}$
$1.2D + 1.6(\text{L}) = 1.2(1.04) + 1.6(0.9) = 2.7\text{ k/ft}$
Live only $1.6L = 1.44\text{ k/ft}$

Bending Moments (Unshored)
$M_u = 2.7(55)2/8 = 1020$ k/ft (factored)
$M_{DL} = 1.04(55)2/8 = 394$ k/ft (service)
$M_{LL} = 0.9(55)2/8 = 340$ k/ft (service)

Check Section and Determine Properties



Assume $a = 2$ in. $Y2 = 5\text{ in.} - (2\text{ in.}/2) = 4$ in.

From composite beam tables for $F_y = 50$ ksi and $Y2 = 4$ in.

Possible selections

W27×84 or W24×76

Try W24×76

From composite beam tables

$Y1 = 0.34$
$Q_n = 814$ kips
$\phi M_n = 1180$ k-ft

Comparing $Y2$ for $\Sigma Q_n = 814$ kips
$b \le 2 \times L/8 = 2 \times 55\text{ ft}/8 = 13.75$
$\le$ spa. = 18 ft
$b = 3.75 \times 12 = 165$ in.

$$a = \frac{\Sigma Q_n}{0.85 f_c' b} = \frac{814}{0.85(4)(165)} = 1.45\text{ in.}$$

$Y2 = 5 - 1.45/2 = 4.28$ in.

Compute number of studs required

$Q_n = 26.1$ kips (Table 5.1)

Number of studs (2) $\Sigma Q_n / Q_n = 2(814)/26.1 = 62.6$

Use 63, ¾ in. dia. shear stud connectors.

Construction Phase Check

A construction phase live load will be assumed. From the *LRFD Specification* (Section A4.1) the relevant combinations are:

$1.4(D) = 1.46$ k/ft
$1.2(D) + 1.6(L) = 1.2(1.04) + 1.6(18\times0.02) = 1.82$ k/ft
$M_u = 1020$ k-ft

From composite beam tables for a W24×76 with $F_y = 50$ ksi and assuming adequate lateral support is provided by forming system (very important to confirm plus no torsional loading)

$\phi M_n = \phi M_p = 1180\text{ k-ft} > 1020\text{ k-ft}$

Service Load Condition

Assume that the fresh concrete load moment is equal to the service dead load moment, with I_{xx} of 2100 in.^{4}

$$\Delta_{DL} = \frac{394(55)^2}{161(2100)} = 3.5\text{ in.} \quad \text{Too High}$$

Switch to W27×84

$$\Delta_{DL} = \frac{394(55)^2}{161(2850)} = 2.59\text{ in.}$$

Specify a camber of 2½ in.

From composite beam tables use $Y2 = 4$ (from above)

$Y_1 = 3.44$ in.
$Q_n = 456$ kips
$\phi M_n = 1270$ k-ft
Number of studs = 2(456)/26.1 = 35 studs
$\phi M_n = 1270\text{ k-ft} > 1097\text{ k-ft}$

For W27×84 with $Y2 = 4$ and $Y1 = 3.44$ the lower bound moment of inertia can be found in the lower bound moment of inertia tables: $I_{lb} = 4860$ in.^{4}

$$\Delta_{LL} = \frac{348(55)^2}{161(4860)} = 1.34 = \frac{L}{490} < \frac{L}{240} \quad \text{ok}$$

Check Shear

V_u = 2.7(55/2) = 7.5 kips
ϕV = $\phi(F_{yw})(A_w)$ = 0.6(50)(26.71×0.46)
= 368 kips > 80 kips ok

Final Selection

Use W27×84 F_y = 50 ksi with 36, ¾ in. dia. shear stud connectors.

(18 shear stud connectors on each side of midspan)

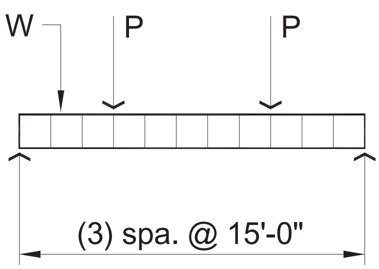
END BAY GIRDER

Design Parameters

Note: In order to minimize cracking, adjacent ramps should not be connected. Thus, this girder should be non-composite.

Girder Span = 45ft (Ref. to Figure A1-1A).

Concentrated Loads at 1/3 points



Calculate Factored Load (Concentrated + Uniform Load)

P_D = 15(0.063)55(1.2) 62.4 kips
P_L = 15(0.05)55(1.6) = <u>66 kips</u>
P_{D+L} 128.4 kips

Unif ≈ (0.2 k/ft)(1.2) = 0.24 k/ft

Calculate Bending Moments + Shears (Factored & Service)

M_u = 0.24(45)2/8 + 128.4(15) = 1986 k-ft (factored)
Equiv. Unif. Ld. = 2.67(128.4) + 0.24(45) = 354 kips
(Table Pg. 4-189)
V_u = 0.24(45/2) + 128.4 = 133.8 kips
M_D = 15(15×55×0.063) + 0.2(45)^2/8 = 830 k-ft (service)
M_L = 15(15×55×0.05) = 619 k-ft (service)

Enter factored load table for F_y = 50 ksi with a
$\phi W_c/L \geq$ 354 kips
For W27×178 $\phi W_c/L$ = 370^K > 354 kips ok

Check service load deflection

$$\Delta_{DL} = \frac{830(45)^2}{161(6990)} = 1.5 \text{ in.}$$

$$\Delta_{LL} = \frac{619(45)^2}{161(6990)} = 1.11 = \frac{L}{484} < \frac{L}{240} \quad \text{ok}$$

Final Selection

Use W27×178 F_y = 50 ksi camber = 1½ in.

BUMPER RAIL

Geometry

Span ≈ 18ft

Design Load

P = 10,000 lb

Note: Codes do not define the bumper load as service or ultimate. Also, the barrier member can fail in bending yet still restrain an automobile.

Calculate Bending Moments

$$M = \frac{P(L)}{4} = \frac{10(18)}{4} = 45 \text{ k-ft}$$

Calculate Min S_y

$$\text{Min } S_y = \frac{45(12)}{0.66(46)} = 17.8 \text{ in.}^3$$

Use HSS 10×6×5/16

S_y = 17.8 in.^{3}

TYPICAL INTERIOR COLUMN

Design Parameters

F_y = 50 ksi

Floor-to-floor height = 12 ft

Column is braced in X and Y − axis $\therefore$ $K_y = K_x = 1.0$

Note: Most codes permit a 20 percent live load reduction, for members supporting more than two floors, however example is one story. Estimated total steel weight approximately 9 psf.

Column Grid—Typical

55 ft × 18 ft

Calculate Design Loads

P_u = 18 ft × 55 ft (1.2 (0.072)+1.6(0.04)) = 149 kips

Select Trial Section

Try W12×40 (12 in. × 8 in. for connection)

ϕP_n = 356 kips > 149 kips

Use W12×40

ASD DESIGN PROCEDURE

Bending Moments (Unshored)

Construction Loads

$M_D = 1.04(55)^2/8 = 393$ k-ft

Loads applied after concrete is hardened

$M_L = 0.9(55)^2/8 = 340$ k-ft

Maximum Moment

$M_{max} = M_D + M_L = 393+340 = 733$ k-ft

Maximum Shear

$V = (1.04+0.9)55/2 = 53.4$ k

Effective Width of Slab

$b = 1/4$ (L) = $1/4(55 \times 12) = 165$ in.
$b = s = 15 \times 12 = 180$ in.

Required Section Modulus (F_y = 50 ksi):

For M_{D+L}

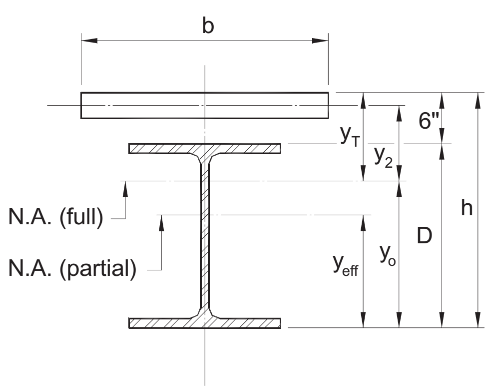
$S_{tr} = 733(12)/33 = 267$ in.^{3}

For M_D (Make sure about top flange bracing.)

Example assumes full top flange bracing.

$S_s = 393(12)/33 = 143$ in.^{3}

Select Section and Determine Properties



$Y2 = 2.5$ in.

$A_{ctr} = (b/n)t_o = (165/7)5 = 118$ in.^{2}

Enter Composite Beam Tables (9th ED)

Pg. 2-270 for $S_{tr} = 267$ in.^{3}

Try W27×84

$S_{tr} \approx 301$ (interpolate for $Y2 = 2.5$ in.)

@ $A_{ctr} = 100$ in.^{2} < 118 in.^{2}

From Property Tables

S_s = 213 in.^{3}
A = 24.8 in.^{2}
I = 2850 in.^{4}
t_f = 0.64 in.
d = 26.71 in.
t_w = 0.46 in.

Calculate Section Properties

From table, at $Y2 = 2.5$ and $A_{ctr} \approx 30$ (partial comp)

$S_{tr} = 285$ in.^{3}
$I_{tr} = 6269$ in.^{4}

$$\bar{y}_{eff} = \left[\frac{6269}{285}\right] = 22$$

$y_t = (26.71 + 5) - 22 = 9.71$ in.

Check Concrete Stress (Unshored)

$$S_t = \left[\frac{I_{tr}}{y_t}\right] = \left[\frac{6269}{9.71}\right] = 645 \text{ in.}^3$$

$$f_c = \left[\frac{340(12)}{645(7)}\right] = 0.9 \text{ ksi} < 0.45(5\text{ksi}) = 2.25 \text{ ksi ok}$$

Check Steel Stresses

Total Load

$S_{tr} = 285 > 267$ in.$_3$ ok

Dead Load

$S_s = 213$ in.^{3} > 143 in.^{3} ok

$\therefore f_b$ is ok

$$f_v = \left[\frac{53.4}{26.71(0.46)}\right] = 4.34 \text{ ksi} < 20 \text{ ksi ok}$$

Check Deflection

$$\triangle_{DL} = \left[\frac{M_D L^2}{161 I_c}\right] = \left[\frac{340(55)^2}{(161)2850}\right] 1.23 = 1.06 = \frac{span}{570} \quad \text{ok}$$

Camber 2¼ in.

$$\triangle_{LL} = \left[\frac{393(55)^2}{161(6269)}\right] = 1.18 < \frac{span}{240} \quad \text{ok}$$

Check B/Flange Stresses

$$f_b = \left[\frac{393(12)}{213}\right] + \left[\frac{340(12)}{285}\right]$$
$$= 365 \text{ ksi} < 0.9(50) \text{ ok}$$

Shear Connectors (Partial Composite)

max. dia. = 2.5(0.64)
1.6 in. > 0.75 in. ok
Use ¾ in. dia.

Total Horizontal Shear

Concrete-Full

$$V_h = 0.85(f_c)\left(\frac{A_c}{2}\right) = 0.85(5)\left(\frac{5(165)}{2}\right) = 1753 \text{ kips}$$

Steel

$$V_h = \left[\frac{A_s F_y}{2}\right] = \left[\frac{24.8(50)}{2}\right] = 620 \text{ kips (governs)}$$

Max S_{tr} = 305 in.^{3} per Spec. para. I2

S_{eff} *Req'd* = 267 in.3

$$V_h' = V_h\left[\frac{S_{eff} - S_s}{S_{tr} - S_s}\right]^2 = 650\left[\frac{267-213}{305-213}\right]^2 = 223.9 \text{ kips}$$

$$N = \left[\frac{V'_h}{q}\right] = \left[\frac{223.9}{13.3}\right] = 16.8 \text{ per } \tfrac{1}{2} \text{ span}$$

Use 34, ¾ in. dia. by 3 in. shear stud connectors.

Note: Typical end beam and interior beam "B" are similar.

END BAY GIRDER

Design Parameters and Geometry

Same as LRFD Design

Calculate Concentrated Loads and Uniform Loads

P_{D+L} = 55 ft (1.04+0.9) = 106.7 kips
W (self wt.) Est. 200 lb/ft or 0.2 k/ft

Calculate Bending Moments and Shears

$$M_{D+L} = 15(106.7) + \frac{0.2(45)^2}{8} = 1651 \text{ k-ft}$$

V = 106.7+(45/2)(0.2) = 129.3 kips

Calculate Required Section Moduli (F_y = 50)

For M_{D+L} make sure top flange is braced, if not reduce f_b

$$S_{s\ Req'd} = \left(\frac{1651(12)}{33}\right) = 600 \text{ in.}^3$$

Select Section

Try W27×217
S_s = 624 in.^{3}
I_s = 8870 in.^{4}
t_w = 0.83 in.
d = 28.43 in.

Check Stresses

Bending

S_s = 624 in.^{3} > 600 in.^{3} ok

$\therefore f_b$ ok

Check Deflection

P_D = 55(1.04) = 57.2

$$\Delta_{max} = \frac{Pa}{24EI}\left(3L^2 - 4a^2\right)$$

$$\Delta_D = \left[\frac{57.2(15)(12)(3(45x12)^2 - 4(15x12)^2)}{24(29000)(8870)}\right]$$
$$= 1.23 \text{ in.}$$

Camber 1¼ in.

$$\Delta_L = \left[\frac{106.7 - 57.2}{57.2}\right]1.23 = 1.06 = \frac{span}{570} \text{ ok}$$

Use W27×217

BUMPER RAIL

Same as LRFD design procedure.

TYPICAL INTERIOR COLUMN

Design Parameters

Same as LRFD Section

Calculate Typical Design Loads

Dead

Slab	63psf
Steel	9psf
Total	72psf

Live

Code 50 psf (Assume 1 floor garage)

P_{D+L} = 18 ft × 55 ft (0.072 ksf + 0.05 ksf) = 121 kips

Select Trial Section

Try W12×40 (12 in. × 8 in. for connection)

Check Stresses

$KLx = KLy$ = 12 ft

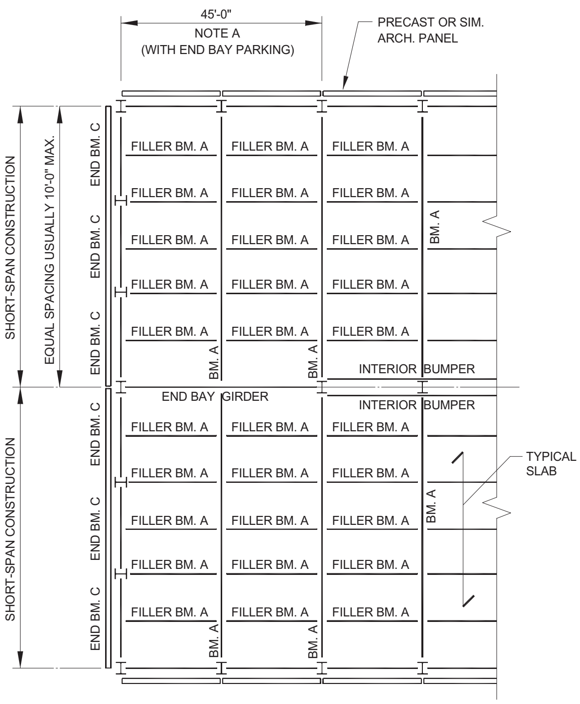
Allowable load on W12×40 = 237 kips > 121 kips (Ref. AISC pg. 3-28)

$f_a < F_a$ ok

Use W12×40

Appendix A2
Example: Cast-In-Place Concrete on Metal Deck

This exercise is an illustration of typical calculations in the design of a typical cast-in-place concrete on metal deck steel framed parking structure.



PARTIAL PLAN CAST-IN PLACE SLAB POURED ON METAL DECK GARAGE

THIS PARTIAL PLAN ILLUSTRATES TYPICAL COMPONENTS FOR A CAST-IN-PLACE SLAB POURED ON METAL DECK GARAGE

Geometry

To begin with the geometry of this example is:

Ramp width = 55 ft

Beam Spacing = Filler Beams – 10 ft c/c

Girder Beams = 22 ft -6 in. or 25 ft

Design Loading

Dead Loads

6 in. slab (total thickness poured on 3 in. × 20 ga. composite deck)	56 psf
Structural steel	10 psf
	66 psf

Live Loads

Gravity	50 psf
Wind	Note "A"
Seismic	Note "A"

Note "A": Refer to local codes for wind + seismic loads.

Component Design

Slab

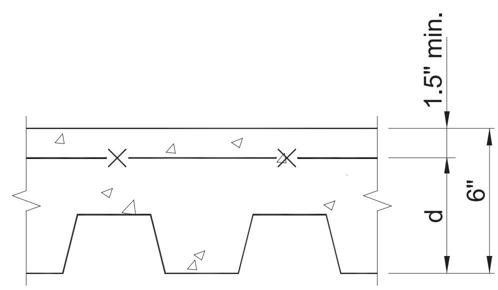
Unlike most cast-in-place slabs on metal deck a parking garage slab cannot use the metal deck as slab reinforcing.

Slab Design

Slab design for either 50 psf uniform load or 2,000 pound concentrated load

w_D = 56 psf
w_L = 50 psf
w_{uD+L} = 1.4(0.056)+1.7(0.05) = 0.163 ksf (uniform load)
w_{uD} = 1.4(0.056) = 0.0784 kip/ft
P_{uL} = 1.7(2) = 3.4 kips

Top Steel



d= 6 - 1.5 - .5 = 4"

Uniform Load Case

– M_u = [0.163(10)^2/12]12 = 16.3 k-in.

Concentrated Load Case

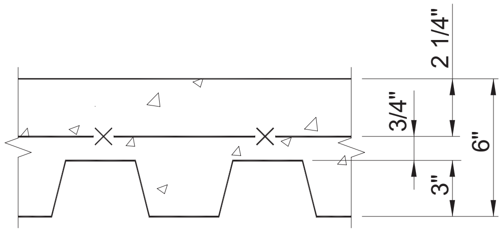
– M_u = [0.0784(10)^2/12 + 3.4(10/8)/4*]12 = 20.6 k-in. governs

* Note effective width of slab for concentrated loads is 48 in.

Set a = 0.2 in.

Required A_s = 20.6/0.9 × 60 × (4-0.2/2) = 0.1 in.^{2}/ft

Bottom Steel



$d = 2^1/_4$ in.

Uniform Load Case

$+M = (0.163(10)^2/24)12 = 8.2^{k\text{-}in}$
$+M = (0.0784(10)^2/24)12+(3.4(10)/8 \times 4)12 = 16.7$ k-in
Set $a = 0.2$ in.
$A_s = 16.7/0.9 \times 60 \times (2.25 \times 0.2/2) = 0.143$ in.^{2}/ft

Final Selection

Draped Wire Fabric

Use WWF 4 × 4/W3.0 × W3.0
A_s furnished = 0.14 in.^{2}/ft

Rebar
Use #4 @16 in. c/c Top & Bottom
$A_s = 0.2(12/16) = 0.15$ in.^{2}/ft

TYPICAL BEAM A

Span Length = 55 ft

Spacing = 22 ft-6 in.

Service Loading

Dead Loads (Service)

6 in. slab on 3 in. deck	56 psf
steel filler beams	3 psf
Total	59 psf

Uniform Dead Loads

$w = 22.5(0.059)+0.1$ (self wgt.) = 1.43 k/ft

Live Loads (Service)

Code 50 psf
Uniform Live Loads
$w = 22.5(0.05) = 1.125$ k/ft

LRFD DESIGN PROCEDURE

Load Cases

$1.4(D) = 1.4(1.43) = 2$ k/ft
$1.2(D) + 1.6(L) = 1.2(1.43) + 1.6(1.125) = 3.52$ k/ft
$1.6(L) = 1.6\ (1.125) = 1.8$ k/ft

Bending Moment

$M_u = 3.52(55)2/8 = 1331$ k-ft (factored)
$M_{DL} = 1.43(55)2/8 = 540$ k-ft (service)
$M_{LL} = 1.125(55)2/8 = 425$ k-ft (service)

Check Section and Determine Properties

Assume $a = 1$ in. $Y2 = 6\text{-}2/2 = 5$ in.
From composite beam tables for $F_y = 50$ ksi
and $Y2 = 5$ in.

Possible Selections

W27×84 or W30×90

Try W27×84

From Composite Table

$Y1 = 0.32$
$Q_n = 921$ kips
$\phi M_n = 1500$ kips

Compare $Y2$ for $\Sigma Q_n = 921$ kips

$b \le 2(L/8) = 2(55/8) = 13.75$ ft
$\le$ spa. = 22.5 ft
$b = 13.75 \times 12 = 165$ in.

$$a = \frac{\Sigma Q_n}{0.85\left(f_c'\right)b} = \frac{921}{0.85(5)(165)} = 1.31 \text{ in.}$$

$Y2 = 6\text{-}1.31/2 = 5.34$ in.

Compute number of studs required.

$Q_n = 26.1$ kips/each (Table 5.1)

Number of studs = $(2)\Sigma Q_n/Q_n = 2(921)/26.1 = 70.6$

Use 71 shear stud connectors

Construction Phase Check

A construction phase live load will be assumed from LRFD Specification (Section A4.1). The relevant combinations are:

$1.4(D) = 2$ k/ft
$1.2(D)+16(L) = 1.2(1.43)+1.6(0.02 \times 22.5 \text{ ft}) = 2.44$ k/ft
$M_u = 1331$ k-ft

From composite beam tables for W27×84

With an F_y = 50 ksi

$\phi M_n = \phi M_p$ = 1500 k-ft > 1331 k-ft

Service Load Condition

Assume that the fresh concrete load moment is equal to the service dead load moment with an I_{xx} = 2850 in.^{4}

$$\Delta_{DL} = \frac{540(55)^2}{161(2850)} = 3.56 \text{ in. (high but continue)}$$

For a W27×84 with $Y2$ = 5.34 and $Y1$ = 0.320

The lower bound moment of inertia can be found in the lower bound moment of inertia tables: I_{LB} = 6410

$$\Delta_{LL} = \frac{425(55)}{161(6410)} = 1.25 \text{ in.} = \frac{L}{528} < \frac{L}{240} \quad \text{ok}$$

Check Shear

V_u = 3.52(55/2) = 96.8 kips
$\phi V_n = 0.6(F_{yw})A_y$ = 0.6(50)(26.71 × 0.46) = 368 kips
> 96.8 kips

Final Selection

Use W27×84 F_y = 50 ksi with 71, ¾ in. dia. × 4½ shear stud connectors (35 each side of midspan + 3½ in. camber)

END BAY GIRDER

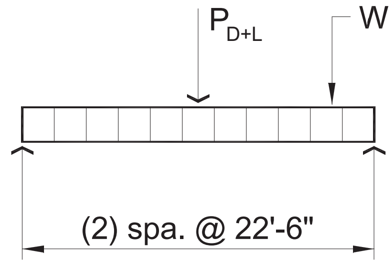
Design Parameters

Note: In order to minimize cracking the adjacent ramps should not be connected thus this girder should be non-composite.

Girder span = 45 ft (refer to Appendix A2-1A)

Concentrated load at midspan
Concentrated Loads

P_D	=	55(22.5) (0.066)(1.2)	=	89.1 kips
P_L	=	55(22.5)(0.05)(1.6)	=	99.0 kips
P_{D+L}	=			188.1 kips



Calculate Bending Moments + Shears

M_u = 0.24(45)^2/8 + 188.1(45/4) = 2176 k-ft
Equivalent uniform load = 2(188.1) = 376.2 kips (Tables pg. 4-139)
V_u = 188.1/2 = 94 kips
M_L = 61.9(45)/4 = 696 k/ft (service)
M_D = 81.6(45)/4 = 9.8 k/ft (service)

Select Section

Enter factored load table for F_y = 50 ksi and a

$\phi W_c/L \geq$ 376 kips
W27×194 $\phi W_c/L$ = 412(L = 46 ft) > 376 kips ok

Check Service Load Deflections

$$\Delta_{DL} = \frac{918(45)^2}{161(7820)} = 1.47 \text{ in.}$$

$$\Delta_{LL} = \frac{696(45)^2}{161(7820)} = 1.11 < \frac{L}{484} < \frac{L}{240} \quad \text{ok}$$

Final Selection

W27×194 F_y = 50 ksi camber = 1½ in.

BUMPER RAIL

For typical bumper rail calculations refer to Appendix A-1A.

TYPICAL INTERIOR COLUMN

For typical column calculations refer to Appendix A-1A.

ASD DESIGN PROCEDURE

TYPICAL BEAM A

Composite Beam

Bending Moments

Construction Loads

$M_D = 1.43(55)^2/8 = 540$ k-ft

Loads applied after concrete is set.
$M_L = 1.125(55)^2/8 = 425$ k-ft

Max. moment

$M_{D+L} = 540$ ft $+ 425 = 965$ k-ft

Max. shear

$V_{D+L} = (55/2)(1.43+1.125) = 70.3$ kips

Effective Width of Slab

$b = 1/4\ (L) = 1/4(55 \text{ ft} \times 12 \text{ in./ft}) = 165$ in. governs
$=$ spa. $= 15(12) = 180$ in.

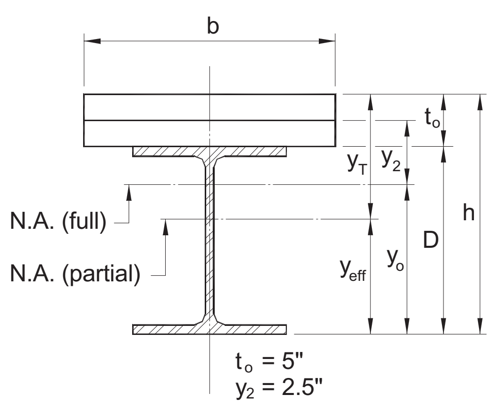
Required Section Modulus (F_y = 50 ksi)

For M_{D+L}

$S_{tr} = 965(12)/33 = 350$ in.^{3}

For M_D

$S_s = 540(12)/33 = 196$ in.^{3}



$Y2 = 4.5$ in.

$A_{ctr} = (b/n)\ t_o = (165/7)3 = 70.7$ in.^{2}

Enter Composite Beam Tables

Try W30×90

$S_{tr} = 353$ in.^{3} @ $A_{ctr} = 30$ in.$^2 < 70.7$ in.^{2}

From Property Tables

$S_s = 245$ in.^{3}	$A = 26.4$ in.^{2}	$I = 3620$ in.^{4}
$t_f = 0.611$	$d = 26.4$	$t_w = 0.47$

Calculate Section Properties

From Table with $a = 4.5$ in. and $A_{ctr} \approx 30$ in.^{2}

$I_{tr} = 8834$ in.^{4}

$\bar{y}_{eff} = 8834/353 = 25$ in. $\quad y_t = 6 + 29.5 - 25 = 10.5$ in.

Check Concrete Stresses

$$S_t = \left[\frac{I_{tr}}{y_t}\right] = \left[\frac{8834}{10.5}\right] = 841 \text{ in.}^3$$

$f_c = 965$ (12 in./ft) / 841 × 7 = 1.97 < 0.45(5 ksi) ok

Check Steel Stresses

Total load $S_{tr} = 353$ in.3 > 350 in.^{3}

Dead Load $S_s = 245$ in.3 > 196 in.^{3}

$\therefore f_b$ is ok

$f_v = 47.7/(0.46 \times 26.71) = 3.89$ ksi

Check Deflection

$\Delta_{DL} = M_{DL}L^2/161(I_s) = 540(55)^2/161(3620)$
$= 2.85$ in., camber $2\,3/4$
$\Delta_{LL} = M_{LL}(L^2)/161(I_{tr}) = 425\ (55)^2/161(8834)$

$$= 0.9 \text{ in.} \frac{span}{730} < \frac{span}{240} \quad \text{ok}$$

Check B/Flange Stress

$f_b = [540(12)/245]+[425(12)/353] = 40.9$ksi
< 0.9 (50 ksi)
B/flg stress ok

Shear Connectors

Max. dia. = 2.5(0.64) = 1.6 in. > 0.75 in. ok

Total Horizontal Shear

Concrete—Full Composite

$$V_h = 0.85 \times f'_c \times A_c / 2 = 0.85 \times 5 \times 3 \times 165 / 2$$
$$= 1051 \text{ kips}$$

Steel

$V_h = A_sF_y/2 = 26.4(50)/2 = 660$ kips governs

Max. S_{tr} = 385 in.^{3} with y = 4.5

S_{eff} req'd = 350 in.^{3}

$$V_h' = V_n \left[\frac{S_{eff} - S_s}{S_{tr} - S_s}\right] = 66.0\left[\frac{350-245}{385-245}\right]^2 = 371 \text{ kips}$$

$$N = V_h' / q = 371/13/3 = 28 \text{ Per } \tfrac{1}{2} \text{ span}$$

Use 56, ¾ in. dia. × 3 in. shear stud connectors

Note: Typical interior beam "B" and end beam "C" are similar.

END BAY GIRDER

Design Parameters

Same as LRFD Example

Calculate Concentrated Load + Uniform Loads

P_{D+L} = 55(22.5 ft)(0.066 ksf + 0.05 ksf) = 149 kips
w = 200 lb/ft Estimate or 0.2 k/ft

Calculate Bending Moments + Shears

M_{D+L} = 1490(45)/4 = 1676 k/ft
V = 149/2 = 75 kips

Calculate Required Section Modulus (F_y = 50 ksi)

$S_{s\ Req'd}$ = 1676(12)/33 = 609 in.^{3}

Select Section

Try W27×217 S_s = 624 in.^{3} I = 8870 in.^{4}
t_w = 0.83 in. d = 28.43 in.

Check Stresses

Bending

S_s = 624 in.^{3} > 609 in.^{3} ok
∴ f_b is ok

Check Deflection

P_D = 55(22.5) 0.066 = 81.7 kips
Δ_D = $P\ell^3/48EI$ = 81.7(45)3 1728/48(29000)(8870)
= 1.04 in.

Camber 1 in.
P_L = 55(22.5)(0.05) = 61.9
Δ_L = 61.9(45)^3(1728)/48(29000)(8870) = 0.8 in.

$$0.8 \text{ in.} = \frac{span}{675} < \frac{span}{240} \text{ ok}$$

Final Selection

Use W27×217 with 1 in. camber

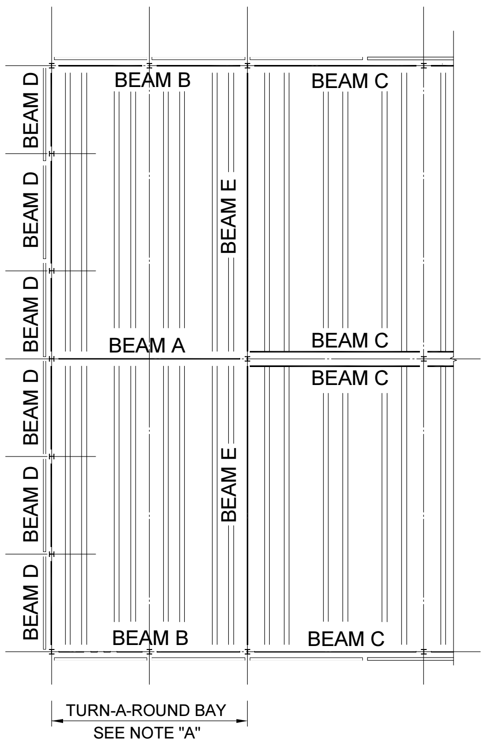
BUMPER RAIL

For typical bumper rail calculations refer to Appendix A-1A.

TYPICAL INTERIOR COLUMN

For typical column calculations refer to Appendix A-1A.

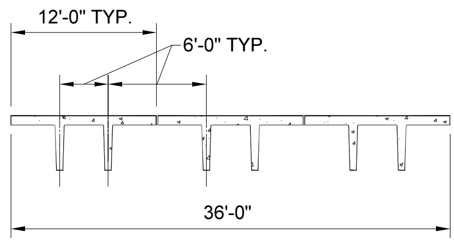
Appendix A3
Example: Precast—Twin Tee Deck



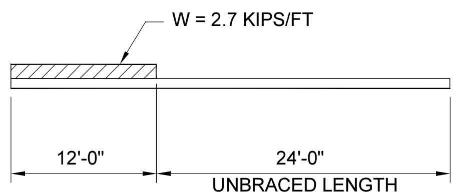
PARTIAL FLOOR PLAN - TWIN TEE DECK

DESIGN STEPS:

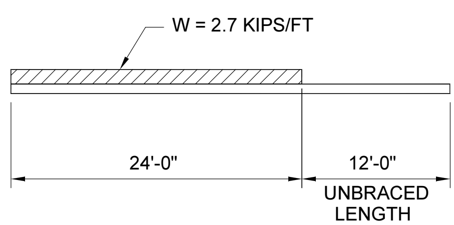
1.) CHECK BEAM FOR DEADAND LIVE LOAD - USING O/C OF THE STEMS AS UNBRACED LENGTH.



2.) CHECK BEAM FOR DEAD LOAD AND UNBRACED LENGTHS DURING ERECTION.



1 - TEE ERECTED



2 - TEES ERECTED

Design Loading Information

Dead Loads

Precast Twin Tee Deck	80 psf
Structural Steel	10 psf
Total	90 psf

Live Loads

Deck Loads (Code)	50 psf

Design Example

The bumper rail and column designs are similar to those in the previous examples and will not be repeated for this Appendix.

For precast deck-beam design, check the construction phase loadings of the beam as well as the final built condition. Figure A-3B illustrates these phases.

Since the beam design procedure is typical, only beam C will be illustrated.

BEAM C

LRFD DESIGN PROCEDURE

Design Loads

Uniform Loads

Dead $w_u = 1.2(2.7) = 3.24$ k-ft
Live $w_u = 1.6(1.5) = 2.4$ k-ft
Total $w_u = 5.64$ k-ft

Dead only

$w_u = 1.4(2.7) = 3.78$ k-ft

Load Case*	Moment	Unbraced Length
1	$5.64(36)^2/8 = 914$ k-ft	6 ft
2	1.4(135) = 189 k-ft	24 ft+3 ft = 27 ft
3	1.4(345) = 483 k-ft	12 ft+3 ft = 15 ft

* See Figure A-3B

Design Moment

Beam Selection and Moment Capacity Check

Using beam tables for Load Case 1 Pg. 4-81

$W_u = 5.64(36) = 202$ k-ft

Try W27×94 $\phi W_c / L = 232$ kips > 203 kips

Using unbraced charts for Load Case 2 and 3.

Load Case 2:

$M_x = 189$ k-ft $M_r = 442$ k-ft $@L_u = 27$ ft

Load Case 3:

$M_x = 483$ k-ft $M_r = 852$ k-ft $@L_u = 15$ ft

$$\Delta_{DL} = \frac{437.2(36)^2}{161(3270)} = 1.08 \text{ in.}$$

$$\Delta_{LL} = \frac{243(36)^2}{161(3270)} = 0.6 \text{ in.} = \frac{span}{722} < \frac{\text{span}}{240} \quad \text{ok}$$

Check Deflection

Final Selection

Use W27×94 $F_y = 50$ ksi With 1 in. camber

BEAM C

ASD DESIGN PROCEDURE

Design Loads

Uniform Loads

Dead: $w_D = 30(0.09) = 2.7$ k/ft
Live: $w_L = 30(0.05) = 1.5$ k/ft
Total: $w_{D+L} = 4.2$ k/ft

Load Case*	Moment	Unbraced Length
1**	$4.2(36)^2/8 = 680$ k-ft	6 ft
2***	27(10.2) = 135 k-ft	24 ft+3 ft = 27 ft
3***	13.2(16/2) = 34 k-ft	12 ft+3 ft = 15 ft

Moments

* See Figure A-3B
** Load Case 1—Dead + Live Load
*** Load Case 2 and Load Case 3—Dead Load Only

Beam Selection and Moment Capacity Check

Using Allowable Stress Design selection table for Load Case 1 - Pg. 2-10

Try W30×99

Load Case 1:

$M_x = 680$ k-ft $M_r = 740$ k-ft $L_c = 7.9$ ft > 6 ft ok

Load Case 2:

$M_x = 135$ k-ft $M_r = 240$ k-ft $@L_u = 27$ ft ok

Load Case 3:

$M_x = 345$ k-ft $M_r = 570$ k-ft $@L_u = 15$ ft ok

$$\Delta_{DL} = \frac{437.2(36)^2}{161(3990)} = 0.88 \text{ in.}$$

$$\Delta_{LL} \frac{243(36)^2}{161(3990)} = 0.5 \text{ in.} = \frac{span}{720} < \frac{span}{240}$$

Check Deflection

Final Selection

Use W30×99 $F_y = 50$ ksi With ¾ in. camber.

Appendix B
Protective Coating System Specification

The following coating specification defines procedures and materials required to achieve corrosion protection for exposed structural steel subjected to normal parking garage conditions. Many of the requirements borrow from the best practices of the steel bridge construction industry, which has used similar coating systems for decades of construction. When properly applied, the coating system outlined in this specification will protect parking garages subjected to road salt or coastal climates for 30 years or more before touch-up is required.

The high-performance, zinc-rich primer applied to abrasive blasted steel slows undercutting corrosion to imperceptible rates. After the first appearance of rust, the owner has an extended period to plan for maintenance painting. The zinc primer and urethane finish will continue to protect the structure and do not require removal in maintenance painting. Future maintenance is of the coating system, not the substrate.

Two modes of deterioration will be addressed in maintenance. The appearance of rust begins where physical damage occurs to the coating and where very thin primer films gradually give way to the forces of nature. The urethane finish coat is formulated to resist degradation from ultraviolet light but 30 years is a long time and southern exposures will chalk and fade by then. Proper color selection minimizes this appearance. Maintenance painting involves power tool cleaning followed by a spot primer and a full finish coat. The spot primer upgrades corrosion protection deficiencies and the full topcoat refreshes the finish and adds barrier protection to the system.

It should be noted that the specification provided here is meant to be a guide, and may not be appropriate for all applications. Proper scrutiny should be employed to ensure that actual project specifications are consistent with the expected performance of coating required, and the limitations of the system, as outlined throughout this Guide. In some cases, this may require the use of a three-coat painting system.

SECTION 09960

HIGH-PERFORMANCE COATINGS FOR STEEL

PART 1—GENERAL

1.1 SUMMARY

A. The General Provisions of the Contract, including General and Supplementary Conditions, and Division One—General Requirements, apply to work specified in this Section.

B. Section includes: labor, materials, tools, equipment, and services required for surface preparation and application of special coatings as specified and in locations scheduled.

C. Related Sections:

Section 05120:	Structural Steel
Section 05210:	Steel Joists
Section 05500:	Metal Fabrications
Section 05510:	Metal Stairs
Section 05520:	Pipe and Tube Railings
Section 09900:	Painting

1.2 DEFINITIONS

Applicator – A fabricator, paint contractor, or other entity that prepares steel or coated surfaces and applies coatings.

Best Effort – Actions expected of a reasonably knowledgeable and trained person to properly perform an activity.

Breaking the Corner (Corner Chamfering) – A process by which a sharp corner is flattened by passing a grinder or other suitable device along the corner, normally in a single pass.

Conformance Certification – A verification issued by the coating manufacturer confirming that a particular batch of product was produced in accordance with the manufacturer's standard. This standard of performance for the product must have previously been approved or accepted by the Owner.

Corner – The intersection of two surfaces.

Edge – An exposed, through-thickness surface of a plate or rolled shape. This may be the as-rolled side face of a beam flange, channel flange or angle leg, or may result from thermal cutting, sawing, or shearing. Edges may be planar or rounded, and either perpendicular or skewed to adjacent faces.

Fastener – A mechanical device used to attach two or more items together, such as a bolt, nut and washer.

Hackles, Fins, Scabs – Hackles, fins, and scabs are as-received defects in the steel surface. Usually, hackles, fins, and scabs affect only a thin (less than $^1/_{16}$ in. or 2 mm) layer. The defects are often apparent after blast cleaning because the abrasive impact causes a loose edge to rise from the plane of the surface. Hackles, fins, and scabs may normally be removed by use of a grinder, scraper or chisel. Sometimes gouging and welding are necessary for deep scabs.

Inaccessible Areas – Partially or completely enclosed surfaces, the majority of which are not visible without the use of special devices such as mirrors.

Sharp – An acute corner or prominence that is able or appears to be able to cut human flesh. (Cut corners are often judged to be sharp, rolled corners (such as flange toes) are usually judged not to be sharp.)

Snipe – The area remaining clipped at a corner to clear a weld or rolled fillet.

Spot Prime Coat – Spot Priming: application of primer paint to localized spots where the substrate is bare or where additional protection is needed because of damage to or deterioration of a former coat.

Stripe Coat – A coat of paint applied only to edges or to welds on steel structures before or after a full coat is applied to the entire surface. The stripe coat is intended to give those areas sufficient film build to resist corrosion.

Visible Coating Defects – Imperfections that may be detected by the unaided eye. Visible Coating Defects include runs, sags, lifting, chipping, cracking, spalling, flaking, mudcracking, pinholing, and checking.

Visual Coverage – Acceptable coating of inaccessible areas or surfaces inaccessible to manual spray painting equipment and dry film thickness (DFT) gages. DFT requirements are waived; however, surfaces may be inspected for visual coverage by the unaided eye, video monitoring or inspection mirror.

Weld Spatter, Tight – Small weld metal droplets expelled during exposed-arc welding with adequate thermal energy to adhere on metal adjacent to the weld area. The droplets retain their individual shape but have sufficient fusion to resist removal by hand scraping with a putty knife, per SSPC-SP 2.

1.3 REFERENCES

A. **American Society for Testing Materials (ASTM):**

ASTM A 6, *Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*
ASTM A 36, *Specification for Carbon Structural Steel*
ASTM B117, *Test Method of Salt Spray (Fog) Testing*
ASTM D 523, *Test Method for Specular Gloss*
ASTM D 1654, *Test Method for Evaluation of Painted or Coated Specimens Subjected to Corrosive Environments*
ASTM D 2244, *Test Method for Calculation of Color Differences from Instrumentally Measured Color Coordinates*
ASTM D 2247, *Testing Water Resistance of Coatings in 100% Relative Humidity*
ASTM D3359, *Standard Test Methods for Measuring Adhesion by Tape Test*
ASTM D4060, *Abrasion Resistance of Organic Coatings by the Taber Abraser*
ASTM D4138, *Standard Test Method for Measurement of Dry Paint Thickness of Protective Coating Systems by Destructive Means*
ASTM D4285, *Standard Test Method for Indicating Oil or Water in Compressed Air*
ASTM D4414, *Standard Practice for Measurement of Wet Film Thickness by Notch Gages*
ASTM D4417, *Standard Test Methods for Field Measurement of Surface Profile of Blast Cleaned Steel*
ASTM D4541, *Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers*

B. **SSPC: The Society for Protective Coatings:**

SSPC-AB 1, *Mineral and Slag Abrasives*
SSPC-AB 2, *Cleanliness of Recycled Ferrous Metallic Abrasives*
SSPC-AB 3, *Newly Manufactured or Re-Manufactured Steel Abrasive*
SSPC-PA 1, *Shop, Field, and Maintenance Painting of Steel*
SSPC-PA 2, *Measurement of Dry Film Thickness with Magnetic Gages*
SSPC-QP 1, *Standard Procedure for Evaluating Painting Contractors (Field Application to Complex Industrial Structures)*
SSPC-QP 3, *Standard Procedure for Evaluating Qualifications of Shop Painting Contractors*
SSPC-SP 1, *Solvent Cleaning*
SSPC-SP 2, *Hand Tool Cleaning*
SSPC-SP 3, *Power Tool Cleaning*
SSPC-SP 5/NACE No. 1, *White Metal Blast Cleaning*
SSPC-SP 10/NACE No. 2, *Near-White Blast Cleaning*
SSPC-SP 11, *Power Tool Cleaning to Bare Metal*
SSPC-SP COM, *Surface Preparation and Abrasives Commentary, SSPC Painting Manual, Volume 2, "Systems and Specifications"*
SSPC-VIS 1, *Visual Standard for Abrasive Blast Cleaned Steel*
SSPC VIS 3, *Visual Standard for Power- and Hand-Tool Cleaned Steel*

C. **Research Council on Structural Connections (RCSC):**
Specification for Structural Joints Using ASTM A325 or A490 Bolts

D. **American Institute of Steel Construction (AISC):**
Sophisticated Paint Endorsement (SPE)
Manual of Steel Construction

E. **Related Reference Documents:**
Applicable Ordinances and Regulations
Equipment and Coating Manufacturer's Published Instructions and Product Data Sheets

1.4 SYSTEM DESCRIPTION

A. Products provided and installation by this Section are special coating materials requiring special expertise in surface preparation, application and safety procedures, and should not be confused with conventional paint systems specified in Section 09900.

1.5 SUBMITTALS

A. General: Submit in accordance with Section 01330, Submittal Procedures.

B. Product Data:

1. Submit complete range of manufacturer's standard colors for selection by Architect.
2. Submit a sample of the custom color in the proposed gloss.
3. Resubmit samples until color match is acceptable to Architect.

C. Quality Control Submittals: Submit certified test reports from acceptable independent testing laboratory indicating coatings comply with specified performance requirements and Manufacturer's Certificate of Compliance.

1.6 QUALITY ASSURANCE

A. Applicator Qualifications:

1. Applicator shall have minimum five years experience applying special coating materials or carry Sophisticated Paint Endorsement, QP 3 or QP 1 certification as applicable. Applicator must provide proof of certification or the following:

   a. Minimum five years commercial experience applying industrial grade coatings.
   b. Minimum five successful projects of similar scope and complexity.
   c. List of references for completed projects.
   d. Qualifications and project history of proposed job superintendent.

2. Applicator shall employ skilled craftsmen to ensure highest quality workmanship. Materials to be applied by craftsmen experienced in use of specified products.

B. Regulatory Requirements: Comply with applicable codes, regulations, ordinances, and laws regarding use and application of coating systems that contain volatile organic compounds (VOC).

C. Pre-Application Conference: Prior to making field samples and placing order for materials, Architect, Contractor, installer and manufacturer's representative shall meet and agree on methods and schedule for application.

D. Manufacturer shall review and advise applicator on proper application procedures and techniques. Initial application shall be observed by coating manufacturer representative.

1.7 DELIVERY, STORAGE AND HANDLING

A. Deliver materials to application site in original, factory-sealed, unopened, new containers bearing manufacturer's name and label intact and legible, with following information:

1. Product identification (name or title of material).
2. Manufacturer's batch number and date of manufacture.
3. Mixing instructions.
4. Thinning instructions.
5. Application instructions.
6. Color designation.

B. Store materials in a protected and well-ventilated area at temperatures in accordance with manufacturers instructions.

C. Use only thinners manufactured and recommended by coating system manufacturer for each paint or coating used.

1.8 PROJECT CONDITIONS

A. Apply coating materials only under following prevailing conditions:

1. Air and surface temperatures shall not exceed minimum or maximum requirements for product application as stated on product data sheet.
2. Do not apply coatings to damp or wet surfaces.
3. Relative humidity that is not above 85 percent and surface temperature that is at least 5 °F above the dew point temperature at the time of application, and for a minimum of four hours after application.
4. Wind velocity must be less than 20 mph.

B. Coordinate special coatings work with other trades to ensure adequate illumination, ventilation, and dust-free environment during application and curing of special coatings.

C. Protect adjoining surfaces not to be coated against damage or soiling.

D. Maintain work area in a neat and orderly condition, removing empty containers, rags, and rubbish daily from the site and disposing of it properly and legally.

E. Maintain a safe work environment in accordance with federal, state, local and project site regulations and guidelines.

PART 2—MATERIALS

2.1 COATING MANUFACTURERS

A. The provisions in Part 2 provide a standard of quality for the coatings system and capabilities of the coating supplier. Coating manufacturers and their systems must demonstrate compliance to these provisions.

B. The Manufacturer of special coatings under this section must be certified to meet the requirements of ISO 9001 and ISO 9002.

2.2 COATING SYSTEM REQUIREMENTS

Primer: Zinc-rich epoxy, gray-green at 4 mils DFT. Primer must be rated for slip-critical connections in accordance with RCSC Class A or Class B and in conformance with design requirements.

Finish: High-build aliphatic polyurethane, 4 mils DFT.

Touch-up primer: High solids aluminum filled epoxy mastic primer at 5 mils DFT.

The coatings products submitted must meet the following requirements to be approved for use. Compositional limitations on the products are minimal. Those stated here are for regulatory reasons. Performance requirements are stringent and, therefore, require verification of an approved independent laboratory.

A. Compositional and Supply Requirements:

1. Products must be of generic types specified under Coating Materials.
2. Each coating must contain no more than 0.06 percent lead in the dry film.
3. The volatile organic compound (VOC) content of each coating must not exceed 3.5 lb/gal. (420 grams/liter).
4. Each coating in the system must be compatible with the other coatings in the system.

B. Performance Requirements:

In order to be approved for use, coatings must be tested in accordance to the listed ASTM methods on steel panels prepared as described below. Results of these tests must meet or exceed performance stated.

1. Panel preparation: Test panels shall be grade ASTM A36, hot-rolled steel, 3 in. by 5 in. or larger, ¼" thick. Panels shall be blast cleaned with metallic abrasive to a cleanliness of SSPC SP 5, white metal blast with an average anchor profile of 2 mils as measured according to ASTM D4417, Method C.
2. Coating thickness: Each coating shall be spray applied at the manufacturer's recommended dry film thickness, not to exceed 5 mils for the primer or finish coat.
3. Curing: Coated panels shall be cured at least 14 days at indoor conditions.
4. Scribing: Test panels shall be scribed in accordance with ASTM D1654 with a single "X" mark centered on the panel in dimensions of roughly 50 mm by 100 mm. The scribing tool shall be a straight-shank, tungsten carbide, lathe cutting tool (ANSI B94.50, Style E). The scribe incision shall expose the steel substrate as verified by a microscope.
5. Substantiation: All testing shall be performed in triplicate according to the cited ASTM methods, including the reporting of the average of each group of three test panels. Test results shall be stated on the independent laboratory stationary.

C. Primer Performance Requirements:

The following performance standards are required for the epoxy zinc-rich primer applied in a single coat to steel panels as described above.

1. Salt Fog Test (ASTM B 117): Exposure duration shall be 5,000 hours. Panels shall show no delamination, blistering, rusting or rust creep at the scribe.
2. Cyclic Weathering Test (ASTM D 5894): Exposure duration shall be 5,000 hours. Panels shall show no delamination, blistering, rusting or rust creep at the scribe.
3. Humidity Test (ASTM D 2247): Exposure duration shall be 4,000 hours. Panels shall show no delamination, blistering, rusting or rust creep at the scribe.
4. Adhesion Test (ASTM D4541). Average adhesion of three panels tested shall be at least 900 psi when tested using a fixed alignment adhesion tester (Annex A.2), manufactured by Elcometer, Ltd.

D. Finish Coat Performance Requirements:

The polyurethane finish is required to meet the abrasion and weathering standards established here.

1. Taber Abrasion (ASTM D4060). The high-build polyurethane may be tested with or without primer. Test shall be conducted using CS-17 wheels for 1,000 cycles at 1,000 grams load. The average weight loss shall not exceed 100 milligrams.
2. Cyclic Weathering (ASTM D5894). After 5,000 hours of exposure the polyurethane finish shall retain at least 70 percent of its original gloss (average of three panels) as measured per ASTM D 523 using an incidence angle of 60º, and the color shift shall not exceed a ΔE 3 as measured per ASTM D 2244 (illuminant D65 and a 2º observer).

PART 3—EXECUTION

3.1 SURFACE PREPARATION

A. Material Anomalies

1. Corner Condition—Remove all sharp corners prior to painting by creating a small chamfer.
2. Preparation of Thermally Cut Edges—Thermally cut edges (TCEs) to be painted shall be conditioned before blast cleaning, if necessary, to achieve proper profile.
3. Base Metal Surface Irregularities—Remove all visually evident surface defects in accordance with ASTM A6 prior to blast cleaning steel. When material defects exposed by blast cleaning are removed, the blast profile must be restored by either blast cleaning or by using mechanical tools in accordance with SSPC-SP 11.
4. Weld Irregularities or Spatter—Remove or repair all sharp weld prominences, weld deficiencies (overlap; rollover; excessive concavity, convexity, or roughness), and all heavy, sharp, or loose weld spatter. Occasional individual particles of rounded tight weld spatter may remain, but widespread, sharp, or clustered particles of tight weld spatter must be removed.

B. Pre-Cleaning

Remove all oil, grease, and other adherent deleterious substances from areas to be painted, in accordance with SSPC-SP 1, prior to abrasive blast cleaning.

C. Abrasive Blast Cleaning

Abrasive blast clean the entire surface to achieve a cleanliness of Near White Finish (SSPC-SP 10/NACE No. 2) with a surface profile of 1 to 3 mils. Expendable abrasives shall meet the requirements of SSPC AB1; recyclable steel abrasives shall meet the requirements of SSPC AB2 and AB3. The surface cleanliness shall be verified using SSPC VIS 1. The surface profile shall be measured per ASTM D4417, Method C (Replica Tape).

3.2 APPLICATION

A. Mix and apply coating materials in accordance with manufacturer's directions, and in accordance with SSPC PA 1. Apply at the minimum specified thickness without exceeding maximum allowed dry film thickness recommended by manufacturer. Wet film thickness shall be monitored by the coating applicator, per ASTM D4414. Dry film thickness of each coating layer shall be measured and recorded in accordance with SSPC PA 2.

B. Apply coating materials by spray application to scheduled surfaces in accordance with manufacturer's recommendations. Faying surfaces of bolted joints shall be primed to a thickness not exceeding RCSC requirements. Primed faying surfaces shall be masked if the finish coat is applied in the shop.

1. Rate of application shall not exceed manufacturer's recommendations.
2. Mix all material as required by the manufacturer for application of materials.
3. Comply with manufacturer's recommendations for drying or curing time between coats.
4. Finished surfaces shall be uniform in finish and color.

C. Work material into surface voids. Daub material behind corner clips in stiffeners and other attachments and in restricted areas inaccessible to spray application. Cut in edges clean and sharp, without overlapping, where work joins other materials or colors.

D. Make finish coats smooth, uniform in texture and color, and free of brush marks, laps, runs, dry spray, overspray and skipped or missed areas.

E. Allow sufficient curing time for coatings to be handled. Steel must be handled using padded lifting points and dunnage for storage, shipping and erection.

3.3 QUALITY CONTROL

A. Surface preparation must be inspected by QC before proceeding with coating application.

B. Inspect each coat before applying succeeding coats in accordance with SSPC-PA 2. If inspection (QC & QA) is to be conducted, it is to be done prior to application of the subsequent coat and in a timely fashion.

C. Furnish and maintain at the project site the following fully calibrated testing and inspection devices available for use by manufacturer's representative or Architect:

1. Sling Psychrometer, U.S. Weather Bureau Tables, and Surface Temperature Thermometer, or electronic psychrometer with surface temperature probe.
2. Testex Micrometer and Replica Tape.
3. SSPC VIS 1 and VIS 3.
4. Notch-Type Wet Film Thickness Gauge.
5. Type 1 (manual-magnetic pull-off) or Type 2 (electronic-constant pressure probe) dry film thickness gage and calibration blocks (NIST traceable or other).
6. Inspection Mirror.

D. Record blast profile, DFT, humidity and air and surface temperature readings.

3.4 TOUCH-UP AND REPAIR PROCEDURES

A. Touch-up of shop-applied coating system.

1. Areas left unpainted, as for welding, areas with damaged coatings and field-installed fasteners and fasteners installed during fabrication, after primer application, shall be cleaned in accordance to SSPC-SP 2, hand tool cleaning in preparation for spot priming.
2. Apply epoxy mastic primer at a dry film thickness of 5 mils by brush in accordance with the manufacturer's recommendations in a neat and workman like manner.
3. Apply urethane finish by brush or spray to a dry film thickness of 4 mils in accordance with the manufacturer's recommendations. Material shall be applied in a manner to minimize touch-up appearance, i.e. to edges of natural break points such as gusset plate edges. The wet edge of touch-up finish must be squared off neatly. If spraying, mask surfaces unintended for coating application.

3.5 CLEANING

A. Remove coating spatters and overspray from inappropriate surfaces.

B. Properly dispose of all waste and trash in accordance with law and regulations.

Appendix C
Bibliography of Technical Information on Parking

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