PDHonline Course S211 (6 PDH)

Antiquated Structural Systems - Part 1

Instructor: Matthew Stuart, PE, SE

2020

PDH Online | PDH Center

5272 Meadow Estates Drive
Fairfax, VA 22030-6658
Phone: 703-988-0088
www.PDHonline.com

An Approved Continuing Education Provider

For this presentation, "antiquated" is defined as meaning outmoded or discarded for reasons of age. Most of the systems that will be discussed are no longer in use because they have been replaced by more innovative or more economical methods of construction.

Reading Terminal – Philadelphia, PA
Source: Historic American Building Survey

This is a list of the structural systems we will discuss . Precast concrete, post-tensioned concrete and open web steel joists are still in use today, therefore this presentation will provide information on the early development of these same systems. Stub-Girders, which are discussed in Part 2 of this course, are a fairly recently developed system, however, this type of steel framing is no longer in use.

- **The Circumferential or S.M.I System of Reinforced Concrete Flat Slabs**
- **Clay Tile Arched Floor Systems**
- **One and Two-Way Clay Tile and Unit Masonry Joist Systems**
- **Prefabricated Clay Tile & Concrete Block Framing Systems**
- **Precast Concrete Framing Systems**
- **Structural Steel Composite Stub-Girder Construction (Part 2)**
- **Post-Tensioned Concrete Construction (Part 2)**
- **Wrought and Cast Iron (Part 2)**
- **Open Web Steel Joists (Part 2)**
- **Miscellaneous Antiquated Systems (Masonry, Draped Mesh Slabs, Brick Arch Slabs, Concrete Reinforcement) (Part 2)**

As developable land becomes more difficult to find, particularly in densely populated urban cities or suburban areas in which open space cannot be used, owners and developers are increasingly turning to existing facilities to convert into new uses.

Manhattan as seen from Newark, New Jersey

The need for existing structural drawings as a part of the evaluation of an existing structure is obvious, however, the ease at which an existing structure can be analyzed in the absence of drawings depends on the nature of the system and the extent to which the structure is concealed or exposed, and accessible.
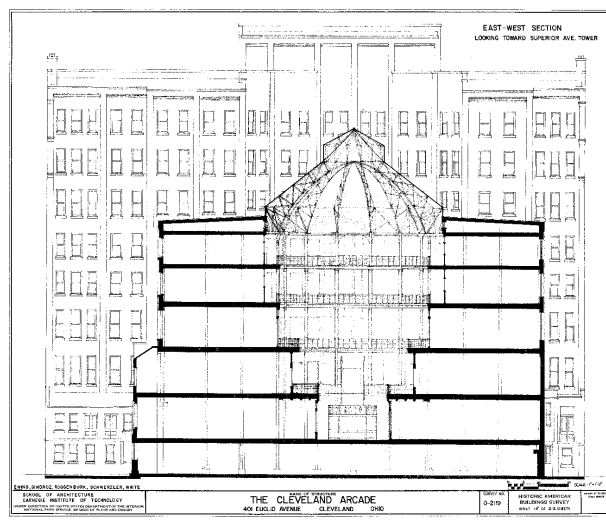
Source: Historic American Building Survey

The analysis of an existing exposed structural steel building is a much easier task than a reinforced concrete structure simply because the steel members can be measured and their capacity quickly determined. With a reinforced concrete structure, determining what the internal reinforcement is in order to facilitate the calculation of the member capacity can be a very difficult and challenging task.

Source: Historic Preservation Dept. - SCAD

How to determine the capacity of an internally reinforced structure:

1. Determine the building usage at the time of the initial construction, research the building code from the same time period, and establish the minimum live load required for the original intended use. Examples of older building code minimum live load requirements are tabulated below:

| Minimum Building Code Live Load - PSF						
Building Type	**New York**	**Philadelphia**	**Boston**	**Chicago**	**Denver**	**San Francisco**
	1927	**1929**	**1926**	**1928**	**1927**	**1928**
Residential:	40	40	50	40	40 & 60	40
Hotels, Hospitals:	40	40	50	40	90	40
Office Buildings:						
First Floor	100	100	125	125	125	125
Upper Floors	60	60	60	40	70 & 90	40
Classrooms:	75	50	50	75	75	75
Public Seating:						
Fixed Seats	100	60	100	75	90	75
Without Fixed Seats	100	100	100	125	120	125
Garages:						
Public	120	100	150	100	150	100
Private	120	100	75	100	150	100
Warehouses:	120	150	125-250	125-250	200	125-250
Manufacturing:						
Heavy	120	200	250	250	250	250
Light	120	120	125	125	120	125
Stores:						
Wholesale	120	110	250	250	120	125
Retail	120	110	125	125	120	100
Sidewalks:	300	120	250	150	150	150

©D. Matthew Stuart

2. Create small exploratory, demolished openings to expose the internal reinforcement at areas of the framing that are not susceptible to removal of small amounts of material. This approach, used in conjunction with a Profometer (or Pachometer) can sometimes enable the determination of existing reinforcing in large areas surrounding the exploratory opening.
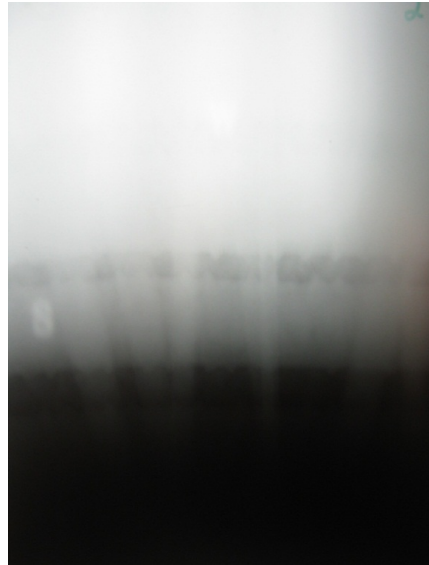

Exploratory demolition can also be utilized in many different situations in order to reveal hidden or concealed structural systems.

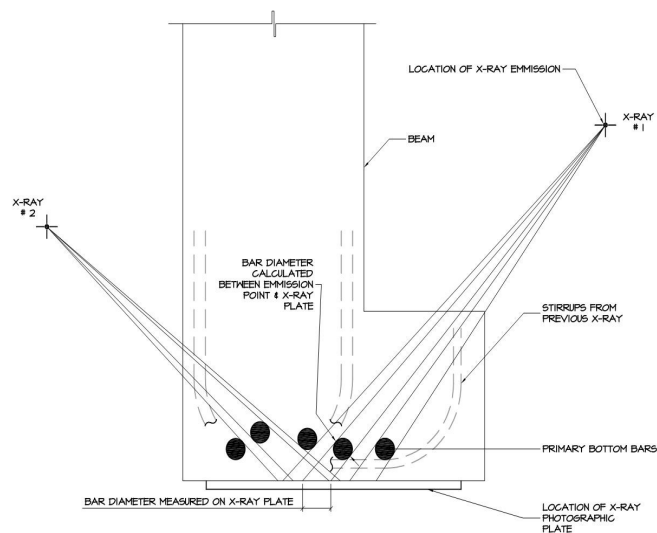3. X-Ray the members in question to locate the size and spacing of the internal reinforcement.
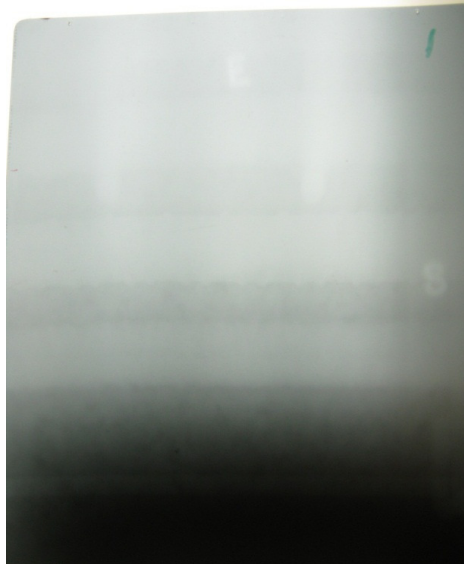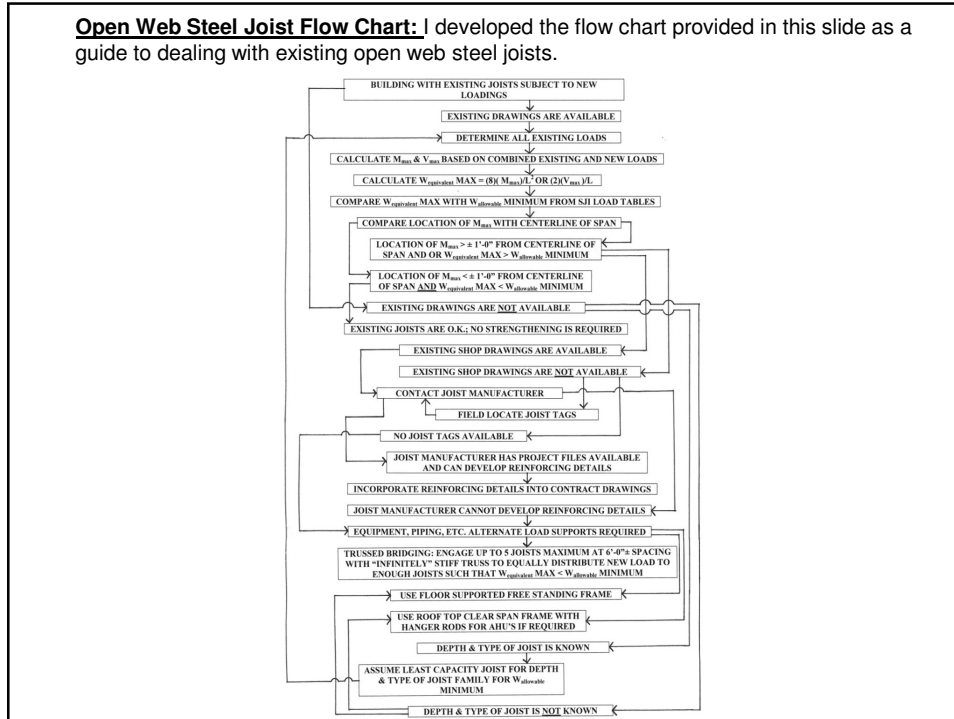

X-Ray #1   **X-Ray #2**

This diagram shows how the previous X-Rays were used to determine the number and size of the primary flexural reinforcement. The analysis of X-Rays requires an understanding of the projection of shadows on to the X-Ray plate relative to the origin of the X-Ray source, which is completely different from the analysis of a photograph.

Interpretation of X-Ray #1 & #2 at a Precast Ledger Beam

Load Tests: If a load test of an existing system is required it should be conducted per ASTM E196, Standard Practice for Gravity Load Testing of Floors and Flat Roofs. I do not recommend the use of load testing unless you already know what the calculated design capacity is for the structure.

Source: Testconsult

Load Tables: Unless there are specific mark numbers on the product in the field that clearly identify the member in relationship to the load table, it is not advisable to establish load capacities of an existing structure using only historical load table data.

Slab and Reinforcing Geometry

STRANDS SHOULD BE EVENLY DISTRIBUTED AND PLACED SYMMETRICALLY ABOUT CENTERLINE

Load Table of Allowable Superimposed Service Loads in pounds / sq. ft.

Strand	Area of Steel Inch sq.	Mu K-ft	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
6 - 3/8" dia.	0.425	34	239	198	165	138	117	99	84	72	61	52	43								
6 - 3/8" dia.	0.51	40	288	239	201	170	145	124	106	91	78	68	58	50	43						
6 - 7/16" dia.	0.575	43	312	260	219	186	158	136	117	101	87	76	65	56	46	42					
7 - 3/8" dia.	0.595	46	337	281	237	201	172	148	128	111	96	84	73	63	55	47	41				
8 - 3/8" dia.	0.68	52	386	323	273	233	200	173	150	130	114	100	87	76	67	58	51	45			
9 - 1/2" dia.	0.765	55	410	344	291	248	214	185	161	140	123	108	95	83	73	64	57	50	43		
8 - 7/16" dia.	0.92	66	414	365	343	306	264	230	201	176	155	137	121	108	96	86	76	67	60	53	47
9 - 7/16" dia.	1.035	72	417	372	348	317	292	264	222	195	173	153	136	121	108	96	86	77	69	62	66
7 - 1/2" dia.	1.071	74	421	379	362	323	301	262	230	202	178	158	141	125	112	100	90	80	72	64	58
8 - 1/2" dia.	1.224	78	424	384	366	328	305	278	244	215	190	169	150	134	120	108	97	87	78	70	63
9 - 1/2" dia.	1.377	85						311	273	241	214	190	170	152	137	123	111	100	90	81	73

(Includes 10PSF dead load.)

Values below lower heavy line indicate web shear controls.
Values above upper heavy line indicate long-time camber less than zero with no superimposed service loads.

Source: Pre-Cast Specialties, Inc.

Open Web Steel Joist Flow Chart: I developed the flow chart provided in this slide as a guide to dealing with existing open web steel joists.

BUILDING WITH EXISTING JOISTS SUBJECT TO NEW LOADINGS

EXISTING DRAWINGS ARE AVAILABLE

DETERMINE ALL EXISTING LOADS

CALCULATE M_{max} & V_{max} BASED ON COMBINED EXISTING AND NEW LOADS

CALCULATE $W_{equivalent}$ MAX = (8)(M_{max})/L^2 OR (2)(V_{max})/L

COMPARE $W_{equivalent}$ MAX WITH $W_{allowable}$ MINIMUM FROM SJI LOAD TABLES

COMPARE LOCATION OF M_{max} WITH CENTERLINE OF SPAN

LOCATION OF M_{max} > ± 1'-0" FROM CENTERLINE OF SPAN AND OR $W_{equivalent}$ MAX > $W_{allowable}$ MINIMUM

LOCATION OF M_{max} < ± 1'-0" FROM CENTERLINE OF SPAN AND $W_{equivalent}$ MAX < $W_{allowable}$ MINIMUM

EXISTING DRAWINGS ARE NOT AVAILABLE

EXISTING JOISTS ARE O.K.; NO STRENGTHENING IS REQUIRED

EXISTING SHOP DRAWINGS ARE AVAILABLE

EXISTING SHOP DRAWINGS ARE NOT AVAILABLE

CONTACT JOIST MANUFACTURER

FIELD LOCATE JOIST TAGS

NO JOIST TAGS AVAILABLE

JOIST MANUFACTURER HAS PROJECT FILES AVAILABLE AND CAN DEVELOP REINFORCING DETAILS

INCORPORATE REINFORCING DETAILS INTO CONTRACT DRAWINGS

JOIST MANUFACTURER CANNOT DEVELOP REINFORCING DETAILS

EQUIPMENT, PIPING, ETC. ALTERNATE LOAD SUPPORTS REQUIRED

TRUSSED BRIDGING: ENGAGE UP TO 5 JOISTS MAXIMUM AT 6'-0"± SPACING WITH "INFINITELY" STIFF TRUSS TO EQUALLY DISTRIBUTE NEW LOAD TO ENOUGH JOISTS SUCH THAT $W_{equivalent}$ MAX < $W_{allowable}$ MINIMUM

USE FLOOR SUPPORTED FREE STANDING FRAME

USE ROOF TOP CLEAR SPAN FRAME WITH HANGER RODS FOR AHU'S IF REQUIRED

DEPTH & TYPE OF JOIST IS KNOWN

ASSUME LEAST CAPACITY JOIST FOR DEPTH & TYPE OF JOIST FAMILY FOR $W_{allowable}$ MINIMUM

DEPTH & TYPE OF JOIST IS NOT KNOWN

Wood Framing: Wood framed buildings are similar to structural steel buildings in that the members, if exposed and readily accessible, can easily be measured in the field to facilitate the analysis of the individual members to determine load carrying capacities. Unfortunately, it can be difficult to establish the appropriate allowable stress of the existing wood that should be assumed for the structural analysis. To solve this dilemma I take small pieces (no larger than a toothpick) of the timber framing and send the samples to the US Forest Products Laboratory in Madison, WI in order to determine the species of the wood. Once the species is determined, along with the age of the building, it is possible to easily determine reasonable allowable stresses from historical resources.

Additional methods of evaluating the properties of an existing structural system include:

▪**Cores Samples** (for determining compressive strength, and depths and thicknesses)

▪**Coupons** (to determine iron or steel tensile strength)

▪**Petrographic Analysis**   (to determine the quality, condition and consistency of concrete)

▪**GPR** (**G**round **P**enetrating **R**adar; to locate hidden embedded materials)

▪**Schmidt Hammer** (to determine in situ concrete compressive strength)
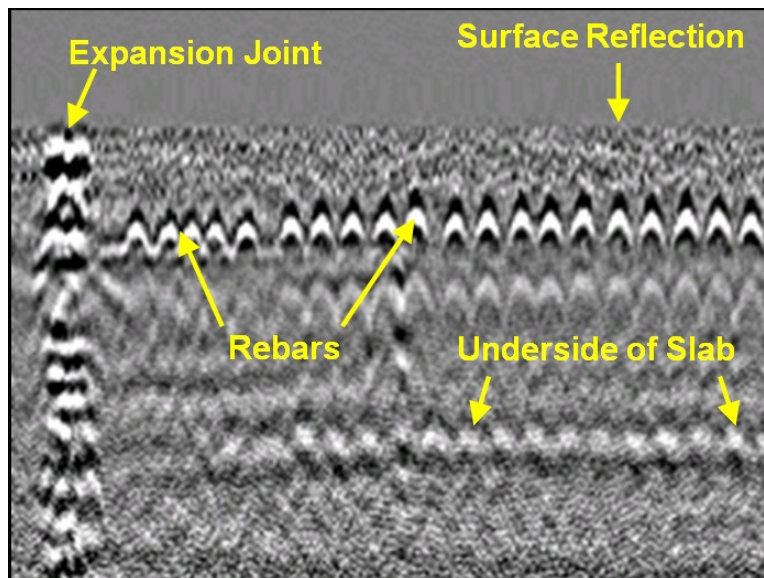
GPR Printout
Source: Siva Corrosion Services

Engineers involved with renovation and rehabilitation projects need to be aware of the specifics of antiquated structural systems in order to develop non-destructive and unobtrusive solutions.

This approach enables the project to be more economically viable because of the extent of structural costs associated with a typical renovation project.

In other words, without any knowledge of an existing structural system it is still possible to develop a structural solution, however, this approach will always be much more intrusive, and therefore more costly, than if the engineer has a sound understanding of the system involved.

Early 20th Century Retail Arcade
Source: Historic American Building Survey

Information concerning antiquated structural systems provided by this presentation, and the source materials referenced in the published articles, has been compiled and made available because the history of structural systems is far less documented than the history of architecture. This lack of documentation can be traced to the general public attitude towards the hidden structural components of a building, which are typically enclosed after erection by the architectural finishes and therefore of less interest to any observer.

This general lack of readily available information on antiquated structural systems has occurred despite the fact that most of the methods of analysis and materials used (including steel and concrete) in this country are not much older than 100 years. At the same time as new materials, technologies and methods of analysis have become available and readily embraced by design engineers and the construction industry, previously used systems were more often than not quickly discarded and forgotten.

19th Century Educational Building
Source: Historic American Building Survey

The information that is provided in this presentation is intended to represent the knowledge that has been available at various stages of different methods of construction over the past 100 plus years in the United States. This information, however, cannot be used from a perspective in which any framing systems can be assumed to explicitly correspond to a specific system described in the material presented.

This is because, as is the case now, just because the records indicate a particular structural component should be able to support a given load does not mean that errors were not made during the original construction or as a part of the initial design.

"Oooops."

Source: Theresa McCracken

A good example of the unorthodox use of a system that we will be talking about later in this presentation is the Lorraine Hotel in Philadelphia. In this building hollow clay tiles normally used for flat arch end construction were turned on their sides and simply used as stay in place formwork for a concrete slab spanning between closely spaced steel or iron purlins.

In addition, it is common to encounter some overlap between a previous and more recent method of construction which has resulted in a blending of two otherwise discreet structural systems.

Also, before the ASTM began to standardize construction materials in the late 1890's, the quality of irons, steels and cementitious products varied greatly. Therefore, when dealing with a building that predates ASTM testing, samples of the existing structural materials should be obtained and tested as a part of the structural due diligence effort.
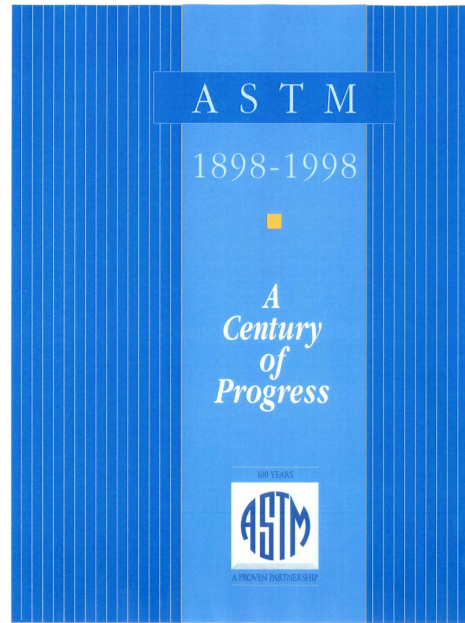
In some instances it is not possible or practical to obtain material strength properties of an existing system in order to complete an analysis using current methods.

However, if the past performance of the structure has been good (i.e. no signs of distress or significant deterioration) then it is very likely that the system is adequate for the continued same future use.

Current Day Jim Thorpe, PA
Source: Historic American Building Survey

The original criteria for the design of antiquated structural systems was a performance based approach based on experience, both good and bad (i.e. failures). The transition to the more recent analytical design approach has come about through the development of strength based formulas based on scientific experimentation and tests.

Structural engineering of buildings as a separate discipline did not exist as late as the 1840's. However, the need for engineers began to grow in the 1850's with the advent of wrought-iron beams which had to be mathematically designed because there was no craftsman's based tradition to provide rule-of-thumb or performance based rules.

In addition, the establishment of the ASCE in 1852 helped to promote the rapid spread of technical information such as records of experiments with cast and wrought-iron performed in England by Hodgkinson and Fairbairn.

American Society of Civil Engineers

1852 – 2002

Building a Better World

It should also be recognized that an existing structural system can often be found to have two different load carrying capacities, one found using the original codes and methods of analysis and another using the current codes and methods of analysis. The differences between these two approaches can typically be explained by the expansion of knowledge in the field of structural engineering.

More often than not comparisons between the original and more current methods of analysis will reveal that the older design was conservative.

In either case, if the properties of the materials can be substantiated it is always possible to analyze an older structure using the latest methods of analysis and most current codes. In most cases in fact the current building code will mandate such an approach.

UNIFORM CONSTRUCTION CODE REHABILITATION SUBCODE

New Jersey Administrative Code
Title 5
Chapter 23
Subchapter 6

Christine Todd Whitman, Governor
State of New Jersey

Jane M. Kenny, Commissioner
Department of Community Affairs

January 5, 1998

© 1997 New Jersey Department of Community Affairs
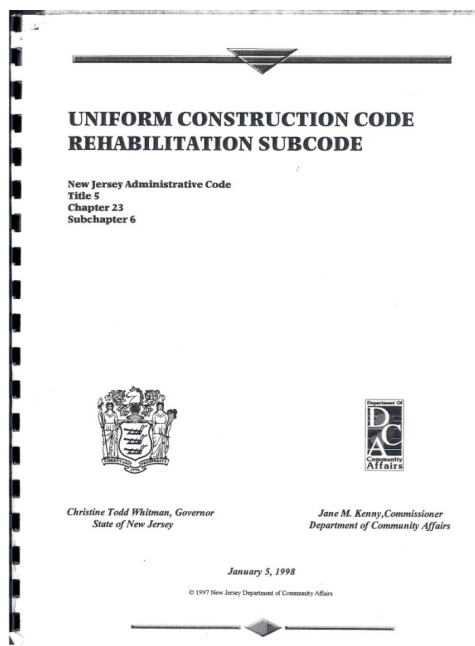
New Jersey State Rehabilitation Building Code

In situations in which it is confirmed that the existing structural system does not have sufficient capacity to support the new loads there are two basic methods that can be used to rectify the condition:

1. Adding new framing members to either independently support the new loads or provide supplemental support of the existing structure, and or

2. Internally or externally reinforcing the existing system.

Strengthening of Existing Slab

The Circumferential or S.M.I System of Reinforced Concrete Flat Slabs

The S.M.I. System of designing reinforced concrete flat plate slabs was developed by Edward Smulski, a consulting engineer from New York City, prior to the 1920's. The system was unique in that the primary flexural reinforcement consisted of concentric rings of smooth reinforcing bars supplemented with diagonal and orthogonal trussed bars placed between the supporting columns and radial hairpin bars located at the columns. (Unless noted otherwise, all images of the S.M.I. System are reprinted with the permission of ACI)
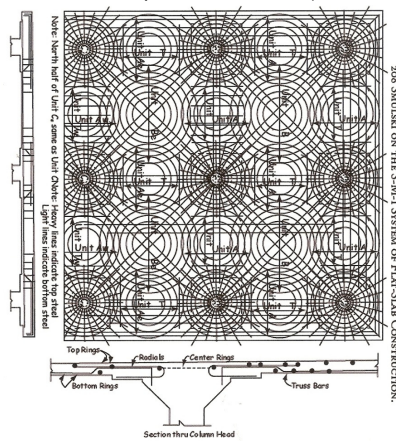
Plan of S.M.I. Reinforcing

I first encountered this type of system while evaluating an existing structure in Philadelphia that had at one time been used as an enclosed parking garage but was currently being used as an office building in the late 1990's. No drawings were available for the structure, therefore small exploratory openings were cut in the slab to reveal portions of the internal reinforcement and slab thickness to enable an analysis of the load carrying capacity of the framed floors.

However, rather than revealing orthogonal reinforcing bars, rings of smooth bars were discovered as a result of the exploratory demolition. A subsequent investigation of the available literature on flat plate construction from the approximate time period the structure had been constructed revealed that the slab was very likely designed and constructed using the S.M.I. System.

Source: Historic American Building Survey

The concentric rings of the S.M.I. System are located in the top of the slab directly above the columns (referred to as Unit C in the available literature), and in the bottom of the slab at the mid-span of what we would now call a column strip (Unit A) and in the bottom of the slab at the mid-span of what is now referred to as a middle strip, or centered in the bay formed by the column grid (Unit B).

There is typically no top reinforcing provided in the middle strip at the intersection with the column strips as is now required by the latest building codes. The concentric rings of bottom reinforcement overlap at the interface zones of Units A and B while the top reinforcement above the column typically overlap the Unit A bottom bars below.
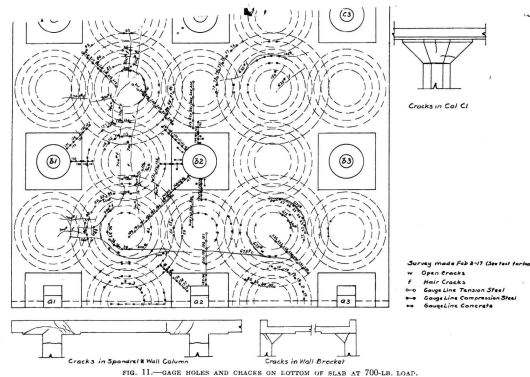
S.M.I. Bottom Reinforcing Plan

The slab is separated into three independent sections as a part of the design of the system. These parts include the column head section (Unit C), the slab between the columns (Unit A) and the central portion of the slab (Unit B). The column head is analyzed as if it were a circular cantilever fixed at the column and loaded uniformly around it's circumference by reactions transmitted to it by the adjacent surrounding components. The slab between the columns and the central portion of the slab are analyzed for positive bending moments only.
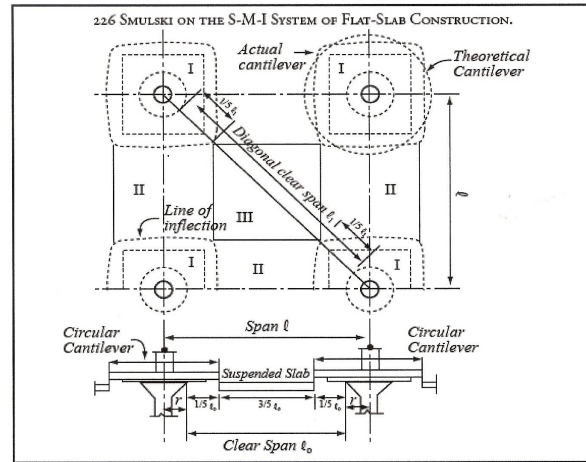
S.M.I Design Basis

The theory behind the design of the S.M.I. System is based on the same flexural theory of reinforced concrete used by all other previous methods of analysis, i.e. bending moments are resisted by internal stress in the concrete, compressive on one side of the neutral axis of the section, and tension on the other.

The primary difference with the S.M.I. System is that the tensile stresses in the structure are offset by the concentric rings of reinforcing bars which resist the tendency of the concrete within the ring to deform/elongate due to the tensile bending forces. In other words, the rings were subjected to hoop stresses (i.e. axial forces acting on the rebar perpendicular to the radial direction of the concrete tension).

The rings consist of smooth bars. The ends of the rings are lapped to develop their full strength. The laps of the concentric rings are staggered to avoid adjacent laps from occurring at the same radial location within the designated Unit.
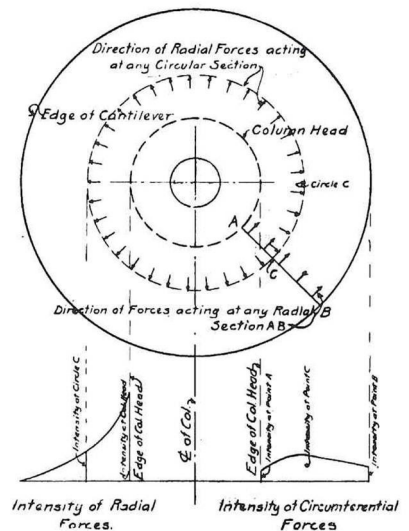
S.M.I Hoop Stress Diagram

Steel Ring Deformation
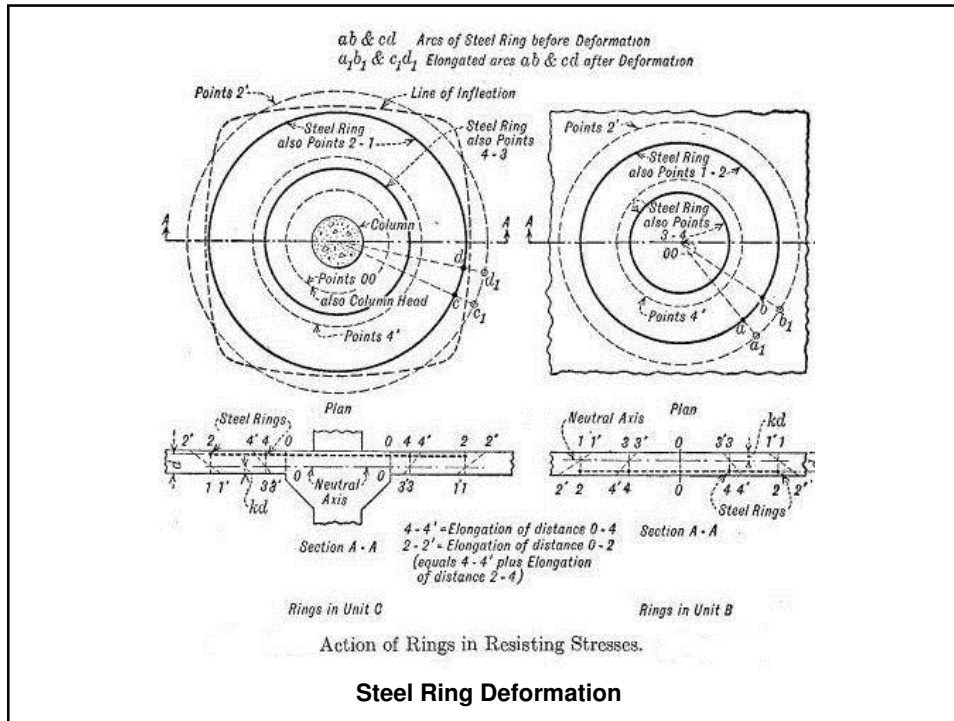
Comments by one of the authors of the 4th Edition of Plain and Reinforced Concrete Volume 1, Sanford Thompson, indicates that the S.M.I System required 20 to 24% less reinforcing than comparable two-way and four-way flat slab systems designed during the same historical time period.

Comparisons between weights of reinforcing for different two-way and four-way flat slab systems provided in the CRSI publication, Evaluation of Reinforcing Steel Systems in Old Reinforced Concrete Structures, does not list the pounds of steel required in a typical interior panel of the S.M.I. System, however, other information concerning this system is provided in the same document.
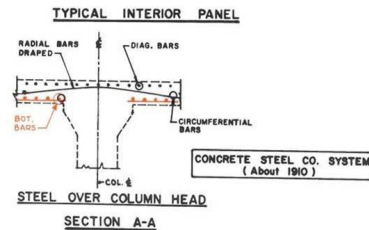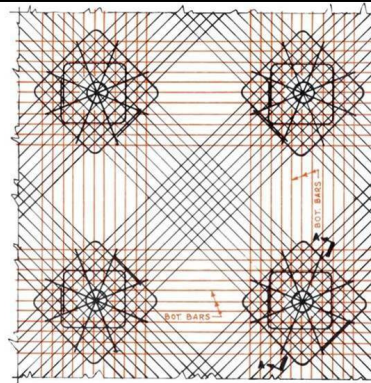
4-Way Flat Slab Reinforcing Plan
Source: CRSI

Load tests of the S.M.I. System were conducted at Purdue University prior to 1920 by Professor W. K. Hatt. The results of these tests were published in the 1918 ACI Journal Proceedings. Stresses within the reinforcing rings were measured using an "extensometer" developed by Professor Claude Berry of the University of Pennsylvania.

The 41-feet x 36.5-feet, 2x2 bay test frame, with cantilevers on three sides and an upturned spandrel beam on the fourth, was loaded using bricks stacked in such a way to prevent arching action of the masonry units. The center-to-center spacing of the columns was 16-feet. All columns included a capital. The slab thickness was 5½-inches. The test frame was loaded from 150 PSF to 950 PSF until failure occurred.
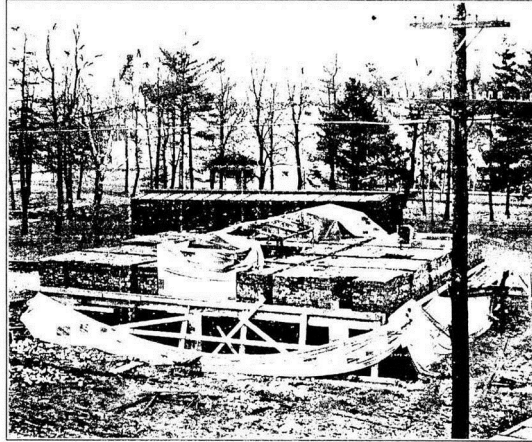
VIEW OF TEST LOAD IN PLACE ON THE SLABS.

S.M.I. Load Test

Working stress formulas that are used to both analyze a S.M.I. slab as well as size the required reinforcement include:

(Unit C) Column Head $2A_s f_s = 6.64(M/jd)$
Where: M = Bending Moment per ½ of the circumference
               As = Sum of the cross-section of rings

Based on assumption that the directions of the bending moments are radial. The circumference of the Unit was typically established as the average of the inflection points for the continuous orthogonal and diagonal moment diagrams between the column spacing's.

(Unit A) Between the Columns $2A_s f_s = 2(M_1/jd)$
Where: M1 = Bending Moment on portion covered by the rings
               As = Area of one section of rings

Based on the assumption that the principle bending moments act primarily in one direction. Span of Unit was typically established as orthogonal distance between the inflection points of the opposing columns.

(Unit B) Center Portion of Slab $A_s f_s = ½(M_2/jd)$
Where: M2 = Bending Moment acting in the distance equal to the diameter of a ring
               As1 = Area of one section of the rings

Based on the assumption that the bending moments act diagonally. Span of unit typically based on diagonal clear span between the inflection points of the opposing columns.

Source: Historic American Building Survey

The principals of circumferential and radial bending moment analysis were also being researched by F.E. Turneaure and E.R. Maurer at the University of Wisconsin in the early 1900's as well. A discussion of their methods of analysis can be found in the Principals of Reinforced Concrete Construction, 3rd Edition.

The available literature that deals directly with the S.M.I. Systems indicates that the method of construction was patented by Edward Smulski. However, a cursory search through the U.S. Patent Office indicates that there were only two patents granted to Smulski, one for a cast-in-place counterfort system for retaining, reservoir and dam walls and one for a two-way, orthogonal reinforced slab system that included encased steel beams.

It is not clear how predominate the use of the S.M.I. System was during both the early 1900's and later in the century. The number of structures that were constructed and the number of structures currently remaining that were built using this system is unknown.

In my opinion, it is not likely that this system was used to a large degree or was very popular because of the assumed difficulty associated with properly fabricating and placing perfectly round and concentrically positioned bars in overlapping top and bottom layers.
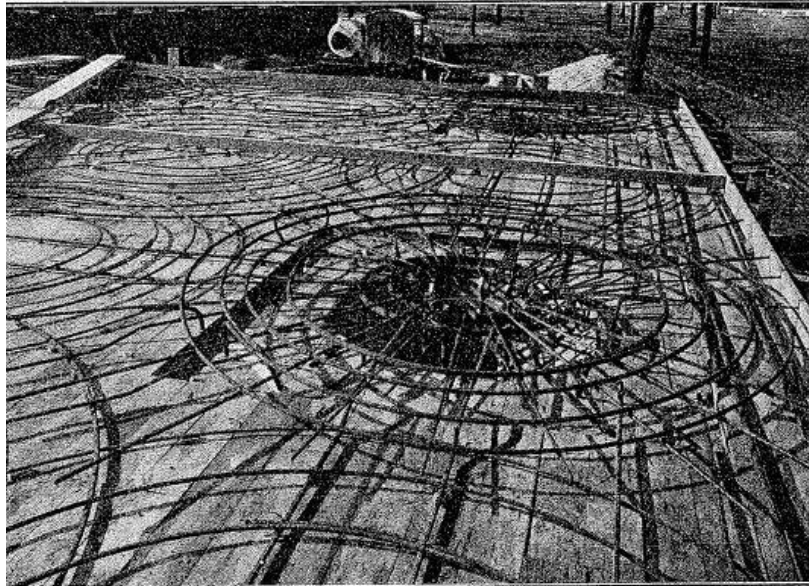
S.M.I Reinforcing Layout
Source: Concrete Engineers Handbook

Prior to the development of the S.M.I. system another very similar reinforced flat slab method of framing was developed and patented in 1911 by Claude Turner. Mr. Turner, who referred to his method of design as the "mushroom" flat slab system, developed the method of construction in Minneapolis, Minnesota.

An article written by Meghan Elliott, which is the source of the image on this slide, provides more information on the system, and can be found in the October 2010 issue of the Construction History Society of America Newsletter.
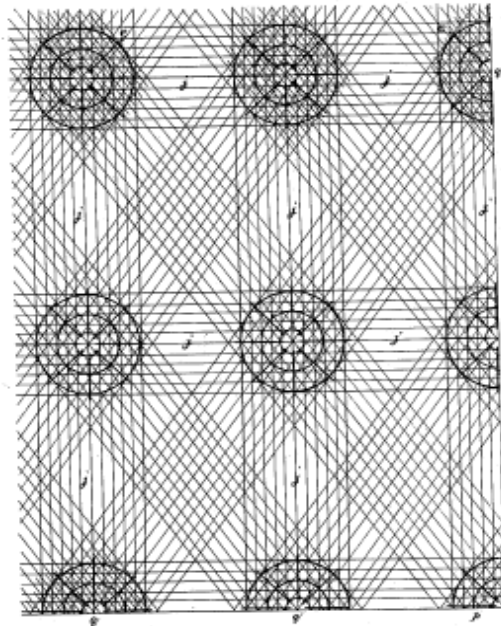
Mushroom Flat Slab System

Clay Tile Arched Floor Systems

Concrete and steel framed floors constructed in the late 1800's and early 1900's often included hollow clay tile arches, which spanned between beams and girders. The arches were typically covered with a concrete topping and often had plaster applied directly to the soffit of the exposed tiles. These types of floor systems were often stronger and stiffer than that calculated by the simple conventional methods of analysis used at the time. In addition, the clay tiles served two purposes; transferring loads to the supporting beams and providing fire protection for the structural steel.
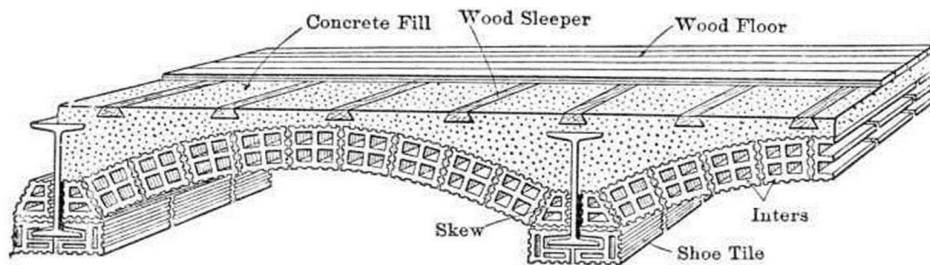
Segmental Arch
Source: Kidder-Parker

There are two basic types of clay tile arched floor systems; **Segmental** and **Flat**. Both systems were constructed using hollow clay tiles of varying sizes and shapes, with internal open cells similar to today's hollow masonry blocks. The typical web and face shell thickness was ½ inch, and all four sides of the closed faces of the tile were also typically scored. The "blocks" were manufactured by a number of different companies, including: National Fireproofing Corporation, Pittsburgh; Henry Maurer & Sons, New York; Whitacre-Greer Fireproofing Co., Waynesboro, Ohio; and Fraser Brick Co., Dallas Texas.
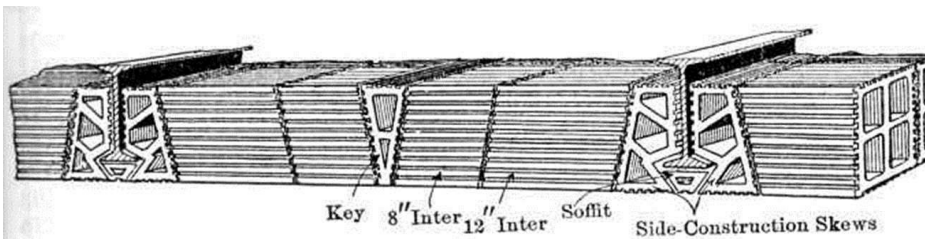
Flat Arch
Source: Kidder-Parker

Flat arch tile units typically varied in depth from 6 to 16 inches. The average dead weight of these units varied from 25 PSF to 58 PSF.

Segmental arch tile units were provided with radial sides so that each tile acted as a voussoir component of the arch. Segmental tiles typically came in 6 and 8-inch depths.

Both types of arches were constructed on timber formwork platforms, which were used to secure the tiles in place during construction. The formwork was typically suspended from timber "jack" beams spanning between and over the tops of the supporting steel beams.
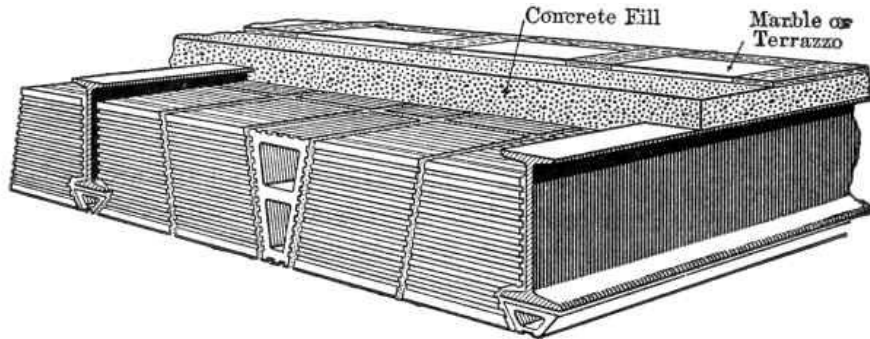
Flat Arch
Source: Kidder-Parker

In a segmental arch, clay tiles are arranged in a shallow profile between adjacent parallel beams as shown in Figure 1. The steel beams were typically held together with tie rods, which helped to resist the outward thrust imposed by the arch on the steel beams, both temporarily during construction and permanently at an end span. The tie rod is not shown in Figure 1. Solid clay bricks were also used in a similar fashion; however, hollow clay tiles typically offered an assembly that was not as heavy as solid brick.
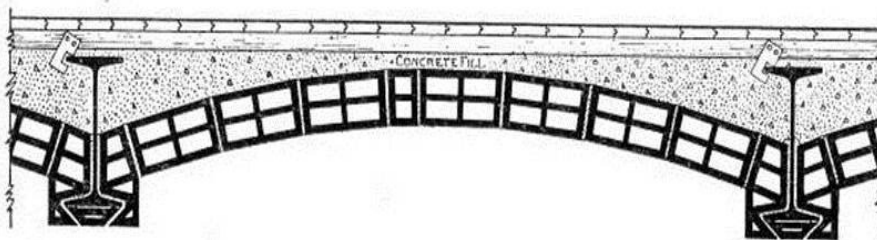
Figure 1
Source: Tile Engineering Handbook

The flat clay tile arch, as shown in Figure 2, transferred the load between the beams acting as a jack arch with a tapered keystone located at the center of the span. Again, the resulting outward horizontal thrust reaction that occurred at the beams was typically resisted via tie rods that were required both temporarily during construction of interior spans and permanently at end spans.
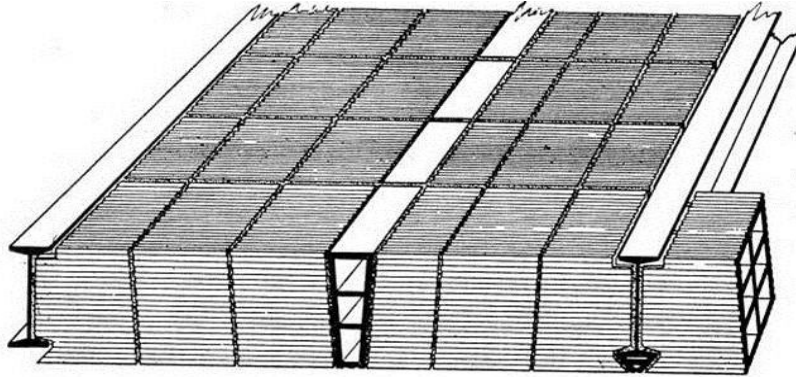
Typical span—End construction flat arch floor.

Figure 2
Source: Tile Engineering Handbook

Another type of flat clay tile arch was the reinforced system shown in Figure 3. For this type of "arch" system, closely spaced internal reinforcing rods were embedded between the tiles near the bottom, which allowed for the entire section to function more as a true flexural member rather than as an arch. This system was also referred to as the Natco "New York" reinforced flat arch. It served as a precursor to one and two-way tile joist systems, which will be discussed later.
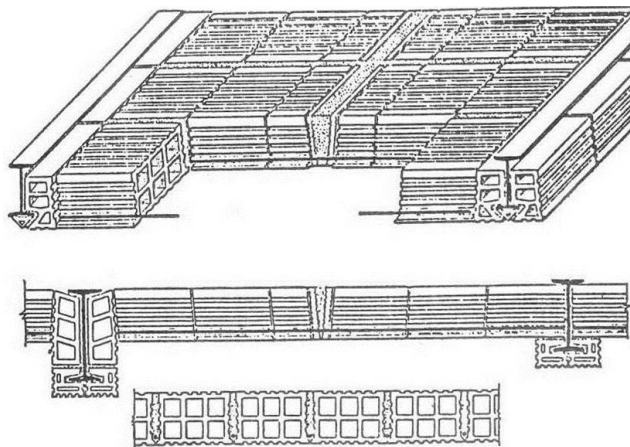
Figure 3
Source: Kidder-Parker

A third type of clay tile arch construction includes the Guastavino timbrel arch, which consists of a series of laminated layers of tile slabs that were laid and bonded together with Portland cement mortar to form solid large-span domes. As this type of construction was not typically used in conjunction with steel floor framing, it will not be discussed as a part of this presentation.

Guastavino Timbrel Arch
Source: Historic American Building Survey

Standard flat arches can be classified into two groups: End Construction and Side Construction. End construction consisted of laying the axis of the tiles' hollow cells parallel to the direction of the span. Side construction consisted of laying the axis of the hollow cells perpendicular to the span to the span.
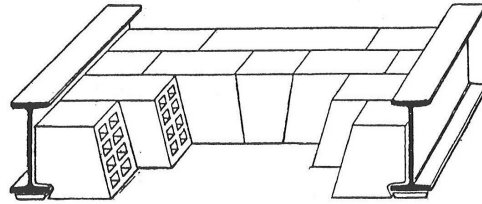
End Construction
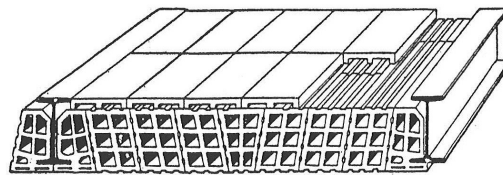
Side Construction

Combination side and end construction can also be encountered. The tie rods used to resist the arch thrust forces were generally placed approximately 3 inches from the bottom of the beams in flat arches. (**Source of Images:** Freitag)

Flat Arch Side Construction

Mid Span Keystone Tile   End Soffit Tile at Support Beam
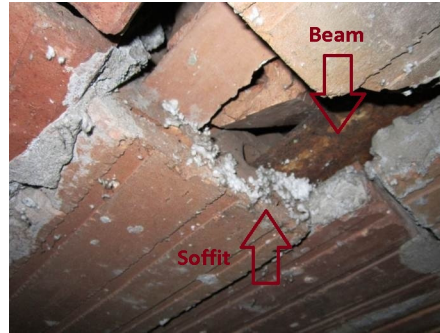
Temporarily Shored Flat Arch End Construction Framing

Typically tie rods were ¾ inch in diameter and were spaced as required to resist the specific thrust of the given arch span, although a minimum spacing of fifteen times the width or eight times the depth of the supporting steel beam was recommended.

Tie rods at an end span were required as there was no opposing thrust present at the outside face of the spandrel beam. At interior spans, with adjacent arches present on either side, tie rods were only required during construction, but were typically left permanently in place.

For this reason, when modifying an existing building constructed with clay tile arches that involves the removal of an interior span, the capacity of the remaining adjacent span's rods should be verified to assure that the end span conditions created on either side of the new opening will remain stable.

The total arch thrust, net area of the tie rods and maximum spacing for both a flat and segmental arch can be found as indicated below:

Total Thrust (in pounds) per Arch Panel:

$$T = (3wD^2/2R)L$$

Where;   w = uniform dead + live load on arch in PSF
              D = arch span in feet
              R = effective rise of arch in inches
                  (typically 2.4 inches less than the depth of the clay tile units for flat arches)
              L = length of the floor beam supporting the arch in feet

---

Total net area of tie rods per panel (square inches): $A = T/f$

        Where;   f = allowable unit stress (typically 18,000 psi)

Maximum spacing of tie rods (feet): $S = (af)/(3wD^2/2R)$

        Where;   a = net area (square inches) of tie rod

Rod Diameter	5/8 inch	3/4 inch	7/8 inch	1 inch
Net Area (a)	.202	.302	.420	.550

Hollow Clay Tile Arched Framing
Source: Historic American Building Survey

Shored Flat Arch Mechanical Chase:

Installed Steel Frame for Arch
Action Continuity at Same Opening:

Source: Structural Engineer Magazine

Flat arch spans typically varied from 3 feet to 10 feet and were capable of supporting safe uniform loads between 126 and 1,400 PSF as indicated in Table 10-31 from the Principals of Tile Engineering Handbook of Design. Segmental arch spans typically varied from 5 feet to 10 feet and were capable of supporting safe uniform loads between 465 and 2,149 PSF as indicated in Table 10-32 from the Tile Engineering Handbook.

TABLE 10-31

FLAT ARCH TOTAL SAFE LOADS IN POUNDS PER SQUARE FOOT

(Dead plus Live)

Factor of Safety of 7

Depth of arch,	6"	7"	8"	9"	10"	12"	15"
Net sectional area, sq. in.	27	27	27	27	36	36	36
Average weight, psf.	26#	30#	32#	36#	40#	48#	56#
Span, ft. and in.							
3'-0"	420	490	560	630	933	1120	1400
3'-3"	357	417	477	537	795	954	1193
3'-6"	308	360	411	462	685	823	1028
3'-0"	268	313	358	403	597	716	895
4'-0"	236	276	315	354	525	630	786
4'-3"			244	279	314	465	558	697
4'-6"		218	249	279	415	497	622
4'-9"			223	251	372	447	558
5'-0"			201	227	336	402	504
5'-3"			182	205	305	365	457
5'-6"				187	277	333	417
5'-9"				171	254	305	381
6'-0"				157	233	280	350
6'-3"					214	258	322
6'-6"					198	238	298
6'-9"						221	276
7'-0"						206	257
7'-6"						178	223
8'-0"						157	197
8'-6"							174
9'-0"							155
9'-0"							140
10'-0"							126

TABLE 10-32

SEGMENTAL ARCH, TOTAL SAFE LOADS① PER SQUARE FOOT

(Dead② plus Live)

(Factor of Safety of 7)

Span of Arch Ft.-In.	Rise② in.	Arch, psf. 6-in.	Arch, psf. 8-in.	Span of Arch Ft.-In.	Rise② in.	Arch, psf. 6-in.	Arch, psf. 8-in.
5'-0"	¾	735	878	8'-0"	¾	465	556
	1	983	1174		1	614	734
	1¼	1198	1431		1¼	750	895
	1½	1405	1678		1½	885	1057
	1¾	1618	1933		1¾	1013	1210
	2	1799	2149		2	1125	1343
5'-6"	¾	667	797	8'-6"	¾	437	522
	1	890	1063		1	577	689
	1¼	1088	1300		1¼	704	841
	1½	1292	1543		1½	832	994
	1¾	1465	1750		1¾	952	1137
	2	1645	1964		2	1063	1270
6'-0"	¾	611	730	9'-0"	¾	412	492
	1	814	972		1	544	650
	1¼	995	1188		1¼	671	801
	1½	1180	1410		1½	784	937
	1¾	1341	1601		1¾	897	1072
	2	1502	1794		2	1003	1198
6'-6"	¾	577	689	9'-6"	¾	390	466
	1	750	895		1	515	615
	1¼	928	1108		1¼	634	757
	1½	1084	1295		1½	747	893
	1¾	1244	1486		1¾	849	1014
	2	1384	1653		2	949	1133
7'-0"	¾	534	637	10'-0"	¾	370	442
	1	695	830		1	488	583
	1¼	860	1027		1¼	602	719
	1½	1007	1202		1½	709	847
	1¾	1153	1377		1¾	810	968
	2	1290	1541		2	905	1081
7'-6"	¾	497	594	10'-6"	¾	357	427
	1	647	772		1	464	554
	1¼	800	956		1¼	572	683
	1½	946	1130		1½	674	805
	1¾	1075	1284		1¾	770	920
	2	1202	1436		2	858	1025

Both of the tables from the Principals of Tile Engineering Handbook of Design were based on load tests, which were reduced by a substantial safety factor of 7. When evaluating existing clay tile arch systems it is recommended that the initial load capacity rating be based on published tables. If, however, this simplified approach indicates that the allowable load carrying capacity is not sufficient for the new reuse requirements, then it is possible to calculate an increased strength by taking advantage of the inherent composite capabilities of the clay tiles and the concrete topping. However, the assumed load capacity of the arched floor should not exceed the capacity of the supporting steel beams.

The principal disadvantage of tile arch floor construction was the difficultly of adapting standard sizes to irregularly shaped spaces. In addition, tile arches are more easily weakened by holes and penetrations than a monolithic floor system. Furthermore, it was difficult to place mortar in end construction, i.e. when the open cells were placed end to end.
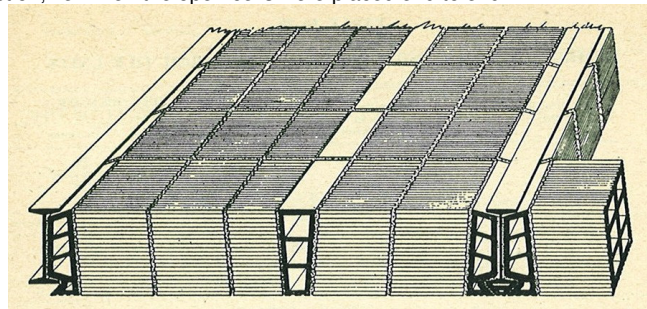

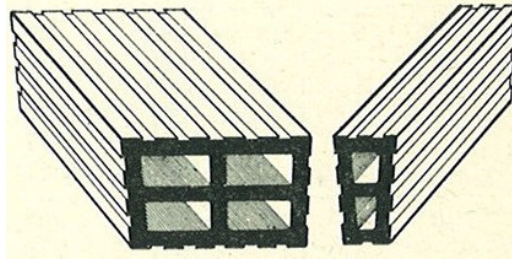**Combined End and Side Construction**
Source: Tile Engineering Handbook

Also, for end construction, if a single tile was removed in a row, then the remaining tiles became unsupported unless the scored sides of the tile were mortared in with the adjacent rows of tiles. Because side-constructed arches (arches in which the scored sides of the tiles were placed adjacent to one another, transverse to the arch span) were more conducive to placing mortar between the tiles, this type of construction had an advantage over end construction.

However, tests conducted during the period of time in which clay tiles were used extensively indicated that tiles were much stronger in an end construction application as opposed to a side construction configuration. Finally, tile arch construction was susceptible to poor workmanship because the quality of the work could only be observed from the top and not from below during construction until after the formwork was removed. (**Source of Images:** Tile Engineering Handbook)
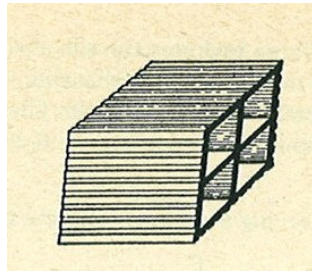
Interior End Flat Construction Tile   **Interior Segmental Construction Tiles**

The Herculean floor system was another type of hollow clay tile flat arch that was manufactured in only one configuration in depths of 6, 8, 10 and 12-inches. The system was capable of spans up to 23-feet. The T-irons that were located as noted in the image below were more than likely intended to allow for load transfer between adjacent rows of tile.
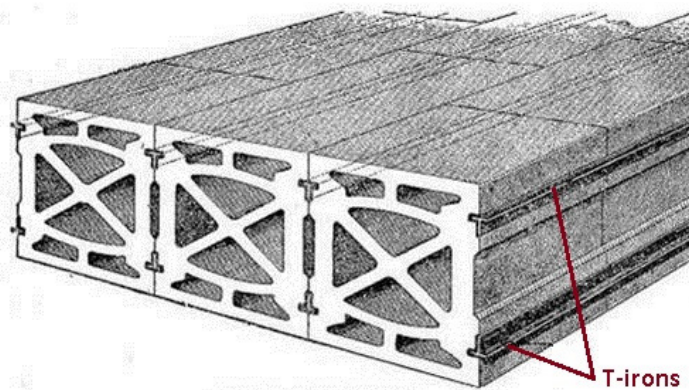
Source: Building Construction and Superintendence

The Excelsior system was similar to the Herculean system.
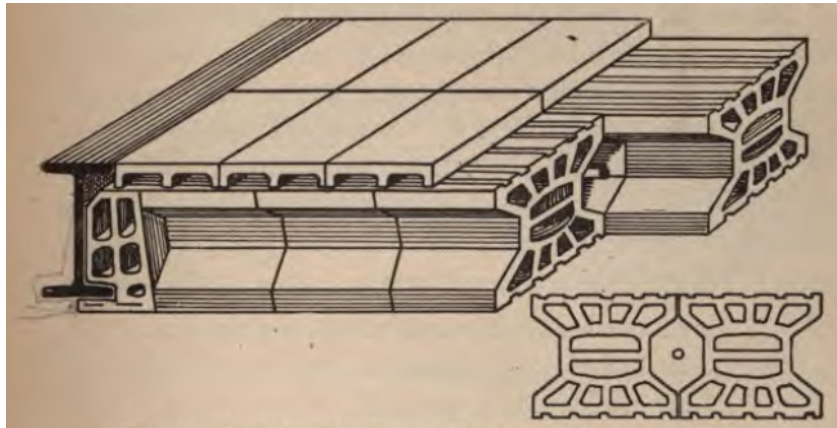
Source: Freitag

Hollow clay tile was also used for roof construction as illustrated in this slide which shows a system that was referred to as Book tiles.
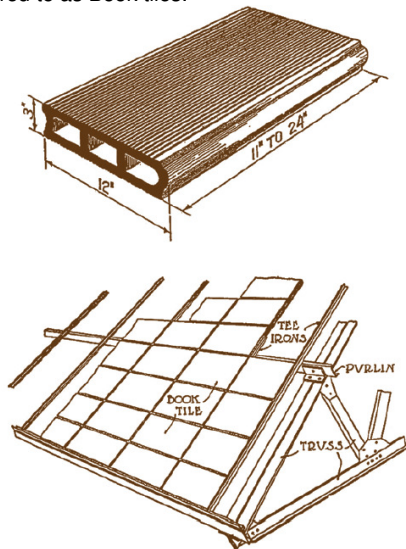
Source: Historic Preservation Education Foundation

Clay tile was also installed as a solid ceiling beneath wood and iron framed structures to improve the fire resistance of the framing.

Terra cotta lumber, a kaolin clay material fired with sawdust , was also available in the early 1880's and could be sawed, drilled, carved and nailed similar to conventional wood lumber.

Source: U.S. Patent Office

Hollow concrete blocks were also used in a fashion similar to hollow clay tile beginning in the late 1800's. Below is photo of a concrete block system used at the Peabody Library in Baltimore.

Source: Sara Wermiel

NISTIR 7563
DRAFT For Public Comments

Best Practice Guidelines for Structural Fire Resistance Design of Concrete and Steel Buildings

6.2.4.3 Fire Modeling with Computer Programs

Three classes of fire models have been programmed for computer analyses of gas temperatures in a room or compartment.

1. Empirical models are based on correlations with full-scale fire experiments.

2. "Zone" models treat the compartment as either one or two homogeneous volumes in which conservation of mass, momentum, and energy are computed in a dynamic process.

3. "Field" models are based on computational fluid dynamics (CFD). Field models use a three-dimensional grid to model the changes in gas and surface temperatures over the room volume.

Some of these models analyze for a user-specified heat release rate; some predict the heat release rate based on user-specified material properties of the combustibles. All three classes of fire models address fully developed fires. The choice of model depends on the reliability of fuel and thermal property data and the desired degree of accuracy.

For compartments with size, aspect ratio, and contents that are comparable to those used for full-scale experiments, the empirical models are appropriate tools.

Zone models are appropriate for a well-stirred combustion chamber. Some zone models, such as COMPF2 (Babrauskas 1979), were designed specifically to model fully developed fires. Others are more general and used primarily to model smoke and heat movement through structures. These models have been applied with mixed success to the prediction of fully developed conditions (Luo, He, Beck 1997 and Buchanan 2001).

Field models can reproduce flashover conditions (NIST 2005f), but the degree of accuracy depends on the quality of the input data. Validation studies (e.g., US NRC 2007) have been performed to quantify the accuracy of the various types of models. The designer should review the documentation for the individual fire model for references to validation work to judge if the model is appropriate for the task at hand.

An international survey of available computer programs for zone and field model that predict fire and smoke growth and spread can be found in Olenick and Carpenter (2003).

NIST

National Institute of Standards and Technology
Technology Administration, U.S. Department of Commerce

One and Two-Way Clay Tile and Unit Masonry Joist Systems

This portion of the presentation deals with one- and two-way clay tile and unit masonry joist systems. In such systems, the individual units were laid in such a way as to form trenches that allowed reinforcing bars to be placed in the bottom of the resulting joist cross sections. This method of construction is very similar to the more recent pan joist system; however, unlike steel pans, the clay and masonry units were left in place for added strength and fire resistance, and to provide a flat ceiling surface.

One-Way Clay Tile Joists

Proprietary one-way floor systems included the Natcoflor and Republic Slagblock systems. Proprietary two-way floor systems included the Schuster, Smooth-Ceiling, Sandberg and Republic Slagblok systems. All of these employed regularly shaped units of varying size and depth that resulted in a uniform modulation of joist sizes and spacings. However, during the 1930's, a patented "wide-center" system was introduced for both one-way and two-way framing that allowed for wider clay tile units to be placed at the center of the span and narrower units to be placed at the end of the span. This resulted in wider joists near the supports, which in turn resulted in greater shear capacity at the end of the span, similar to the more recent tapered end pan joist system.

With the exception of the Smooth-Ceiling and Sandberg systems, the clay tile and unit masonry could be constructed to span between steel beams, concrete beams or loadbearing walls. In addition, most of the systems could be placed with or without a concrete topping. When a monolithic concrete topping was used, the thickness typically varied from 1½ inches to 3 inches. Joists were typically analyzed as T-beam sections when a monolithic topping was used. With the exception of the Natcoflor system, joist widths typically varied from 4 inches to 6 inches. (**Source of Image:** Tile Engineering Handbook)
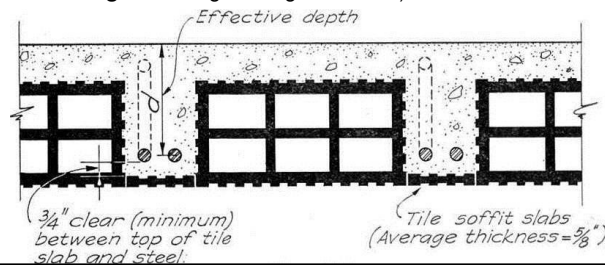
Typically ¾-inch clear cover was provided between the square or round deformed reinforcing bars and the adjacent tile or masonry units or the top and bottom of the exposed concrete surface of the joist. It was typical to use straight bottom bars and trussed top bars bent down to align with the bottom bars near the center of the span. When a concrete topping was used, it was typical for temperature/shrinkage reinforcement to be provided orthogonal to the joist span. The amount of this steel was typically 0.0025 times the gross cross-sectional area of the topping, and it was spaced at no more than 18 inches on center.

One-way systems were very efficient for spans over 12 feet and were used very frequently for spans up to 24 feet with loadings that ranged from 40 to 125 PSF, and up to 18- and 20-feet spans for heavier loadings. For two-way systems, and at the end of the span for one-way systems, it was common for the open webbed ends of clay tiles (or masonry units) to be filled with cardboard or metal inserts to prevent concrete from flowing into the voids in order to minimize the dead load of the slab. (**Source of Image:** Tile Engineering Handbook)
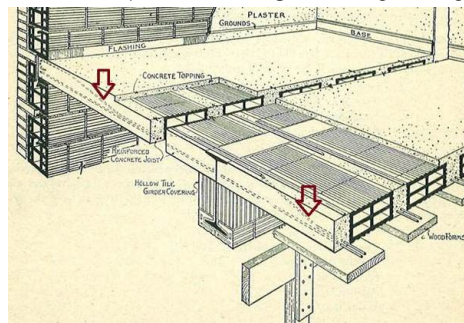
The **Natco** floor system used specially manufactured clay tiles with curved flanges that allowed only the bottom of the tiles to be exposed as the ceiling soffit. Other one- and two-way clay tile systems could be formed and cast either with the bottom of the concrete joist exposed or with tile soffit pieces along the bottom of the trenches that resulted in a uniform tile ceiling soffit. The **Natco** floor joists were no more than 2 inches in width, spaced at 13 inches on center, with a depth that varied from 4 inches to 12 inches (Figure 1). The joists were typically cast using cement grout consisting of one part cement and two and one-half parts sand. A composite concrete topping was not required above the tiles in order to attain the maximum load-carrying capacity of the system. (**Source of Image:** Tile Engineering Handbook)
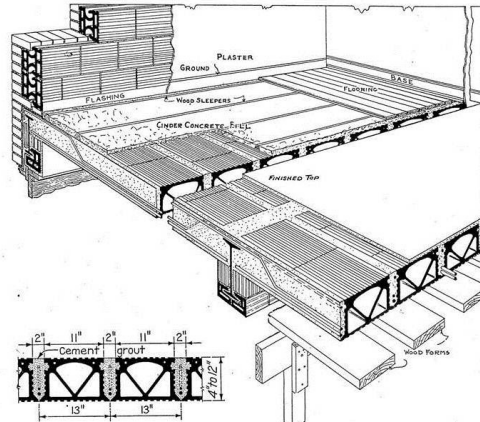
Figure 1

Another one-way tile system that can be categorized as a method of joist framing was the **Faber floor** system.
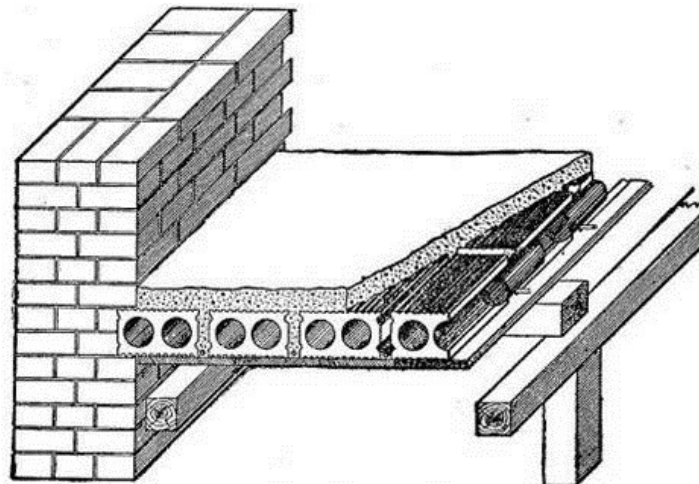
Faber System of Tile and Concrete Construction.

Source: Building Construction and Superintendence

The **Schuster** two-way system (Figure 2), which was patented in 1915, used clay tiles that were 12 inches x 12 inches or 16 inches x 16 inches and had depths of 4, 6, 8, 10 or 12 inches. The joists were typically spaced at 16 inches on center or 20 inches on center; however, tiles could be doubled up to allow for joist spacings of 28 or 30 inches on center. This two-way system was typically used in square bays or rectangular bays in which the longer span was not more than 50% greater than the shorter span. (**Source of Image:** Tile Engineering Handbook)
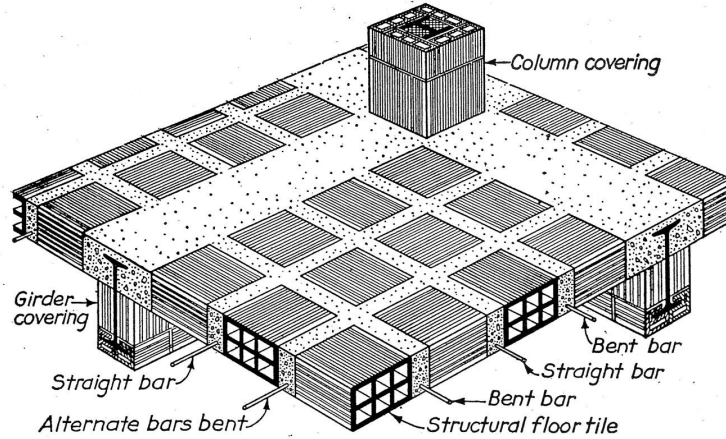
Figure 2

The **Republic Slagblok** system could be installed in either a one-way or two-way configuration. The slagblok unit measured 8 inches x 16 inches and came with one open end and one closed end. Each unit was placed in combination with another slagblok to form closed cells that were 16 inches x 16 inches. Slagbloks came in 3-, 4½-, 6-, 7- and 8-inch depths. The concrete ribs or joists were typically 4 inches in width and spaced at 20 inches on center. Typical spans for this system varied from 15 to 25 feet. In fact I have seen similar one-way joist systems constructed as recently as the 1970's using regular concrete masonry units.

Two-Way Slagblok Joists

The **Smooth-Ceiling** system, which was patented in the 1930's, and the similar **Sandberg** system both eliminated the need for beams or drop panels by employing embedded internal steel shear reinforcement around either structural steel or reinforced concrete columns. Typically both systems eliminated all tiles from around the column to enable this area to be cast as solid concrete. Although load tables, which included considerable factors of safety, were provided by the manufacturers, the actual design of the joists was accomplished using conventional working stress methods of analysis that were available at the time. Moment and shear coefficients were typically employed to establish the maximum positive and negative moment and end shear design envelopes.

Even though load tables and methods of analysis are available for all of the above clay tile and unit masonry systems, when one encounters any of these same systems in an existing building, and there are no original drawings available, it is difficult to determine what the internal reinforcement is and subsequently the load carrying capacity of the system. (**Source of Image:** Tile Engineering Handbook)
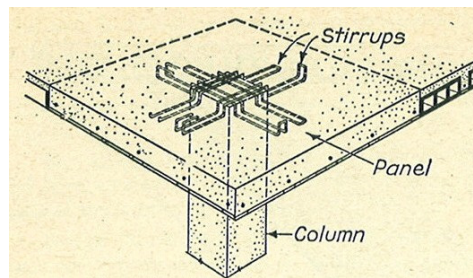
Prefabricated Clay Tile & Concrete Block Framing Systems

As discussed previously, one and two-way joist framing systems were constructed using clay tile and masonry units, which were first arranged and supported on formwork to enable placement of internal reinforcement and infill and topping concrete in situ. In addition, similar modular clay tile and masonry units were also constructed offsite into prefabricated beams and slabs that could be delivered to the job site. This method of construction ultimately progressed to solid precast concrete units, which will be addressed later in this presentation.

The prefabricated clay tile systems included both one-way beam and slab construction and one-way slab construction. The one-way beam system involved the placement of prefabricated beams spaced parallel to each other at regular intervals between already constructed load-bearing walls or steel beams.

The areas between the beams were then infilled with tiles that were capable of spanning between each adjacent beam. The one-way slab system involved prefabricated slab units that were placed directly adjacent to each other, spanning between previously constructed load-bearing walls, steel beams or joists.

Both the beam and slab systems included a site-cast concrete topping, which was poured over the beams and filler tiles or one-way slabs.

The prefabricated beam and slab systems offered the advantage of not having to construct supporting formwork before the framing could be erected; however, shoring in the center of the span was sometimes employed to increase the clear span capability of the members through composite action with the site-cast topping.

Examples of these clay tile systems (Figures 1-6; **Source of Images:** Tile Engineering Handbook) included on the following slides are: the **"T" Beam Floor**, the **"U" Beam** System, the **Joistile** System, the **Sheffield Floor** System, the **Adel Joistile** System, the **Kalex Floor** System, the **United Floor** System and the **Tilecrete Floor** System. Some of these systems required filling the joints between the adjacent ends of the clay tile units with mortar, while other systems allowed the ends of adjacent tiles to butt up against each other. All of the prefabricated beam and slab systems, except for the infill tiles, included internal longitudinal flexural reinforcement for positive moment resistance. Negative moment reinforcement for continuity across a supporting wall, beam or joist was also sometimes placed in the site-cast topping. None of the units included shear reinforcement. Table 1 summarizes all of the clay tile systems mentioned above. The slide is a photo of a residential Kalex floor.

Source: Jeff Poole

One point I would like to make before the next few slides is that you will notice that I have included a number of slides in this presentation in which only an image and name of the system is provided without much corresponding structural or technical information. This is because it has been my experience that when you do encounter an archaic system that you are not familiar with, the biggest initial hurdle you face is identifying the type or specific name of the system. Once you know the name of the system it is must easier to research and look for any available information on the same system. As a result I developed an Illustrated Dictionary of antiquated structural systems, which is available at the STRUCTURE Magazine website under the same Continuing Columns section of the website where all of the original articles are archived. This slide is an illustration from one page of the dictionary. If you know of any additional systems not included in the dictionary please let me know**.**

Source: Granco

Source: Truscon Buildings

Source: H.H. Robertson

Source: Truscon Steel Co.

Figure 1

Perspective view of "Precast 'U'-Beam Floor" and sectional details showing optional erection methods with span tile webs up or down as desired.

Figure 2

Perspective view of standard "Precast Joistile" floor system and typical sectional detail.

Figure 3

Perspective view of "Sheffield" precast tile floor system and sectional detail.

Figure 4

Perspective view of "Adel" precast joistile floor system and typical slab section.

Figure 5

Perspective view of precast "Kalex" tile floor system and typical floor sections.

Figure 6

TABLE 1							
Prefabricated Clay Tile One-Way Beam and Slab Systems							
System	Figure	Regional Use	Mortared Joints	Beam Spacing or Slab Width	Beam Depth (including topping) x Width or Slab Depth	Typical Span	Notes
"T" Beam	1	Southwest, Midwest	Yes	18" to 30"	9½" x 8"	24-ft.	1, 3
				18½" to 22½"	7½" to 8" x 8"		2, 3
"U" Beam	2	Texas, Oklahoma	No	28½"	8½" to 12" x 7" to 8"	14-ft.	4, 5
Joistile	3	East, Southwest	Yes	12"	4" and 5"	8-ft.	4
					6"	12-ft.	
Sheffield	4	Midwest, North Central	Yes	8"	5"	18-ft. to 22-ft.	6
Adel Joistile	5	Midwest	Unknown	12"	5"	Unknown	4
Kalex	6	Ohio, Pennsylvania, New York, Illinois, Wisconsin	No	12"	4" and 6"	Unknown	7, 8, 9, 10
United	None	New York, New Jersey	No	Unknown	Unknown	30"	11, 12
Tilecrete	None	Missouri	No	16"	4" and 6"	12"	13, 14

Notes:
1. Included the use of unreinforced, 4" thick Filler Span Tile for one-way span between beams.
2. Drop-in Filler Tile was used for flush ceiling applications.
3. System load tested at Iowa State College; Engineering Experiment Bulletin No. 286.
4. Longer spans were possible with the use of center span shoring.
5. Included the use of unreinforced, 2" thick ribbed Filler Tile for one-way space between beams.
6. Patented June 1936 by Professor Walter M. Dunagan, Iowa State College.
7. Patented 1937 by D.D. Whitacre, Waynesburg, Ohio.
8. System also used as vertical wall element.
9. Reinforcement included bolted rods, which implies an applied pretensioning force.
10. Load tests of 4" slabs conducted by Professor George E. Large, Ohio State University, for the Rochester, NY Building Board in 1939.
11. Unreinforced slab system used in conjunction with open web steel joists.
12. System used in conjunction with a topping slab that provided composite action with steel joists.
13. Patented system used in conjunction with open web steel trusses; however, tiles were supported on bottom chord, which allowed a concrete topping to be placed that encapsulated the trusses, resulting in concrete ribs capable of spanning up to 24 feet.
14. Tested in 1939 at the National Bureau of Standards; BMS Report No. 16.

Similar prefabricated beam and slab systems were also developed from modular concrete block. Most of these systems used conventional internal reinforcement for flexural strength; however, a few were developed using prestressed bars and strands. Probably the most widely used masonry block product in the eastern U.S. during the 1950's was the **Dox Plank** system, which was invented by Doc Vander Heyden (Figure 7). This product was manufactured with recessed slots in the bottom of the block to allow for the flexural reinforcement to be grouted into the bottom of the plank. There was no mortar required between the adjacent ends of each block.
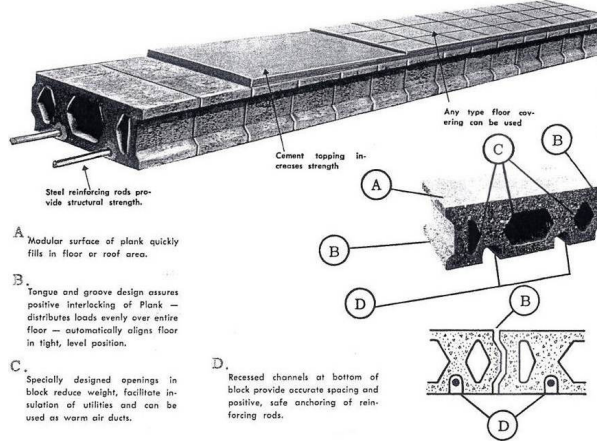

Figure 7

Source: NCMA

Other cross-sectional variations of the Dox Plank were developed by members of the Dox Plank Manufacturers Association. Figure 8 shows an example of an alternate block that differed from that originally developed by NABCO. In this case, the internal reinforcement was completely encapsulated by the block by means of a continuous sleeve. It is not clear whether the continuous reinforcement in the sleeves was grouted in place, or if the bars were threaded at each end of the plank so that the modules could be precompressed together via tensioning of the bar as it was tightened against each end of the member using a nut.
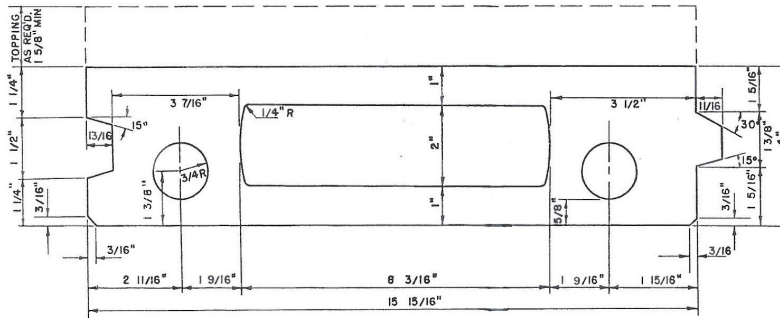
Figure 8
Source: NCMA

Flexicore, a product similar to the NABCO Dox Plank, was also available in the 1950's. A hollow core plank is still manufactured today under this same name; however, the current product is a true precast, prestressed concrete member.

In the 1950's, the consulting firm of Bryan and Dozier and the Nashville Breeko Block Company designed and constructed prefabricated, post-tensioned concrete block beams. This method of construction resulted in the first linear prestressed structure to be built in the US - the Fayetteville Tennessee High School Stadium - and the first prestressed bridge to be built in the US - at Madison County, Tennessee. This method of construction was made practical and economical by the Roebling Company through the development of high-quality tendons that could be bonded without expensive end anchorages.
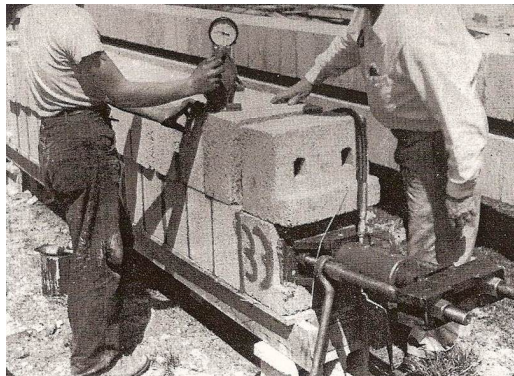
Breeko Block Beam
Source: Ross Bryan

The Breeko Block system was further refined through the use of external, deflected tendons. However, by the late 1950's, this system was replaced by precast, pretensioned concrete members.

Other, more obscure examples of prefabricated modular concrete block beams and slabs include a prestressed bar system developed by P.H. Jackson of California in 1987; a prestressed wire system developed by C.W. Doering in 1888; a system patented by K.E.W. Jagdmann in 1919; a stressed reinforcement system patented by Albert Stewing and Stefan Polonyi in 1967; and a tensioned, Y-shaped block system patented by Hossein Azimi in 1987.

All of the tile and concrete block systems were designed based on the basic reinforced masonry and concrete beam analysis theories of their era. Load tables were also commonly developed and published by most of the manufacturers.

The problem with all of the above systems, when one encounters them in a building, is that in the absence of existing drawings it is difficult to determine the internal reinforcement and, subsequently, the load carrying capacity of the system.
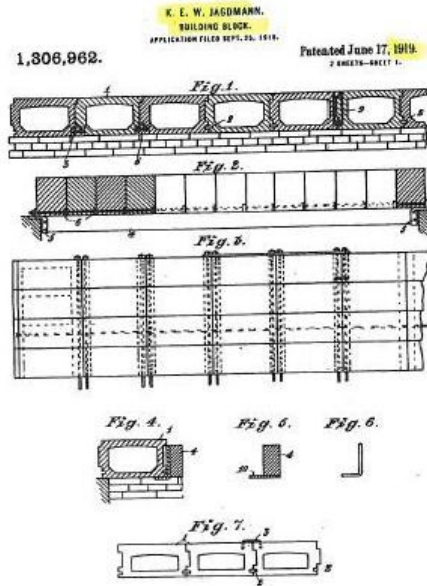
Source: U.S. Patent Office

Precast Concrete Framing Systems

As previously discussed, contractors prefabricated modular clay tile and masonry units off site into beams and slabs that could be delivered to the job site. This method of construction ultimately progressed to solid precast concrete units.

Modern Day Precast Double Tees
Source: Osco Construction Group

In the 1950's, one of the most prevalent precast concrete systems in general use was the **F&A** System. This system (see Figure 1) used conventionally reinforced precast concrete inverted T-joists spaced at 28 inches on center, which supported concrete block filler slabs. The entire assembly then received a 2-inch, cast-in-place concrete topping, which acted compositely with the precast joists. (**Source of Image:** Nitterhouse Precast Concrete)
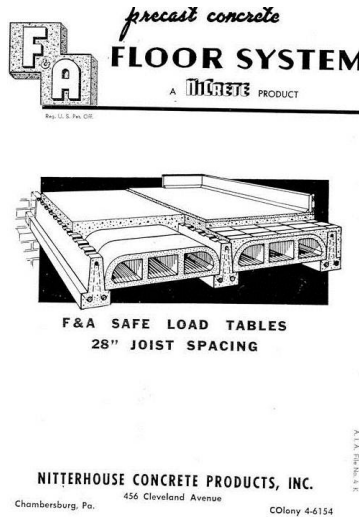
Figure 1

Prior to the F&A System, Peter Rutten developed and patented a similar system in the 1930's (see Figure 2).
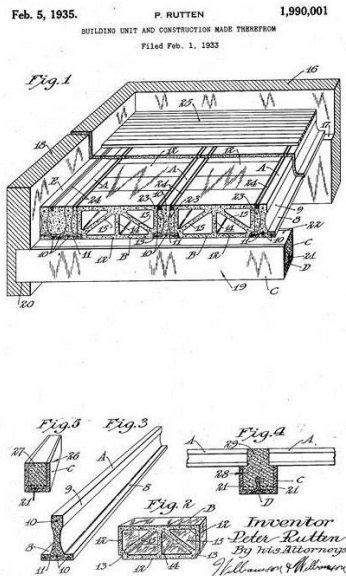
Figure 2
Source: U.S. Patent Office

Another system similar to the F&A system was the precast joist and block Omnia floor and roof framing system. (**Source of Images:** Omnia Industries)
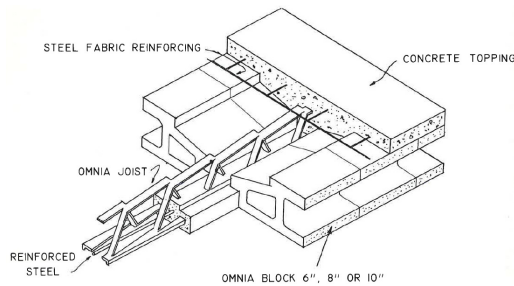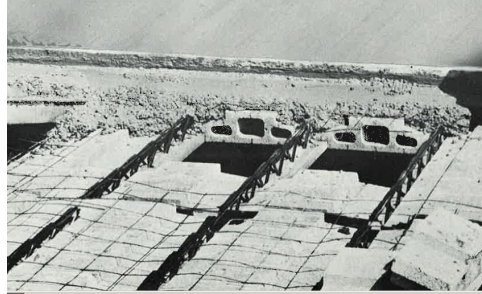
The **F&A** precast joists were available in depths of 6, 8, 10 and 12 inches and were capable of spanning anywhere from 6 feet to 36 feet for load capacities from 30 to 900 pounds per square foot, depending on the span, depth of joist and reinforcement. The ends of the precast joists could be cast integral with a site cast concrete beam or bear directly on either precast concrete girders or steel beams.

The **F&A** System included filler blocks that could be either placed flush with the bottom of the joist or recessed at the same level as the bearing ledge of the joist. Similar precast concrete systems that were in use during the same approximate time period included **Tee Joists** and **Keystone Joists. Tee Joists** (see Figure 3) came in depths of 16 and 20 inches and were typically prestressed.

Tee Joists ultimately evolved into Single Tees, Quad Tees and the Double Tee members that is still in common use today.
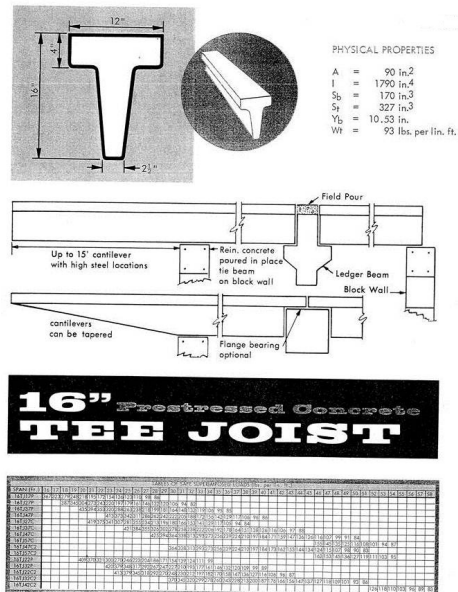
Figure 3

Source: Nitterhouse Precast Concrete

Keystone Joists (see Figure 4) were available in 8- and 12-inch depths and could be either conventionally reinforced or prestressed. (**Source of Image:** Nitterhouse Precast Concrete)

Figure 4

Channel Slabs were another prevalent precast concrete member in the 1950's. These were typically used for roof construction between supporting precast beams or steel members. The slabs were typically 24 inches wide and 1 inch thick, with 3½-inch-deep by 2-inch-wide, down-turned edge "flanges". This product was capable of spanning up to 9 feet and supporting up to 60 pounds per square foot of superimposed load.

Channel Slab
Source: Mid Con Products

Examples of other proprietary precast systems that are no longer in use include:

Gypsteel Floor and Ceiling Slabs (Figure 5): The floor slabs of this system consisted of 24-inch-wide, 2½-inch-thick molded precast gypsum, reinforced with cold drawn wires that projected from the coped or rabbeted bearing ends of the panels. The wires were twisted together with the adjacent panel end and the slot was then filled with grout for a smooth top finish.

These slabs were manufactured to span both 24 inches and 30 inches between steel support framing members. Ceiling slabs were 24-inch-wide, 2-inch-think molded precast gypsum, reinforced with flat steel bars that projected from the ends of the panels to act in conjunction with hangers suspended from the top flange of the supporting steel framing. The Gypsteel system was manufactured in New Jersey and used extensively in New York City.
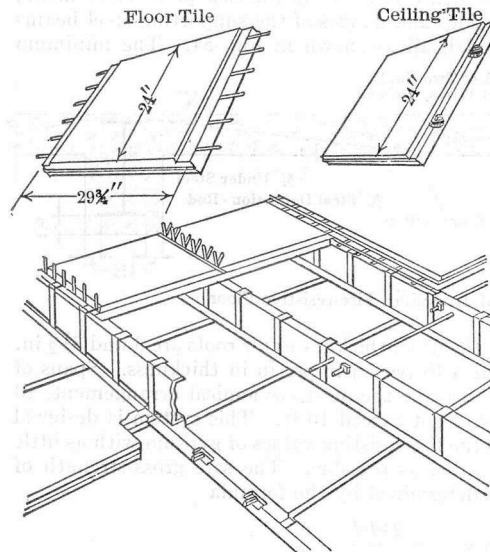
Figure 5
Source: Kidder-Parker

Waite's Concrete I Beams (Figure 6): This system was used in a number of buildings constructed by The Standard Concrete Steel Company of New York City. The floor framing system consisted of precast concrete I-beams spaced at approximately 18 inches on center of either 10-inch or 12-inch depth, which were supported from the bottom flange of steel beams that were spaced 5 to 7 feet apart. A field-cast concrete topping was then placed on top of the I-beams, while the spaces between the lower flanges were infilled, as well, to provide a flat ceiling surface.
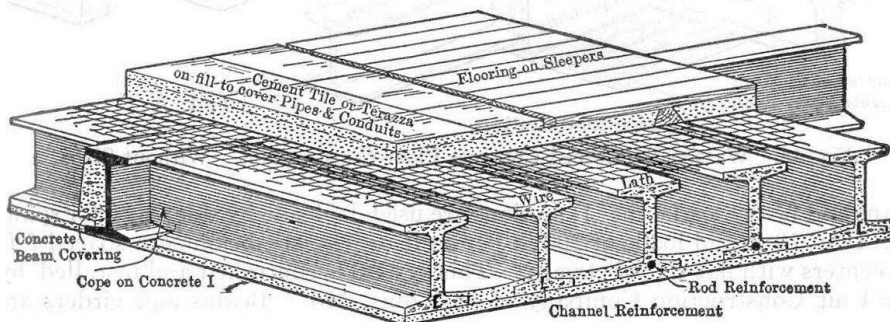
Figure 6
Source: Kidder-Parker

Watson Reinforced Concrete Floor System (Figures 7A and 7B): This type of precast construction was installed by the Unit Construction Company of St. Louis and included two types of framing. The first configuration was intended for long spans and heavy loads and involved the use of precast T-sections placed side by side. The T-sections were supported by steel beams that were encased in concrete. For shorter spans (less than 20 feet) and loads of 200 pounds per square foot or less, precast beams spaced at 5 feet on center were used to support precast channel slabs. The precast beams were in turn supported by steel beams encased in concrete. (**Source of Images:** Kidder-Parker)
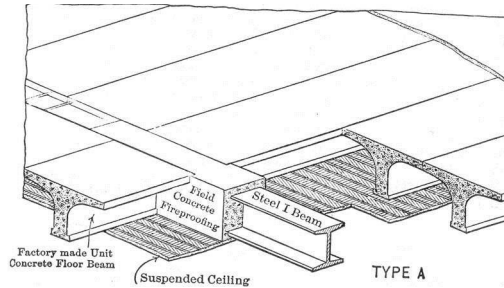
Figure 7A
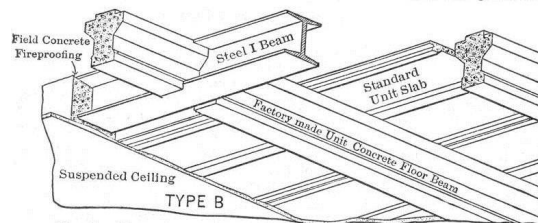
Figure 7B

Miller Precast System (Figure 8): This system was devised by the Precast Floors Corporation of New York in 1929. The precast units were shipped in three separate segments, which were aligned and supported on temporary shoring at the job site. Projecting reinforcement at the interior ends of the segments was embedded in a dry mix concrete, which was used to fill in the 9- to 10½-inch-long gaps between the segments. The center segment was produced in a standard fixed length, while the end segments were produced in varying lengths to allow for adjustment to accommodate different span lengths. Negative moment reinforcement was then embedded in a field-cast topping for continuity across the supporting steel beams. The voided, 12-inch-wide, box-shaped units were produced in 6-, 8- and 10-inch depths.
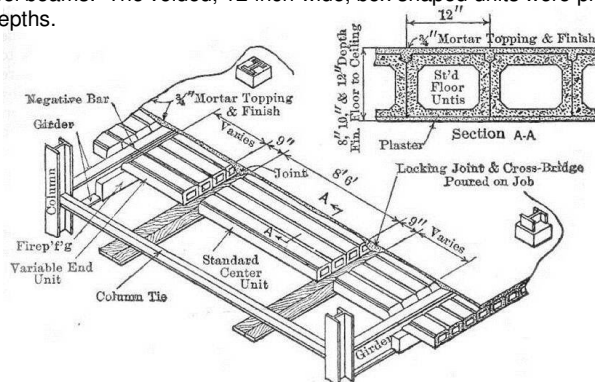
Figure 8
Source: Kidder-Parker

©D. Matthew Stuart

Lith-I-Bar System (Figure 9): This system was developed in Michigan and involved a dry-mix, lightweight concrete that was placed in an I-shaped cross-section mold and compacted with cast-iron rollers. The units were typically spaced at 18 or 24 inches on center with a 2-inch field topping cast on either removable or stay-in-place metal lath formwork for composite action with the members.
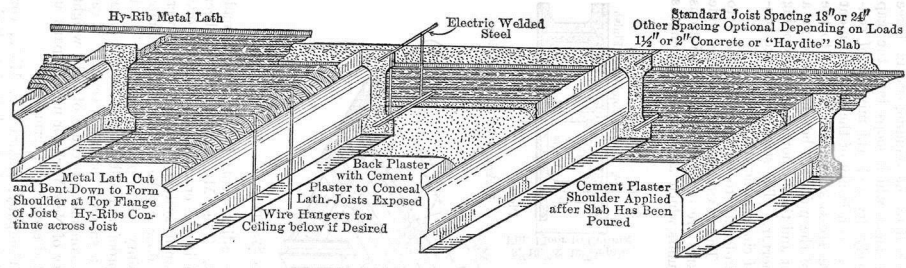
Figure 9
Source: Kidder-Parker

Porete Floor System (Figure 10): This system was manufactured in New Jersey and consisted of precast hollow formed units of 4 to 6 feet in length. This system was similar to the Miller system in that units were aligned and supported on temporary shoring at the job site. The gap between the abutting aligned units and the continuous pocket along the sides of each unit, in which field-positioned bottom reinforcement was first placed, were then filled in with a mortar/grout. All of the units were closed at each end to prevent the mortar/grout from flowing into the hollow voids of the precast member. These units were typically supported by steel beams over which top reinforcement was positioned in the continuous member pockets to provide continuity of the floor slab system. This precast system was capable of spans from 10 to 25 feet.
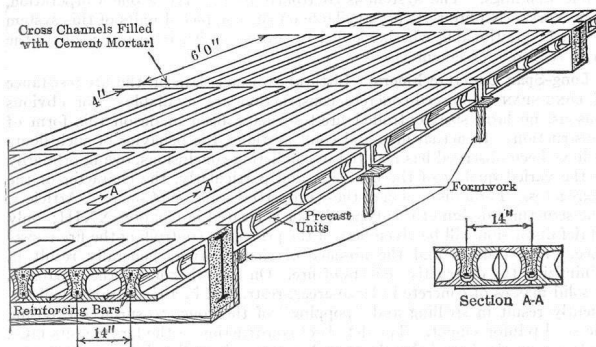
Figure 10
Source: Kidder-Parker

Tee Stone System (Figure 11): This precast member could be used as a floor beam, roof beam or wall panel and was originally manufactured in New York. The T-section was 8 inches deep, with a 16-inch-wide flange and a 1-inch-wide stem, and was manufactured in standard lengths of 8 and 16 feet. For floor construction, the T could be installed in either a flange up or flange down position. The units were placed in the field with a 1-inch gap between the edges of the flanges, which was filled with grout. The flange mesh reinforcement extended into these continuous gaps to produce a monolithic slab.
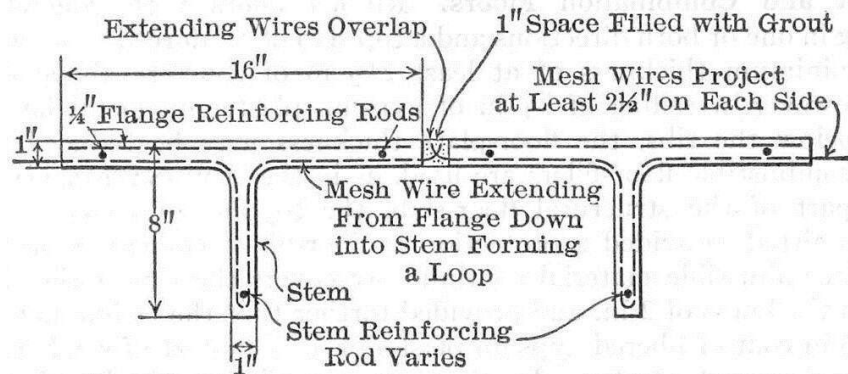
Figure 11
Source: Kidder-Parker

Pyrobar Precast Roof System (Figure 12): This cast gypsum system was manufactured for use as a roof slab and was available in both 3-inch-deep solid and 4-inch-deep hollow-core sections for short-span applications, as well as 5- and 6-inch hollow-core sections for long-span applications. The short-span sections were made in 12-inch widths and 30-inch lengths. The long-span sections were made in 18-inch widths and lengths from 4 feet 0 inches to 6 feet 6 inches. The short-span members were typically supported by steel bulb tees, while the long-span members were supported by underslung steel wide flange and channel beams.
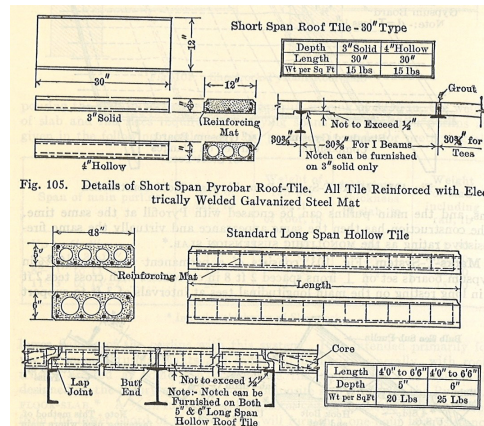

Figure 12
Source: Kidder-Parker

All of the above precast systems were designed based on the basic reinforced concrete beam analysis theories of their era. Load tables were also commonly developed and published by most of the manufacturers.
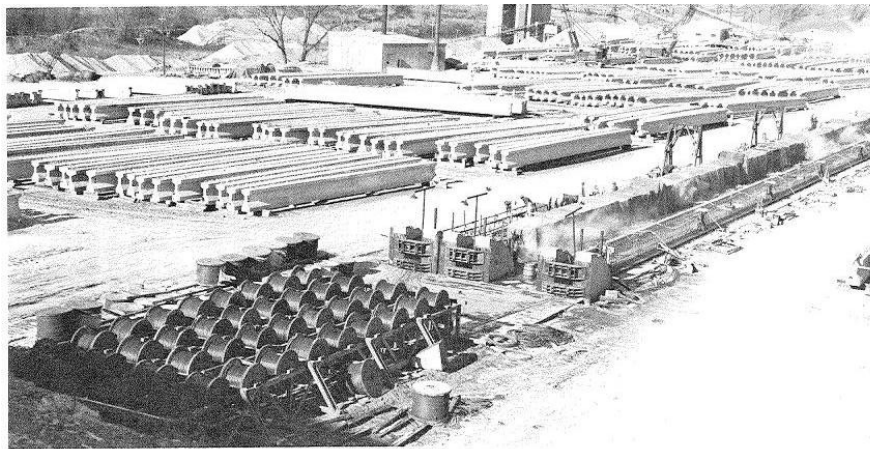
The problem with all of the above systems, when one encounters them in a building, is that in the absence of existing drawings, it is difficult to determine the internal reinforcement and, consequently, the load-carrying capacity of the system.

Source of Image:
Modern Prestressed Concrete

Early Precast Sections

Early Precast Plant

Source: Modern Prestressed Concrete

Early Prestressing Strand Stress-Strain Curve
Source: Modern Prestressed Concrete

Prestressed Bridge Girder
Source: Modern Prestressed Concrete

Supreme Chuck (anchor grip) in place on a seven-wire strand.

BARREL

JAWS

SPRING

CAP

Exploded view of Supreme Chuck

Prestressing Strand Chuck
Source: Modern Prestressed Concrete

Preformed Line Products Co. splice for seven-wire strand showing strands ready for splicing and the splice subsets to be used. The splice subsets are made of high-strength wires which have their contact surfaces with the strand coated with a gripping compound.

Preformed Line Products Co. splice partially assembled on seven-wire strands.

Wire Strand Splices
Source: Modern Prestressed Concrete