DEVELOPMENT AND PERFORMANCE OF A PRECASTING SYSTEM FOR CONCRETE HYPERBOLIC PARABOLOID SHELL CONSTRUCTION

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CHAPTER I

INTRODUCTION

The uses of reinforced concrete have been greatly expanded as a competitive structural material in the building industry during the last decade. American architects and design engineers have recently begun to utilize reinforced concrete in designing such configurations as the folded plate, dome, barrel shell, and the hyperbolic paraboloid, henceforth discussed as the h-p. Because of its inherent qualities of variable strength, plasticity, "built-in" fire protection and low maintenance costs, this "versatile product" has become popular with designers as a structural material.

Until the late 1950's the principal precast concrete construction consisted of the use of concrete blocks, which limited construction mostly to modular units. Some experimental work in tilt-up construction was also conducted. However, during the past five years the use of concrete shells utilizing the four basic geometric shapes previously listed, has broadened the outlook of the concrete industry considerably. A major advantage which the use of concrete shell roofs has introduced is the large savings in material and reinforcement while covering large floor areas. Due to the basic shapes of the shell structures, large vertical loads are readily transmitted into axial stresses in the plane of the roof where they are transferred into edge members. Thus, the tensile and compressive properties of reinforced concrete are utilized effectively.

If these advantages of reinforced concrete shell construction can be further enhanced by the development of suitable methods to precast and erect these structures, costly formwork and large crews of skilled labor can be eliminated. Only then can reinforced concrete compete favorably with other producers of pre-packaged buildings on the consumer market.

The Problem

Trends in light building construction in the United States during the last decade indicate the need for a practical, economical, one story structure for use as a farm or light industrial building. Several types of buildings of various building materials have been developed and are now available on the market; however, at the present time few concrete structures are available that are ready for assembly on a selected site.

A limited amount of research has been done on precasting light concrete structures. Most of the work done previously has been on prefabricated steel or wooden frame structures. Some work has been done on the design and insitu construction of the basic thin shell concrete structures using the principles of the folded plate, dome, barrel shell, and the h-p configurations. To compete favorably with steel and wooden prefabricated structures in satisfying the demands of the light construction industry, the concrete producer needs structures which can be precast and assembled rapidly, have versatility, and have close tolerances.

The general requirements of a structure which will satisfy the demands of agricultural or light industrial use are defined as follows:

It should (1) be structurally sound, (2) be attractive (3) be erected by small contractors, (4) provide good fire protection, (5) provide low maintenance costs and depreciate over a long time period, and (6) be functional.

The major requirements which must be overcome in the design of a concrete shell precast structure are: (1) suitable joints and connections, (2) a simple system of precasting, (3) capability to resist moving or transportation stresses in the precast elements intransit to the construction site, (4) efficient use of formwork, (5) adequate lifting and placing equipment, (6) a temporary support system and sequence for erecting and assembling the precast elements, (7) adequate lifting devices and attachments on all precast elements, and (8) an adequate footing system to support the building during adverse atmospheric and soil conditions.

Objectives

The objectives of this study are:

1. To design a precast shell which can be incorporated with a precast column system to form a h-p shell structure.
2. To develop a readily transportable support framework which will rigidly stabilize the structure during erection and assembly.
3. To develop a step-by-step procedure for the assembly of a precast h-p shell.
4. To load test the structure to verify the structural design.
5. To evaluate construction costs of the system.

Limitations

Of the four basic shell structures which are most frequently used today, the h-p, Figure 1, has proven to be one of the most efficient and easily analyzed. Because of the financial limitations of this project, the study was limited to one structure, a four quadrant h-p with two supporting columns, Figure 2. This structure seems to compare favorably with the other three basic h-p structures, Figure 1, as far as the rural customer is concerned, because it is suited to a number of agricultural uses.

The prototype used for this research was limited to a 20 by 20 foot shell as this size was adequate for testing the procedures developed in this study.

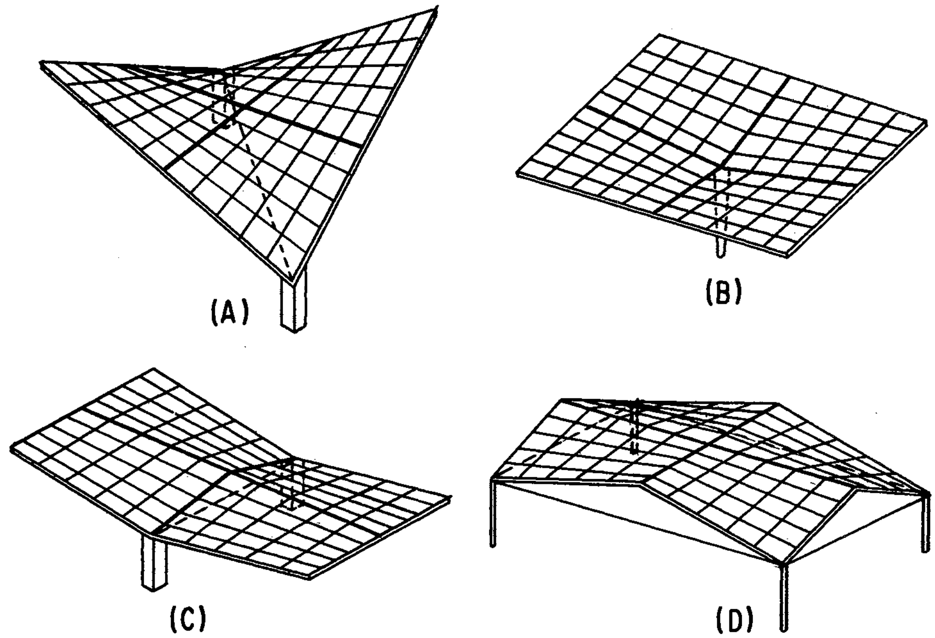


Figure 1. Four Basic Hyperbolic Paraboloid Shell Configurations.

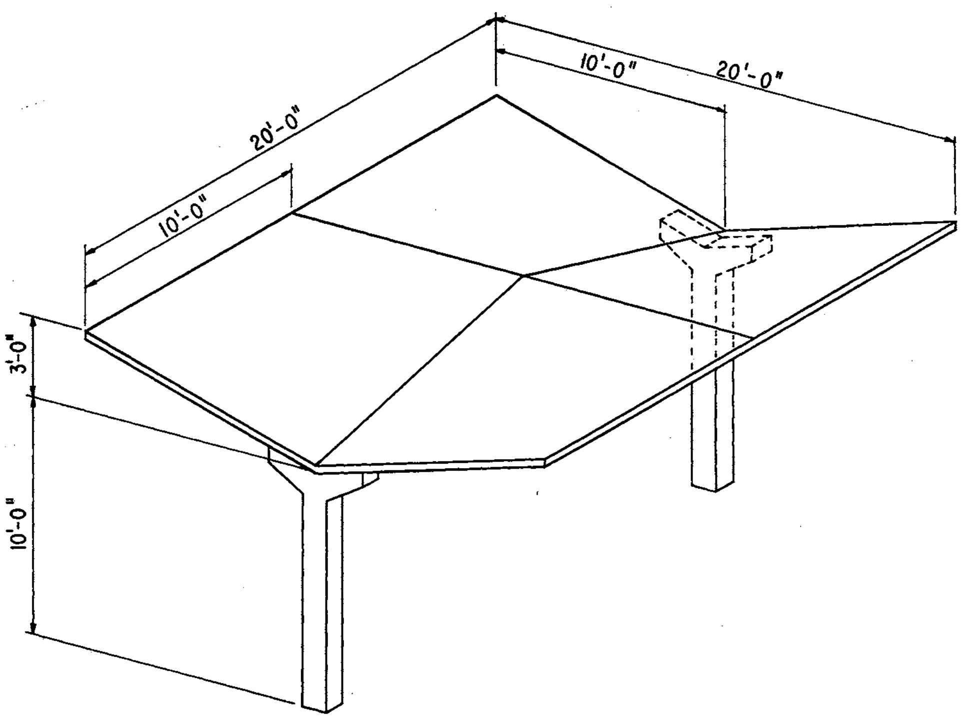


Figure 2. The Hyperbolic Paraboloid Shell Configuration Used in This Study.

CHAPTER II

LITERATURE REVIEW

Introduction

The precasting of reinforced concrete shells is a relatively young field in modern construction. This method of erecting concrete structures lacks only the development of assembly line procedures in factory or job site casting beds and standardized erection methods on the site before it can be a competitor with other prefabricated structures. Precast concrete elements are becoming increasingly popular in the building industry due to the savings which are realized by simplified or eliminated formwork and by the multiple use of forms. Less skilled labor is required during erection, quality control is greatly improved, and structural elements may be factory cast year 'round, which reduces lost time on the job for curing the concrete. Curing processes can be more closely controlled by steam or hot water curing in plant precasting. Precast units can be stockpiled in erection sequence, thus eliminating excessive handling and storage on the worksite.

Precasting

Beauchemin (1) discussed the advantages and disadvantages of precast concrete. He concluded that the advantages outweigh the disadvantages by a large margin. Listed below are the specific advantages and disadvantages which are usually encountered in precasting with a brief discussion of each.

1. The principle disadvantages of precasting are:

    a. Shrinkage

    To completely hydrate, one bag of cement requires approximately 2 U.S. gallons of water. Extra water added to give plasticity to the mix, not only weakens the paste but also causes the concrete to shrink when portions of it dry out. Added steel reinforcement will aid in resisting stresses due to shrinkage.

    b. Weight

    The average concrete mix weighs in the range of 140 to 150 pounds per cubic foot. This means that in precast concrete work, handling becomes more difficult and, therefore, more expensive. Hauling and shipping costs are higher, placing requires special lifting and handling equipment; and the over-all size of the precast unit is limited.

These weight difficulties can be reduced by use of (1) hollow cores, (2) high strength concrete, (3) lightweight aggregates, (4) prestressing, or (5) various combinations of these items.

c. Assembly and Continuity

Practically every precast concrete product must eventually be connected to another concrete product or to some other construction material. Large pieces, such as columns, roof slabs, wall panels, and beams, present assembly and joint difficulties. Reinforcing bars and dowels which protrude from precast elements are sometimes welded or bolted together or to other members at construction joints to develop continuity. Post-stressing is also used to join several components into a composite unit.

2. The principle advantages of precasting are:

a. Economy

Precasting economy is incurred by labor and forming cost reductions.

b. Quality

Quality is closely controlled by the use of right mixes and by maintaining optimum humidity and temperature during the curing process. These control measures can best be achieved in a concrete products plant. Concrete placement is facilitated because of low level formwork and mechanical placement aids, such as vibrators.

c. Speed of Construction

The erection of precast concrete takes far less time than in-situ concrete, because the former requires little or no formwork or curing period, unless grouting is required; even then curing time is reduced.

d. Flexibility

When concrete is poured, it is a plastic material which can be molded into any shape or form desired. This quality, peculiar to concrete, is one reason why concrete has always interested designers. Although some shapes are difficult to achieve with in-situ concrete, precasting supplies these shapes with greater ease and economy.

e. Availability

Wherever the construction may take place, the designer can usually locate a good source of precast concrete products within a reasonable distance.

Several precast structures built within the past four years indicate the potential of precast construction.

Amirikian (2) designed and erected a multipurpose building of precast thin shell panels. This structure was a frame type building with bolted joints and connectors using a modular system of panel assembly. For this frame design, Amirikian used the statically determinate three hinged bent principle. He stated:

> As yet no standard procedure for the assembly and erection of a panel building has been devised. This is something which must be developed by builders and fabrication geared for mass production and erection of these structures.

In 1957, the gymnasium roof of the Westmore High School of Daly City, California, was constructed of precast reinforced concrete barrel arches supported by precast bents. The arch shaped units, each 61 feet long, 15 feet wide, $3\frac{1}{2}$ inches thick, with a $3\frac{1}{2}$ feet rise, weighed 20 tons. The bents were three-hinged and spanned 91 feet. The barrel arches were made of 3,000 psi lightweight aggregate concrete which weighed 102 pcf. The contractor set up a casting line using six sets of forms and cast six shells in less than four hours. No camber was introduced for the 52 foot span as the maximum deflection at midspan was only one-half inch. The erection of the 20 ton units was handled by two 50 ton capacity cranes.

Early in September, 1960, Hurricane Donna cut across the island of Puerto Rico leaving hundreds of people homeless. Within hours after the tragedy, IBEC Housing Corporation proposed a program of commercial housing to the Puerto Rican Government at a cost of 1,000 dollars per living unit. IBEC and the contributing suppliers delivered a house shell, approximately 630 square feet in area on a foundation slab measuring 21 by 31 feet with front and back doors and eight jalousie-type windows installed. The entire package included 12 elements; six flat wall sections, two curved wall sections, two roof beams and two roof sections. Components were precast in IBEC's casting yard near San Juan and trucked 45 miles to the site. A test house was erected in one hour using a seven man crew.

Faerber (3) describes the construction of a precast folded plate roof for a residence in Naples, Florida. The building was designed in the shape of an octagon, incorporating eight separate gables, each designed as a folded plate. The roof slabs were cast one on top of the other as in the lift slab method of construction. Sheets of polyethylene film were used to facilitate separation. Each of the 400 square foot sections were four inches thick and weighed 11 tons.

A slightly different approach toward precast concrete was conducted by Riley (4). He constructed a barrel shaped roof by shaping the earth into the desired form and precasting his roof in place. The columns were placed prior to casting the roof in order to allow sleeves cast in the roof to utilize the columns for stability during the erection sequence. The roof was cured on the casting bed, then raised by means of hydraulic jacks. After the roof was raised, the mound was leveled off and the floor slab was cast in the conventional manner. Riley used a polyethylene film between the earth and the concrete on the casting bed which gave a smooth undersurface to the roof, making finishing the underside of the roof unnecessary. This particular method of construction has limited uses but the method of forming the casting bed from an earth mound has good future possibilities. This type of form could be re-used many times by jacking or lifting the forms off the casting bed or by the lift slab principle where several slabs can be cast one on top of the other. The second method could be used to store a limited number of slabs in place for use on a specific project.

One of the largest precast concrete construction projects yet undertaken is described by Thompson (5). This project consisted of roofing the new terminal building and ticket building of the Oakland International Airport in California. Two types of precast shells were used. The roof of the terminal building consisted of inverted umbrella shaped h-p surfaces; each precast element had a minimum shell thickness of $2\frac{1}{4}$ inches and weighed 16 tons. The conoidal shaped barrel vault was used to span the ticket building. Both roofs used a mix design of 2,750 psi. The h-p shells were designed to support a full load of water in case of a plugged drain.

Many more examples of precast shell construction similar to those mentioned above are in evidence at the present time. Extensive preparations and plans have been made by several organizations to promote precast concrete design and construction. One of the major promoters of precasting, the Portland Cement Association, publishes several types of literature promoting the use of precast shell structures utilizing new space frame techniques. The American Concrete Institute has placed additional emphasis on the need for uniform practices in precast concrete design and construction by additions and major revisions of the ACI Building Code.

Although there are various reasons for the design and construction of each individual shell structure, the predominate motive is economy. Because of the substantial savings of time, material, and manpower which can be obtained by precasting, the demand for precast reinforced concrete elements in building construction will continue to grow.

Hyperbolic Paraboloid Shell Structures

Felix Candela (6), internationally recognized in the architectural world for his extensive work with thin shell h-p surfaces, stated:

> Hyperbolic paraboloidal surfaces are extremely interesting from a structural and constructive point of view. Their use in reinforced concrete shell roofs offers the same advantages inherent to all shells of this material, i.e., lightness, incombustibility, economy of material, security against explosions, bombardments and earthquakes, and little sensitiveness to foundation settlements. These last properties are consequences of their structural action; not restricted to one plane, but working as space-frames.

The theory of the h-p is an old one but only in the past decade have the basic principles of membrane stresses been put to work as economical space enclosures. Although they are doubly curved surfaces, the surfaces are formed by two systems of straight generatrices. This fact greatly simplifies the basic formwork for casting the shell by allowing the formwork to be composed only of straight lumber, provided the shell is rectangular in the horizontal plane. The principle stresses which exist in the h-p shell surface are tensile and compressive stresses which form angles of 45 degrees with the direction of the generatrix. These stresses accumulate and are transferred by shear from the shell edge into the edge beams which are parallel to the generatrices. The shears accumulate along each side of the warped parallelogram, resulting in either tangential tensile or compressive forces which are redistributed in the shell or act as compressive thrusts at the column. By taking one h-p quadrant and combining it with three other quadrants of the same dimensions, a variety of structural shapes can be obtained.

Candella (7), in his discussion of warped surfaces at the Conference on Thin Concrete Shells in 1954 at the Massachusetts Institute of Technology, pointed out the fact that the h-p stress analysis does not involve higher mathematics, and is even elementary when surfaces with small slopes are considered. He stated that "on account of their double curvature, it suffices to investigate the membrane state of stresses, without considering bending or deformation."

Candella was one of the first designers in the western hemisphere to experiment with h-p's. His first h-p shell was the Cosmic Rays Pavilion at the University of Mexico. This structure has received much acclaim due to its extremely thin surface. Because of a functional requirement that the top part of the shell have no more mass than eight pounds per square foot, the shell thickness was only five-eighths inch in the upper part of the structure. The success of this structure prompted him to design other structures as h-p shells; one of the most notable was Rio's Warehouse in Mexico City. The basic program requirements were to economically cover 55,000 square feet of floor space and at the same time to provide a small amount of roof light, a clearance height of 15 feet, and 50 foot bays. The solution was found in a reinforced-concrete structure containing 36 umbrellas which were approximately 30 feet by 50 feet. Standard weight 2,000 psi concrete was used throughout and was vibrated evenly in the thin shell roof. As a result, the good compaction eliminated the need for waterproofing the roof shells. By tilting each umbrella slightly, Candella obtained a north light effect in a very economical manner. In a warehouse built more recently from h-p shells, he used glass blocks cast in the roof slabs for additional top lighting. To solve the problem of footings in one of the world's worst subsoils (150 feet of clay which varies from 75 to 90 per cent water content by total weight), Candella designed an umbrella shaped footing which he cast over a shaped earth mold.

Parme (8) presents a good mathematical analysis of the h-p shell theory. His discussion shows that there are no forces normal to the edges of an h-p shell subject to a uniform load. Parme stated that:

> For most hyperbolic paraboloid shells of moderate rise, it is deemed satisfactory to consider the load as uniform. However, when the rise is great, the dead load can no longer be considered as uniform on the projected area.

One of the largest single h-p shell units in the United States is the entrance to a new department store which is part of the Denver Court House Square Development. The shell, designed by Tedesko (9) was opened to the public in August, 1958 as an exhibit pavilion. The roof consists of four h-p surfaces and is supported through steel hinges on buttresses at the four corners of a rectangle. The three inch shell which rises to a height of 28 feet spanned a floor area of 14,800 square feet.

An example of the economical large scale production using h-p roofs is illustrated by Madsen and Biggs (10). An h-p shell roof was designed for a shopping center in Minneapolis, Minnesota. The structure consisted of 44 h-p shells, each 46 feet 4 inches by 48 feet 6 inches, covering a floor area of 100,000 square feet. The structural design follows the classic formula developed by Felix Candela.

This construction program was designed around the reuse of movable forms. They scheduled all construction operations on an assembly line basis. The steel was pre-tied where possible so that placing could be accomplished in a minimum time. The concrete placing was scheduled so that eight h-p's could be cast per week, allowing 44 hours curing time before forms were stripped. Curing procedures were started within five to ten minutes of final smoothing. A check on deflections after the 28 day curing period indicated that the corners of the h-p's had deflected three-fourths inch, while at the midpoint of the edge, the deflection was also three-fourths inch.

Many applications of h-p shells are being used, ranging from airport structures, hospitals, libraries, and industrial buildings, to modern farm structures. In addition to its application as a roof surface, the h-p has been well adapted to use as a foundation structure for soils of low bearing pressures. New ideas are continuously being developed to utilize its full potential as a structural shape. Many problems continue to exist in h-p construction leaving opportunities for future development.

Major Problem Areas in H-P Shell Development

Although many problems exist in designing and constructing h-p shell structures, some are more predominant than others. Some of the major problem areas are listed below. Each problem will be discussed specifically and analyzed in terms of this project.

(1) Footings - Because of the various shapes and sizes of h-p shells and the conditions under which they are erected, footing problems will vary from one location to another. Footing systems are usually broken down into two groups, the in-situ footing and the precast footing. Each method of placement is dependent upon the local soil conditions and, therefore, must be designed under the same criteria. Footings of both types can be standardized to some extent for a specific structure but must be checked for each individual building site to determine the design adequacy.

(2) Lifting Equipment - Building sizes, materials to be handled, and location of construction sites determine the types and sizes of lifting equipment required for specific construction projects. For in-situ construction, the lifting requirements are usually limited to fairly small loads such as steel members and concrete buckets. However, in precast construction, the sizes and weights of precast elements may be quite large and are the major factors which determine the crane sizes. Peurifoy (11) gives a good analysis of the safe lifting capacity and radius of operation of several sizes of cranes which could

be used to lift bulky precast elements.

(3) Formwork - The costs of concrete formwork may be excessive if multiple reuse of forms is not made possible. Minor structural failures in forming systems are relatively common. A study of the cause of structural forming failures was made by ACI Committee 622; the most common deficiencies leading to form failures in building construction are listed in their report (12).

(4) Curing - Proper curing of concrete elements is one of the most difficult operations in construction. Optimum curing is usually desired on construction projects to obtain maximum concrete strengths, but is usually difficult to attain. The variables which control curing are (1) temperature, (2) moisture content, (3) time, and (4) freedom from physical disturbances. Providing the temperature is acceptable, only the moisture content need be controlled if the mass is free of physical disturbances during the curing period. ACI Committee 612 (13) makes the following recommendations for optimum curing:

(a) Horizontal Units

1. Initial curing - As soon as finishing operations are completed, cover with two thicknesses of an approved woven fabric or quilted fiber mat which is

saturated when placed. Cover is kept saturated with water and is kept in place until the heat of hydration has been dissipated.

2. Final curing - (a) Same cover left in place throughout the curing period, (b) two inches of moist earth or sand continually saturated, (c) three inches of wet hay, grass, or clean straw uniformly distributed and saturated continuously, (d) approved impervious light colored paper or plastic covering placed in constant contact with the concrete surface, or (e) approved impervious compound or coating sprayed on the surface in liquid form.

   Coatings should be light in color when concrete is exposed to the direct sunlight. When the temperature is above $40^{\circ}F$, the final curing agent should remain in place at least 72 hours or more as strength requires. When the air temperature is less than $40^{\circ}F$, concrete should be so protected to maintain 50 to $70^{\circ}F$.

(b) Precast Units

1. Initial curing - Immediately after the

casting operations, enclose each member by two layers of an approved water-saturated fabric until placed in position for final curing.

2. Final curing - Members may be cured under the original saturated fabric, or moved to a special chamber where they may be uncovered in a completely saturated atmosphere of mist, water, or steam. The temperature for a curing room should be uniformly maintained between 50 to 180°F. Final curing may be performed under a pressure between 100 psi and 150 psi in saturated steam at 335 to 366°F.

In many cases, precasting on the job site will not permit use of factory controlled final curing procedures. In this case, the final curing procedures listed under Horizontal Units should be applied.

(5) Joints and Connections - Of the many problems encountered in h-p shell development, whether precast or in-situ construction, designing adequate joints and connections is one of the most difficult problems to overcome. Cazaly (14) points out the fact that joints must: (a) withstand bending moments without breaking down, (b)

absorb concentrations of stress and strain, and (c) occupy minimum space and present a neat appearance when exposed. For economic reasons, they must be: (d) safely formed by normal labor, (e) cheap to fabricate without expensive or excessive formwork, (f) capable of erection in all kinds of weather, (g) fast to erect without cranes and other trades, and (h) able to take a considerable amount of tolerance. Mr. K. C. Naslund (15) summed the problem of joints and connections neatly when he stated that:

> The engineer must determine his scheme of erection, then design his members and joints for the stresses that occur during fabrication, delivery, and erection, as well as with final conditions. He must visualize how the members will be erected to assure that the erection is safe, feasible, and that it is economical.

(6) Safety - Safety is a continuous problem in practically all types of construction, yet it is too often overlooked on the job as well as in the design. Design safety factors and features should be one of the first considerations given to a structural design.

(7) Waterproofing - Waterproofing of shell surfaces can usually be accomplished by three basic methods: (a) use of a built-up roofing surface such as a bituminous coating, (b) use of a sealing compound such as a neoprene roofing material or

a light colored polyester-based paint to seal the pores in the concrete, and (c) by designing the concrete to obtain a dense, impermeable mass. Other methods of waterproofing are available, but are less frequently used. Of the three methods mentioned, the newest method is the use of a neoprene roofing compound which can be placed on the surface with paint rollers.

For example, one commercial product, Armstrong F/A Roofing is applied in three basic steps: (a) joints are sealed with a deck sealer and flashing tape is applied as a reinforcing membrane where needed, (b) two layers of the F/A 400 Base Course are applied in two colors to aid in visual gaging of the film thickness during the second application, and (c) two applications of F/A 600 complete the installation and provide a final waterproof coating and a variety of roofing colors for modern structures. The favorable characteristics of the neoprene compounds are its flexibility for expansion or contraction of roofs, versatility for conforming to any roof surface or slope, and ease of repair or maintenance.

Results and Conclusions of Testing

1. H-P Models and Prototypes

Several tests have been conducted on h-p shell structures to determine the capacity of shells under a variety of tests. One recent test was conducted by the Structural Development Section of the Research and Development Division of Portland Cement Association (16). The shell used for testing was an inverted umbrella with a 24 by 24 feet outside dimension, a 1 1/2 inch shell thickness and a 2 feet 10 inch rise. The reinforcing in the shells consisted of No. 3 bars at 12 inch centers.

The loads which were applied were: (a) a uniform vertical load of 50 psf, (b) four equal concentrated loads applied symmetrically on the shell, one at the center of each quadrant, using a 2 by 3 inch washer as a contact area, and (c) an unsymmetrical loading of 75 psf was applied to two adjacent quadrants. During the uniform load, the sum of dead and live load produced a calculated thrust in the perimeter beam reinforcing of 26,300 psi. No excessive stress was noted under this load. The concentrated loads produced some minor radial and circumferential cracking at the load points when the loads reached 5,000 pounds. This load also produced a local bending moment of 1.3 kip-ft/ft at the point of application of load with a punching shear of 500 psi. No distress was observed during the unsymmetrical load over the major portion of the shell even near ultimate capacity.

The tests on this shell demonstrated that h-p shells with a thickness of only 1 1/2 inches can resist large concentrated loads as well as unsymmetrical loads.

A similar umbrella shell was constructed and tested at Oklahoma State University in 1962 (17). The size of this shell was 20 by 20 feet with a minimum shell thickness of 2 inches. Testing of this shell was accomplished by closing the drain and filling the shell with water. Deflection readings were taken at each corner after each load increment was added. A total load of approximately 14,000 pounds, or 35 psf horizontal loading was applied. The maximum deflection noted under the total design load was 0.004 feet. No other effects of strain were noted.

Harrenstien (18) discusses tests conducted on two reinforced concrete h-p shell prototypes which were constructed as a class project at Iowa State University. The shells were 10 feet square in plan, 1 inch thick, and had a maximum rise of 1 foot 8 inches in 5 feet. The average 28 day compressive strength of the concrete was 7,500 psi, and the average modulus of elasticity was 4.75×10^6 psi. The inverted umbrellas were mounted inversely on an 8 inch steel column and were loaded simultaneously by a single hydraulic cylinder jack as a concentrated load on each shell. Point loads were individually applied at 60 locations on each shell, with a maximum applied load of 548 pounds at each point. A system of strain gages located the principle stress contours for each individual load. The test results were used to set up a prediction equation to determine the final stresses in the shell due to applied loads.

Waling and Greszczuk (19) conducted experiments on thin-shell h-p models at Purdue University, using styrofoam stretched on high strength wires as a forming material. Results of their studies indicated that styrofoam would make a good forming material for field construction.

Although other testing has been done on reinforced concrete h-p shells, these examples indicate the types of experimental work that have been conducted.

2. Lightweight Aggregate Concrete

Lightweight aggregate concretes are now generally accepted for conventional construction. They are especially useful when they will produce a structural strength equal to that produced by normal weight concrete, and at a lower cost. A savings can be obtained due to lower total weights of structures, which require smaller or lower strength support members. Lightweight concrete also has up to 5.5 times the insulation quality of standard weight concretes.

To determine their structural qualities, a considerable amount of testing has been done on lightweight aggregate concretes. Several of these tests have been conducted to compare lightweight aggregate with conventional aggregate concretes.

Hanson and Kleiger (20) made extensive tests of the freezing and thawing performances of lightweight aggregate concrete as compared to normal weight concrete. They tested nine lightweight aggregates and one sand and gravel aggregate. Each aggregate was used in both an air-dried and a saturated condition. The ten samples were each designed at two different strength levels, 3,000 psi and 4,500 psi. Various percentages of air entrainment were introduced in the test samples. The conclusions derived from the freezing and thawing tests were:

(a) Entrained air increases resistance by freezing and thawing.

(b) The amount of entrained air required for both conventional and lightweight concrete is approximately the same.

(c) The initial moisture condition of the lightweight aggregate has a significant influence on the resistance to freezing and thawing compared to only minor influences for the standard concrete.

(d) The variation in durability among the concretes made with the different lightweight aggregates appears no greater than might be encountered with normal weight aggregates.

(e) Aggregate properties are obviously of importance in determining the level of durability, even in air-entrained concretes.

Hanson (21) tested seven commercially available lightweight aggregates in reinforced concrete beams. These tests were part of an investigation by the Portland Cement

Association Laboratories to augment available technical information necessary for the design of structural concrete using lightweight aggregates. In these tests, the beams were loaded at third points. The results indicated that at comparable strengths the sand and gravel concretes showed a nominal shear strength no greater than that of the lightweight aggregate concretes. It was evident throughout the tests that the lightweight beams failed more suddenly than the sand and gravel concretes, especially at higher concrete strengths.

Shideler (22) conducted tests on eight lightweight concrete samples and one standard sample. Results from his testing indicated that:

(1) Structural grade concrete was obtained with each of the lightweight aggregates.

(2) The unit weights of the various lightweight aggregate concretes in the lower strength series (3,000 to 4,500 psi) ranged from 90 to 110 psf compared to 146 psf for standard concrete.

(3) The various lightweight aggregates require a wide range of cement content to produce similar strengths.

(4) The modulus of elasticity of the lightweight aggregate concretes in the 3,000 to 4,500 psi series varied from 53 to 82 per cent of the modulus of sand and gravel concrete at 28 days.

(5) Flexural strengths of the lightweight and sand-and-gravel concrete were approximately equal at early ages, but after 28 days the standard concrete showed greater strength gain with continuous moist curing.

(6) Bond strengths of some of the lightweight concretes were approximately equal to those of sand and gravel concretes.

Shideler (22) concluded that within the group of lightweight aggregates studied, rather wide variations were obtained in the structural properties of the concretes. He felt that it was important that the individual producers of lightweight aggregates for structural concrete conduct investigations to provide reliable design data on the performance of their product.

In recent tests, Hanson (23) has determined tensile strength and diagonal tension resistances of structural lightweight concrete. Comparisons of the unit shear strengths at diagonal cracking with the ACI Building Code working stresses reveal that inadequate factors of safety existed for the lightweight concrete beams with long spans and low steel percentages.

Due to the high moisture absorption characteristics of lightweight aggregates, the American Concrete Institute has published a new standard, "Recommended Practice for Proportioning Lightweight Aggregate Structural Concrete (ACI 613A-59)."

CHAPTER III

THE EXPERIMENTAL STUDY

The procedure used in this research program was to design, erect, and test a precast concrete shell. The plan of research that was followed is discussed in the following section. However, before a procedure was set up, certain problem areas were outlined on which the study was made. Problem areas which were investigated are:

(1) Use of lightweight aggregate concrete for the shell surface.

(2) Joints and shear connections between precast roof elements and edge beams.

(3) Support system to stabilize roof elements during erection.

(4) Lifting device to lift shell elements into position.

(5) Precasting system for shell.

(6) Suitable erection and assembly techniques.

(7) Foundations.

(8) Shell to column anchorage.

Due to the limitations of this program, this study was conducted on one specific configuration of the h-p shell.

This structure consisted of a four-quadrant shell with two supporting columns. This type of structure appeared to be more practical to precast than a structure with four supports when used as modular units for large roof areas such as a hay storage shed, an equipment shelter, or a dairy barn. It seems that the two-support structure would be more appealing to farmers for small structures as it appears more stable and yields a larger clear floor area than the inverted umbrella structure, when used in widths of only one modular unit.

Research Outline

This program was outlined to meet the objectives stated in Chapter I by analyzing the problems listed in Chapter II in a logical order. Although separate stages of research were carried on concurrently, the steps which were followed are:

(1) Design the shell and supporting structure for precasting.

(2) Design and construct a simple support system to stabilize precast elements during assembly and erection.

(3) Prepare a building site and layout plan for precast units and construction material and equipment.

(4) Cast shell components and take samples of material at time of casting.

(5) Test samples at various time intervals to determine strength of precast elements.

(6) Erect the columns and tie.

(7) Erect the roof and record time required for assembly.

(8) Load test the structure to obtain load deflection data.

  (a) Design live load was 40 pounds per square foot, uniformly distributed.

  (b) Test to approximately $1\frac{1}{2}$ to 2 times design live load.

  (c) Test to design load on one-half the roof surface.

  (d) Measure deflections during sustained loading.

(9) Analyze the test data.

(10) Prepare a detailed cost analysis on labor, equipment, and material requirements for precasting and erection.

(11) Assemble pertinent data and combine results of research on erection procedure, testing, and cost analysis.

CHAPTER IV

DESIGN OF STRUCTURAL ELEMENTS

Shell Design

The basic design of the prototype shell was analyzed using the basic equations listed in the Portland Cement Association publication, Elementary Analysis of Hyperbolic Paraboloid Shells (24).

Several basic decisions were made initially concerning the desired parameters of the prototype. These design factors were:

1. The over-all dimension in plan would be 20 feet by 20 feet.
2. The minimum shell thickness would be two and one-half inches.
3. The vertical rise, h, would be three feet.
4. Design static loading would be 40 psf in the horizontal plane.
5. Lightweight aggregate concrete with an estimated density of 110 pcf would be used for shell material.

Horizontal Thrust in Parabolic Arches

Because of the doubly curved surface of the h-p shell, the load, W, is supported by two arch-like elements so that each element will support one-half of the load intensity, $\frac{W}{2}$, Figure 3. The internal moment developed in this two-hinged arch is $\frac{W}{2} \cdot \frac{L^2}{8} - H\,(-h_{xy}) = 0$, or $H\,(-h_{xy}) = \frac{W}{2} \cdot \frac{L^2}{8}$. Thus, $H = -\frac{W}{2} \cdot \frac{L^2}{8\,h_{xy}} = -\frac{WL^2}{16\,h_{xy}}$, where

H = the horizontal thrust at the end of each arch per foot of shell width.

h_{xy} = vertical rise of each arch.

L = horizontal length of each arch.

Further simplification of the analysis for horizontal thrust, H, yields $H = \frac{W \cdot a \cdot b}{2h}$, Figure 4, where

W = the total unit load in pounds per square foot.

a = the length of a horizontal side of one quadrant.

b = the length of the adjacent horizontal side of the quadrant.

h = the vertical rise of the shell.

The approximate dead load per square foot = 2.5 in. x $\frac{1}{12 \text{ in/ft.}} \times \frac{110 \text{ lb.}}{\text{ft}^3} = 22.9 \text{ lb/ft}^2$ or 23.0 lb/ft^2

The design static load = 40 lb/ft^2

Total design load = 40.0 + 23.0 = 63.0 lb/ft.

$$H = \pm \frac{wab}{2h} = \pm \frac{63.0 \times 10 \times 10}{2 \times 3} = \pm \frac{6{,}300}{6} = \pm 1{,}050 \text{ lb/ft.}$$

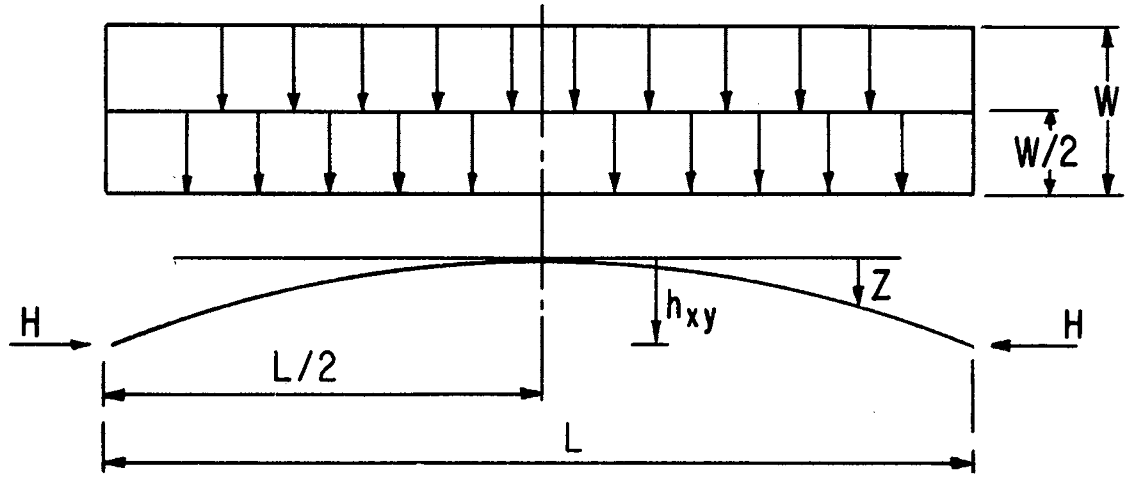


Figure 3. A Typical Parabolic Arch With Horizontal Load Distribution.

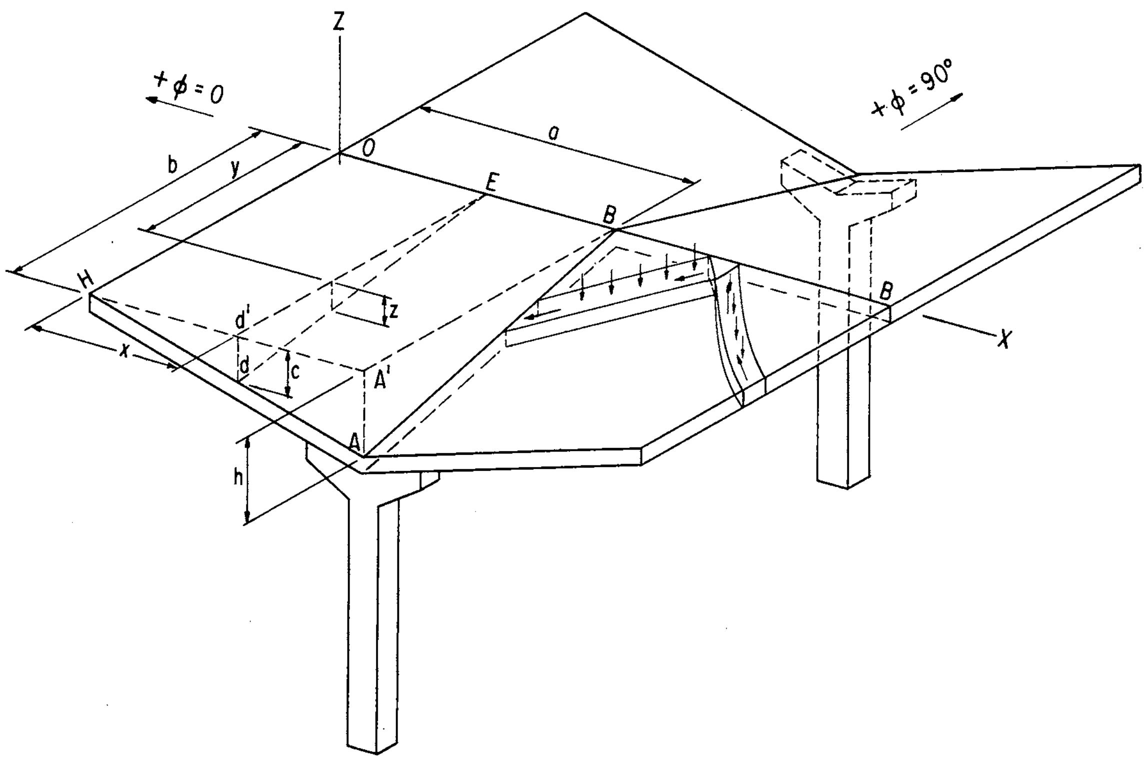


Figure 4. Quadrants Showing Dimensions and Stress Distribution in the Parabolic Arches.

Tensile Shell Reinforcement

The area of steel required per foot of shell width in the direction of the parabolic arches, Figure 4, is $A'_s = \frac{H}{f_s} = \frac{1,050 \text{ lbs.}}{20,000 \text{ psi}} = .0525$ sq. in.

Area of Steel Required Perpendicular to Edge Beams

To simplify steel placement during forming, the shell steel area perpendicular to the edge beam was computed; thus, $A_s = A'_s \times \sec 45^{\circ} = .0525 \times 1.414 = .075$ sq. in./ft. No. 2 bars at eight inch centers were used to provide .08 sq. in. of steel per foot.

Although theoretically no steel was required for the parabolic arches in compression, the same amount of shell steel was placed in both directions because of requirements for temperature and shrinkage reinforcement.

Compressive Stress in the Shell Concrete

The maximum compressive stress in the shell concrete under the design load was $f_c = \frac{1,050 \text{ lbs./ft.}}{2.3 \text{ in.} \times 12 \text{ in./ft.}} = 35$ psi.

Horizontal Interior Edge Beam

The total force in any edge beam is equal to the sum of the shear forces acting along its length. In the horizontal interior edge beam, Figure 5, the shearing forces, transmitted from the shell, build up to a maximum tensile

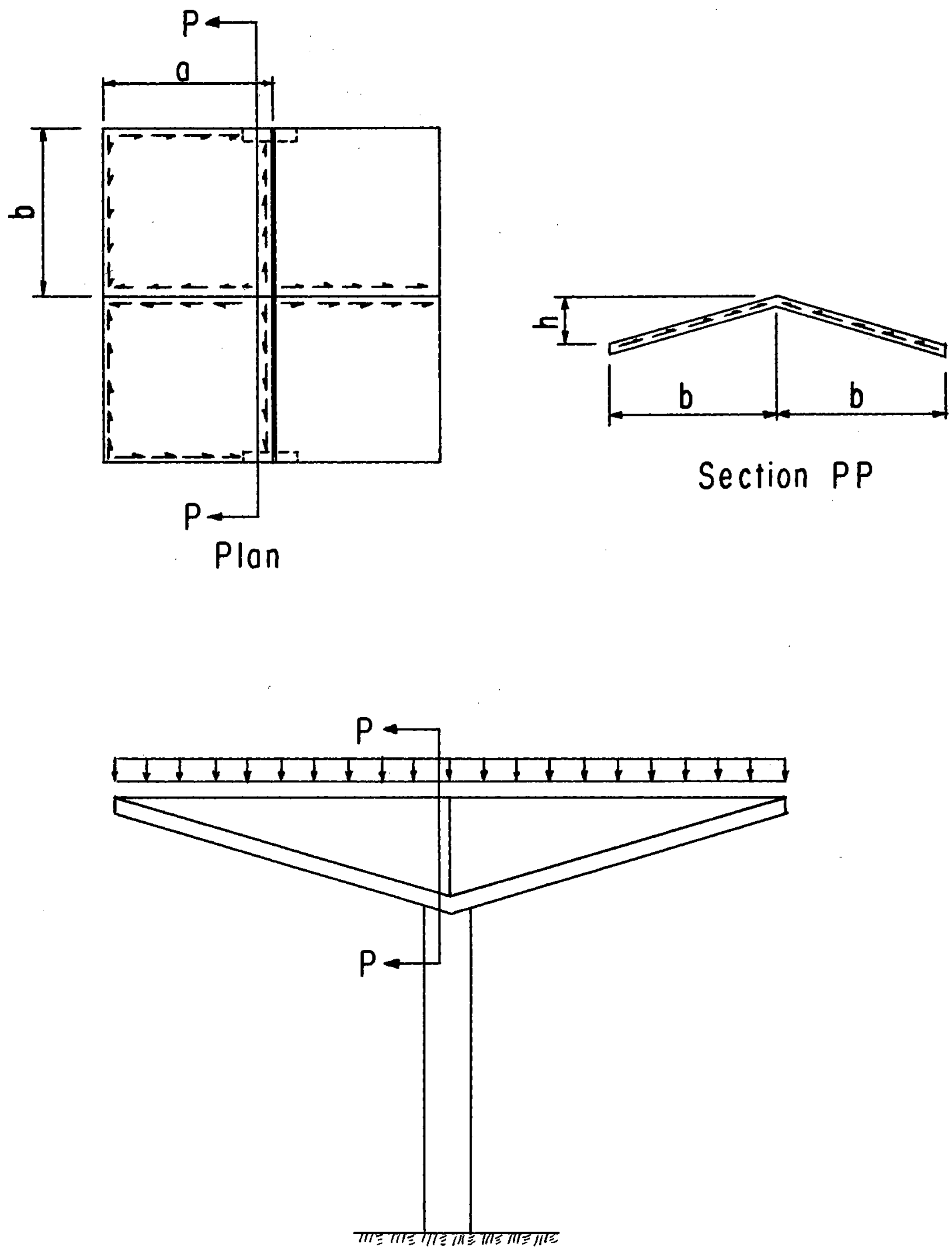


Figure 5. Shear Distribution in the Shell.

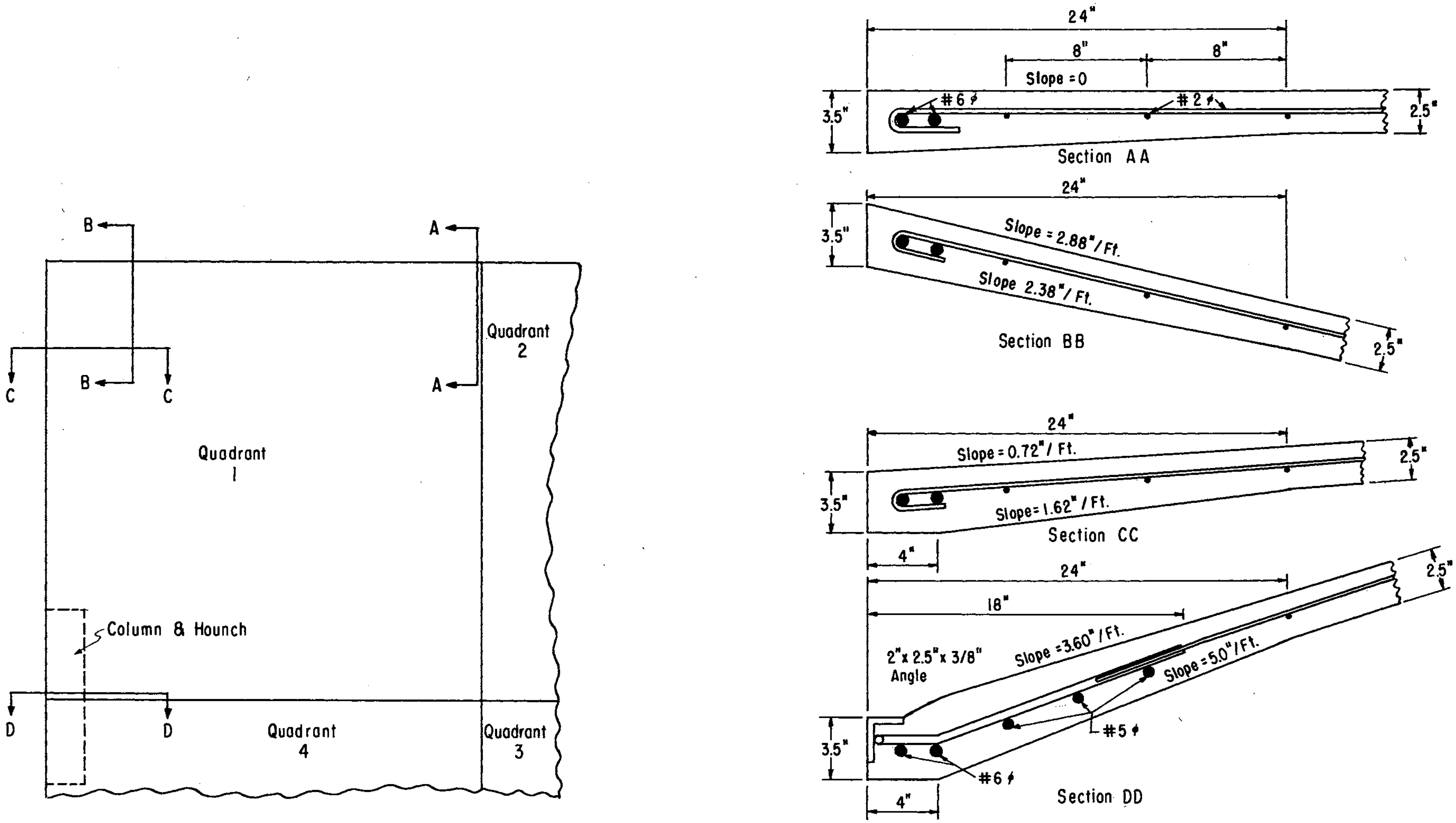


Figure 6. Cross-sectional Views of Exterior Edge Beam Sections.

force in the edge beam at the center of the roof. Therefore, the maximum force generated by the accumulated shearing forces are the forces acting in the edge beam over half of the roof span. Shearing forces on both sides of an interior edge beam contribute to the total direct stress in the edge beam.

Thus, the total tensile force, $H_T = 2 \times H \times a = 2 \times 1{,}050 \times 10 = 21{,}000$ lb. The area of steel to resist the tensile force was $A_s = \frac{21{,}000 \text{ lbs.}}{20{,}000 \text{ psi}} = 1.05$ sq. in.

A welded shear connection was selected to connect the precast quadrants; therefore, steel angles were used for all interior edge beams. Two 2 in. x 2 in. x 3/8 in. angles, $A_s = 2.72$ sq. in., provide adequate width and thickness for the attachment of dowels and flat bars by welding. The composite interior edge beam was made up of two angles connected by a 1½ in. x ⅜ in. flat bar welded on top of the angles to form a "T" shaped section.

<u>Sloped Interior Edge Beam</u>

From Figure 3, $AB = [(A'B)^2 + (A'A)^2]^{1/2} = (100 + 9)^{1/2} = 10.44$ ft.

The total compressive force was $H_c = 2H(AB) = 2(1050)(10.44) = 21{,}960$ lb. The compressive stresses are transmitted by the edge beam steel to the column. The steel area was $A_s = \frac{21{,}960}{16{,}000} = 1.37$ sq. in. This area of steel was furnished by two 2 in. x 2 in. x 3/8 in. angles, $A_s = 2.72$ sq. in.

Horizontal Exterior Edge Beams

The total compressive force in the horizontal exterior edge beam was $H_c = H \cdot a = 1{,}050 \times 10 = 10{,}500$ lbs/sq.in. The area of steel required to transmit the compressive stresses was $A_s = \frac{10{,}500}{16{,}000} = .656$ sq. in.

Two No. 6 deformed steel bars, $A_s = 0.88$ sq. in., were used. The shape and dimensions of typical sections of the exterior edge beams are illustrated in Figure 6.

Sloped Exterior Edge Beams

From Figure 5, the slope length of the exterior edge beam was $DA = [(DA')^2 + (AA')^2]^{1/2} = (100 + 9)^{1/2} = 10.44$ ft. The total compressive force was $H_c = H\ (DA) = 1{,}050 \times 10.44 = 10{,}980$ lb. The required area of steel to transmit the compressive stress was $A_s = \frac{10{,}980}{16{,}000} = .696$ sq. in. Two No. 6 bars were used. The compressive steel was also required to resist bending in the edge beam during eccentric roof loads.

Tension Tie Plate at Center of Roof

An area of steel of 1.05 sq. in. was required to take the calculated maximum tension at the center of the horizontal interior edge beam. However, as this is a critical structural point, an additional factor of safety was introduced by using a 2.0 in. x 3/4 in. x 24 in. flat bar which had an $A_s = 1.5$ sq. in. This was also used because of dead loads greater than the design load which were induced during the testing procedure.

The welding of this plate to the horizontal interior edge beams was also noted as a possible weak point; therefore, a weld leg width of 3/8 in. was made on both sides of the 24 inch bar. This gave a calculated allowable load capability of q = 48 in. x 3/8 in. x .707 x 5,000 = 63,630 lbs. under dynamic loading, or q = 48 in. x 3/8 in. x .707 x 14,000 psi = 178,160 lbs. under static loading. This weld connection itself allowed a factor of safety of 5 for static loading and would limit any failure of the horizontal interior edge beam to the steel.

Connection Between Edge Beam Angles and Shell

To provide adequate bond between the edge beam angles and the shell concrete, 10 in. dowels of No. 6 bars were used; each dowel was bent in the shape of an "L" to allow approximately 2 inches of welding surface against the angle. This allowed approximately 8 inches of length for the dowels to develop bond with the concrete, Figure 7. The dowels were spaced at 8 inch centers, to line up with the shell steel. The calculated bond stress at design load was $u = \frac{(V)}{\Sigma \cdot 7/8d} = \frac{(1{,}050)}{1.2(7/8)8} = \frac{1{,}050}{8.4}$, or u = 125 psi, which was less than the 350 psi allowable stress (25).

Connection of Shell to Column

To counteract localized radial shearing and bending

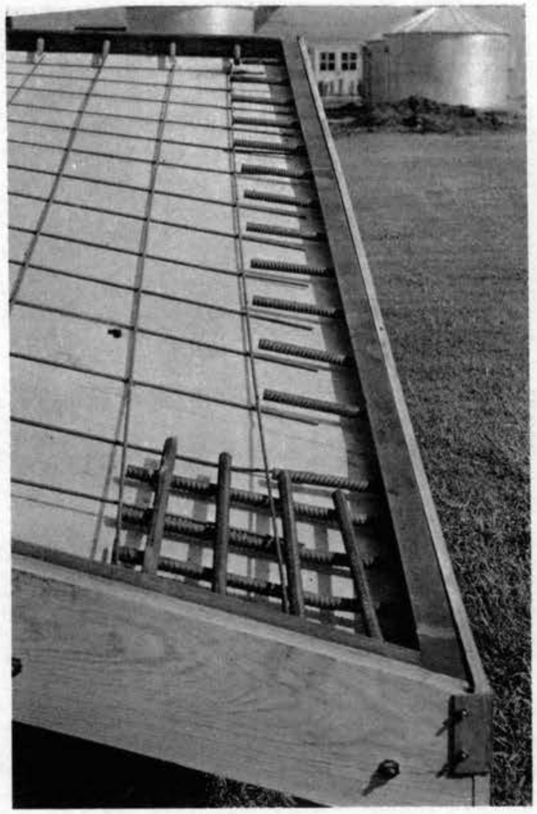


Figure 7. Shell and Edgebeam Reinforcing Steel.

Figure 8. Tie Connection Welded to Bearing Plate on Top of the Column.

stresses at the corner of the shell supported by the column, a reinforcing mat was constructed of No. 5 bars at 4 inch centers, Figure 7. This mat was approximately 18 inches square in plan with the bars bent to conform to the slope of the shell surface.

The mat bars perpendicular to the interior edge beam replaced the 8 inch dowels and were welded in the same manner. The bars parallel to the interior edge beam were welded to a 23 inch angle cast into the exterior sloped edge beam.

The 23 inch angle was cast into the exterior sloped edge beam at the lower corner, Figure 7, to connect the shell to the column. This angle was welded to the angle on top of the column haunch to develop the tensile strength of the haunch against overturning moments.

A special wide flange "T" section was also utilized to connect the shell to the column. This section was made by cutting the web of a 10 in. x $5\frac{3}{4}$ in. wide flange on a 3 by 10 slope to match the slope of the interior edge beam. This piece was then welded to a metal plate cast into the top of the column, Figure 8. When the two quadrants were lowered onto the column, the top slope of the inverted "T" section was even with the top slope of the edge beams for welding.

Lift Ring Design

Four lift rings were cast into each quadrant approximately at quarter points in plan, Figure 9. The lift rings were No. 5 bars, 18 in. long plus 6 in. pieces of No. 5 bars, as illustrated in Figure 10.

The allowable shearing strength of each ring was calculated assuming the total quadrant weight was carried by one lift ring. Thus, $S = \frac{P}{A} = \frac{3,260}{2 \times .31} = 5,260$ psi, which is considerably less than the 13,000 psi allowable working unit shear stress for structural steel (26). The rings were placed diagonally beneath the shell steel and spot-welded in order to transfer the lifting stresses into the entire shell.

Shell Dimensions

The nominal thickness of the shell was $2\frac{1}{2}$ inches. This depth would not give adequate coverage of the edge beam steel, therefore, a depth of $3\frac{1}{2}$ inches was used for the thickness of the exterior edge beams. A horizontal surface 4 inches wide was specified for the bottom side of the exterior edge beams to provide a uniform surface for juncture with future wall construction, Figure 6, section D-D.

Shell Concrete

To reduce the over-all weight which was lifted during assembly of the shell on the column, the shell quadrants were precast of lightweight aggregate concrete which had a 21 day density of 117 lbs./ft^3. Lightweight concrete,

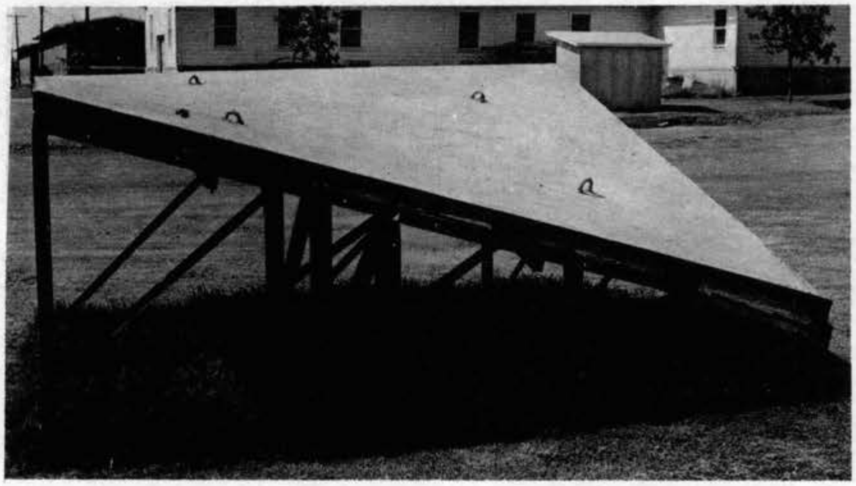


Figure 9. Precast Quadrant Showing Lift Rings.

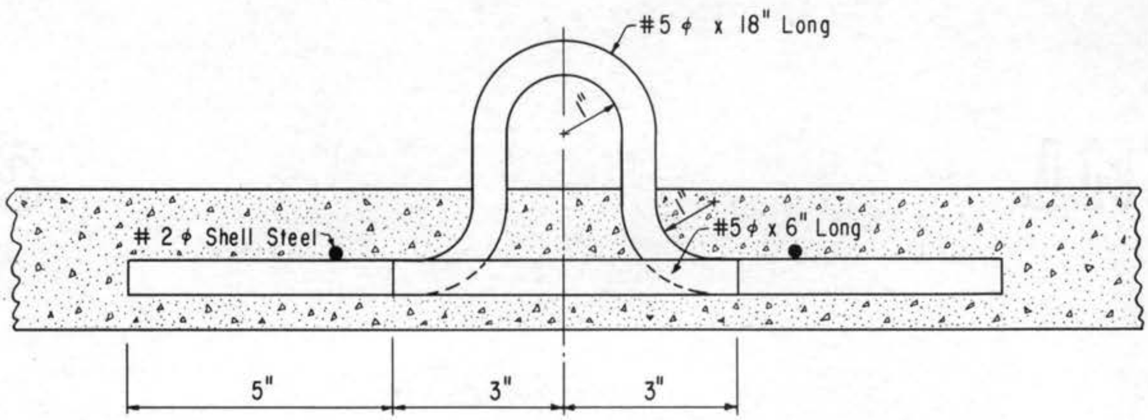


Figure 10. Lift Ring Detail.

having an ultimate strength of 3,750 psi, was requested from the concrete plant. The average test cylinder strength from three samples was 4,480 psi.

The volume of concrete required for the shell was determined by the $2\frac{1}{2}$ in. depth over 256 sq. ft. (horizontal projection) and 3 in. average depth over 144 sq. ft. (horizontal projection). Thus, $V_c = \frac{2.5}{12} \times 256 + \frac{3.0}{12} \times 144 = 53.32 + 36 = 89.32$ cu. ft. $= 3.31$ cu. yd. An additional 5% of volume was added to the total volume, therefore, $V_T = V_c \times 1.05 = 3.31 \times 1.05 = 3.47$ or 3.5 cu. yd. was required.

Support System Analysis

The support system for the structure consists of the column, haunch, tie bar, and footing. The design of each item is considered individually in the following paragraphs.

Column Design

One of the first considerations given to column design was the possibility of failure due to overturning moment. This moment could be composed of static loads on one-half of the roof (adjacent quadrants parallel to the centerline) plus wind forces. Conventional wind data do not apply to a roof configuration of this type; also, very little has been published concerning the behavior of wind forces on open sided h-p shells.

Mannschreck (27) conducted wind tunnel experiments on h-p shell models of square configuration in 1963. His tests were conducted for values of Reynolds numbers less than 8.0×10^5. Mannschreck used Reynolds number, $N_R = \frac{VL\rho N_e}{\mu}$, where L was the length of one side of the roof. For wind speeds of 60 feet per second on the 20 foot square prototype in this study, a value of $N_R = 6.54 \times 10^6$ would be obtained.

Mannschreck developed three dimensionless coefficients for the force components of lift, drag, and moment on the model. He tested models which had rise to span ratios of 1/6, 1/8, 1/10, and 1/12 at various ratios of column height versus roof span over a range of wind speeds up to approximately 60 feet per second. Each of the coefficients was plotted against N_R. The resultant lift force was applied at the center of the roof in Mannschreck's analysis, Figure 11, and does not cause eccentricity, therefore, only the coefficients for moment and drag will be considered.

The coefficients which Mannschreck developed are:

1. Drag coefficient, $C_x = \frac{R_x}{N_e \cdot H \cdot w \cdot \rho \cdot v^2}$
2. Lift coefficient, $C_z = \frac{R_z}{N_e \cdot H \cdot w \cdot \rho \cdot v^2}$
3. Moment coefficient, $M_o = \frac{M}{N_e \cdot H \cdot w^2 \cdot \rho \cdot v^2}$

where:

N_e = Newton's Second Law Coefficient = $\frac{1}{32.2} \cdot \frac{lb_f}{lb_m} \cdot \frac{Sec^2}{ft.}$

P = Air Density = .070 lb_m/ft^3

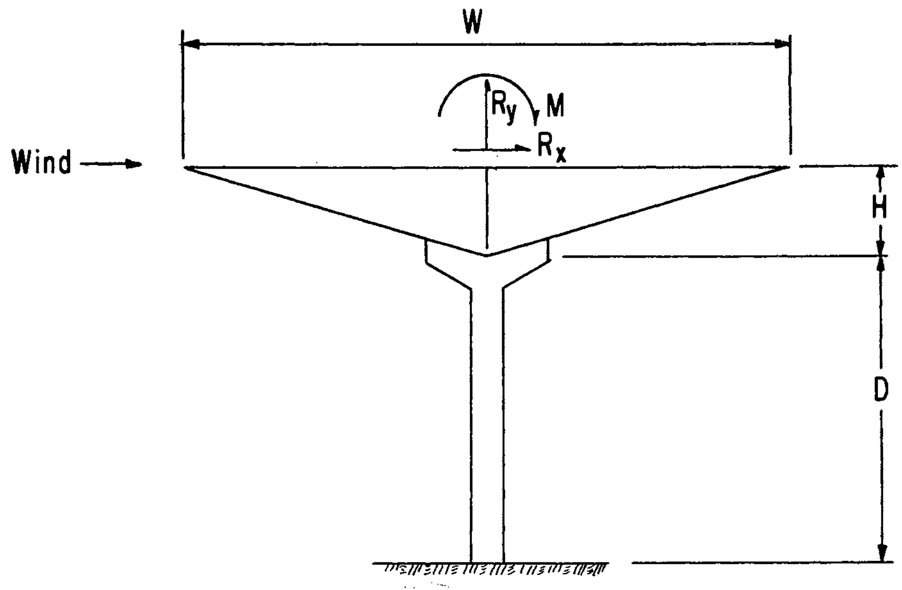


Figure 11. Side View of Structure Showing Lift, Drag, and Moment Reactions.

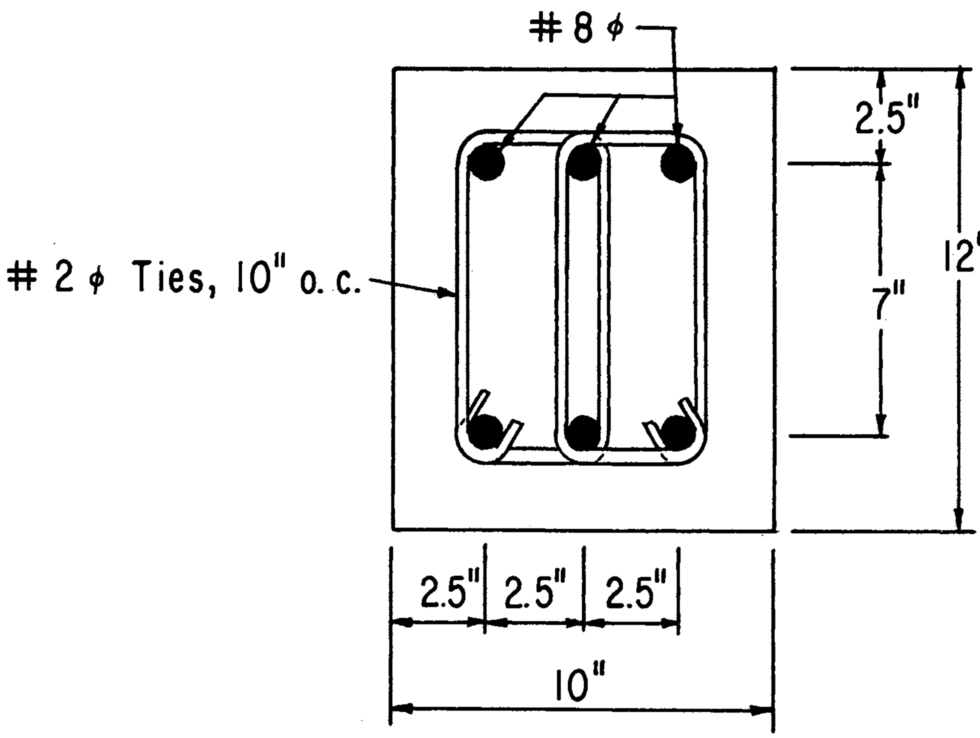


Figure 12. Column Steel Arrangement.

V = Wind Speed $\left(\frac{ft}{sec}\right)$.

By rearranging the values for the coefficients, equations can be set up to estimate the resultant reactions acting on the structure, or

4. Drag, $R_x = C_x \circ N_e \circ H \circ w \circ \rho \circ V^2$

5. Lift, $R_z = C_z \circ N_e \circ H \circ w \circ \rho \circ V^2$

6. Moment, $M = M_o \circ N_e \circ H \circ w^2 \circ \rho \circ V^2$.

For the plot of C_x versus N_e for a model which was dimensionally similar to the prototype in this study, C_x decreased as N_R increased. A conservative value of C_x would be the lowest value which was obtained during the model testing, C_x for $N_R = 7.91 \times 10^6$ would be approximately 0.180. For purposes of computing the moment due to Drag, a value of $C_x = 0.250$ will be assumed at $N_R = 7.91 \times 10^6$.

As N_R increased, M_o increased at a constant rate. It is not likely that the slope of the M_x verus N_R plot would remain constant for N_R up to 7.91×10^6 as this would give a value of M_o of approximately 1.20. As N_R increased from 8.0×10^5, the slope probably decreases as separation of streamlines occurred at the edge of the structure. The highest values of M_o obtained on the model was approximately 0.245 at $N_R = 8.5 \times 10^5$. A value of $M_o = 0.70$ will be assumed to compute the wind load moment acting on the shell. A value of wind speed = 66 fps will be assumed to be one-half of the maximum wind speed that would act on the structure at any time in the direction of overturning.

The maximum moment for design will consist of one-half roof load plus a wind moment computed from the half wind speed. Assuming ρ = .070, V = 66 fps, and $N_e = \frac{1}{32.2}$, the Drag Force, $R_x = C_x \cdot N_e \cdot H \cdot w \cdot \rho \cdot V^2 = 0.25 \cdot \frac{1}{32.2} \cdot 3.0 \cdot 20 \cdot .070(66)^2 = 142.5$ lb. Thus, $M_1 = R_x \cdot (H+D) = 142.5 \cdot 13.0 = 1{,}850$ ft.-lbs. The wind force moment, $M_2 = M_o \cdot N_e \cdot H \cdot w^2 \cdot \rho \cdot V^2 = \frac{(0.70)(3.0)(20)^2(0.070)(66)^2}{32.2} = 7{,}210.0$ ft.-lbs. The total moment due to one-half of the maximum wind velocity would be $M_w = M_1 + M_2 = 7{,}210.0 + 1{,}850 = 9{,}060$ ft.-lbs.

The moment due to design roof load on one-half of the roof would be $M_{DL} = p \cdot b \cdot \frac{w}{2} \cdot \frac{w}{4} = \frac{pbw^2}{8} = (40)(20)\frac{(20)^2}{8} =$ 40,000 ft.-lbs., or M_{DL} = 20,000 ft.-lbs./column. Therefore, the total moment acting on each column will be $M_T = M_w + M_{DL} = \frac{9{,}060}{2} + \frac{40{,}000}{2} = 24{,}530$ ft.-lbs.

By the method of ultimate strength design from Reinforced Concrete 38 (28) a design factor of 2 is used. The design axial load, P = 14.52 kips, and the ultimate design load, P_u = 2(14.52) = 29.04 kips. The design moment, M_T = 24.58 kip-ft., thus, the ultimate moment, M_u = 2(24.58) = 49.16 kip-ft.

From Table 6, "Eccentrically Loaded Tied Columns" (28) for f_c = 3,000 psi, f_y = 40,000 psi, and column size = 10 in. by 12 in., P_u = 30 kips, and M_u = 56 kip-ft., 4 No. 10 bars are recommended. However, for P_u = 30 kips, and

M_u = 48 kip-ft., four No. 9 bars are required, which have an area of steel of 4.0 sq. in. It was considered desirable to substitute six No. 8 bars, A_s = 4.74 sq. in., Figure 12, as this bar combination gave a better steel distribution in the column. The ties consisted of No. 2 bars spaced at 10 inch centers with two ties per set.

Haunch Design

The general dimensions of the haunch were selected and were then checked by analytical methods for the required lengths and depths of section, Figure 13. The top of the haunch was given a slope value of 17° which was approximately the slope of the exterior edge of the shell. The bottom of the haunch was assigned a slope of 30°. The horizontal length of the haunch from the face of the column was 18 inches. The vertical depth of the end of the haunch was 8 inches. For the haunch section at the face of the column, $M_{DL} = \frac{W(L^3)}{2} = 40 \times 500 = 20{,}000$ ft.-lbs., and $M_{MAX} = M_{DL} + \frac{M_W}{2} = 20{,}000 + 4{,}530 = 24{,}530$ ft.-lbs. One-half the M_{MAX} will be resisted by each haunch arm, thus, M = 12,265 ft.-lbs. From Table 1, Reinforced Concrete Handbook (29), K = 236 psi. Solving for the distance from the center of the reinforcing steel to the extreme fiber, $d = \left(\frac{M}{Kb}\right)^{1/2} = \left(\frac{12{,}850 \times 12}{236 \times 10}\right)^{1/2} = (65.4)^{1/2} = 8.08$ in., which was less than the 9.0 in. actual distance. The steel area required at the column face was $A_s = \frac{M}{f_s jd} = \frac{12{,}850 \times 6}{20{,}000 \times 7/8 \times 9} = \frac{61{,}700}{63{,}000} = .98$ sq. in. Two No. 7 bars,

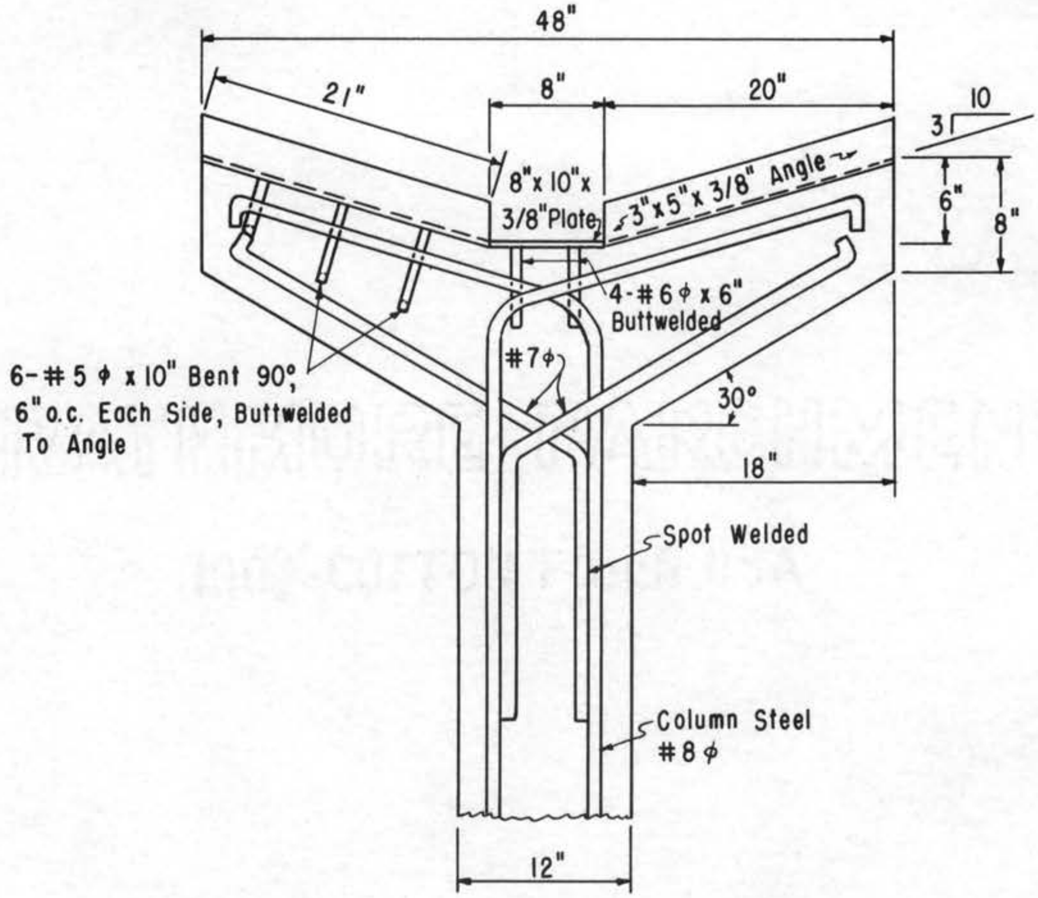


Figure 13. Haunch Detail.

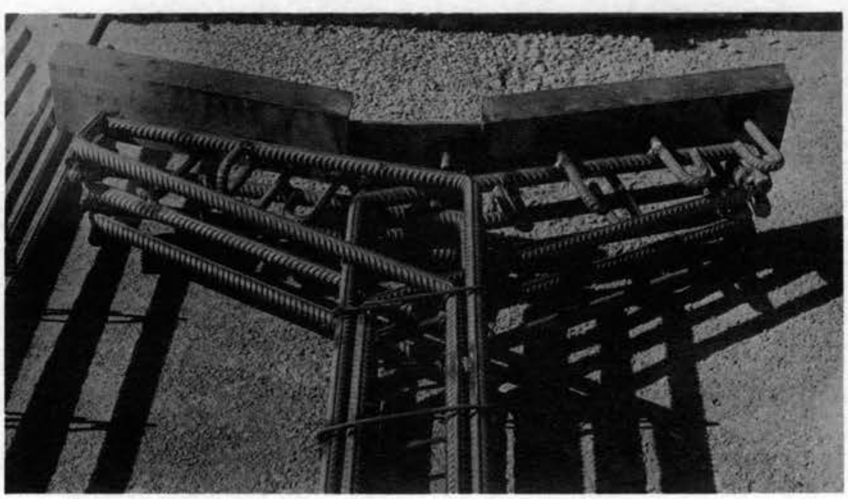


Figure 14. Column and Haunch Steel Cage.

$A_s = 1.20$ sq. in. were required for the bottom steel in each haunch. The top steel consisted of the three No. 8 bars which were continued from the column and bent to the desired shape. The required amounts of bond perimeter and shear capacity are $\Sigma_o = \frac{V}{ujd} = \frac{M}{L} \cdot \frac{1}{ujd} = \frac{12{,}265}{2 \times 300 \times 7/8 \times 9} = 2.62$ in. < 5.5 in., and $v = \frac{V}{bjd} = \frac{M}{Lbjd} = \frac{12{,}265}{2 \times 10 \times 7/8 \times 9} = \frac{5{,}140}{63} = 81.5$ psi < 90 psi, both of which were adequate.

To develop the moment capacity of both haunch arms simultaneously, a tension connection between the haunch and shell was developed by precasting steel angles into each member, Figure 14. These two angles were placed together during the erection process and were welded together to form a positive load transferring connection.

For a moment of 12,265 ft.-lbs. and a moment arm of 2 ft., the equivalent force acting upward at the end of the haunch will be 6,132 lb; therefore, two No. 5 dowels were used near the end of the haunch, and the required bond length was $a = \frac{A_s \cdot \frac{f_s}{2}}{u \cdot \Sigma_o} = \frac{.306 \times 20{,}000/2}{300 \times 1.96} = 5.2$ in. Three sets of No. 5 bars were spaced at 6 inch centers to anchor the 3 in. x 5 in. angle on each haunch, Figure 14.

An 8 in. x 10 in. x 3/8 in. bearing plate was cast into the top of the column to act as a base for anchoring the column tie. Four No. 8 bars, 6 inches long were used for dowels on the base plate, Figure 14.

Tie Bar Design

The thrust in each sloped interior edge beam was

$\frac{2 \cdot H \cdot a}{\cos 17°} = 2 \cdot 10{,}500 \cdot 10.43 = 21{,}950$ lbs. The horizontal component of this thrust was $H_c = 2 \cdot H \cdot a = 21{,}000$ lbs. $A_s = \frac{H_c}{f_s} = \frac{21{,}000}{20{,}000} = 1.05$ sq. in. The A_s was furnished by one No. 10 bar which had an area of 1.47 sq. in.

The length of weld which was necessary to develop the full strength of the tie bar was $L = \frac{P}{.707 h \cdot S_s} = \frac{21{,}000}{.707\ (3/8)(14{,}000)} = 5.66$ inches. The tie bar was welded to the inverted "T" sections, which were welded to the bearing plate on each column, after the column footings were cast.

Footing Design

The basic design of the footing was taken from a study on pole type buildings (30). The depth of set formula specified that the equation was used to determine the required depth of embedment where no constraint was provided at the ground surface. This empirical equation, $d = \frac{A}{2}\left[1 + \left(1 + \frac{4.36h}{A}\right)^{1/2}\right]$, specified the following parameters:

$A = \frac{2.34P}{S_1 \cdot b_1}$

P = Applied horizontal force or equivalent in kips.

S_1 = Average soil resistance above the point of rotation in ksf.

S_2 = Average soil resistance below the point of rotation in ksf.

b_1 = Diameter of round post or the diagonal dimension of a square post, in ft.

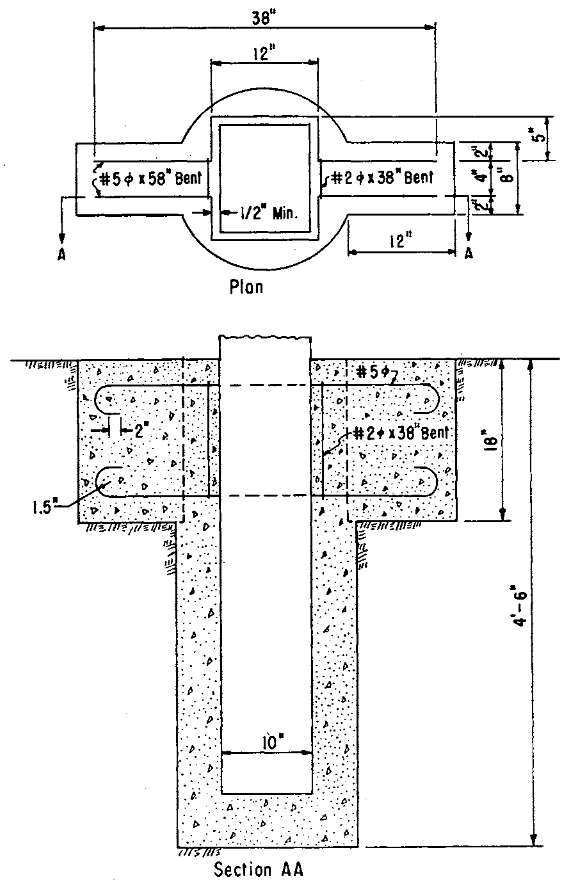


Figure 15. Footing Details.

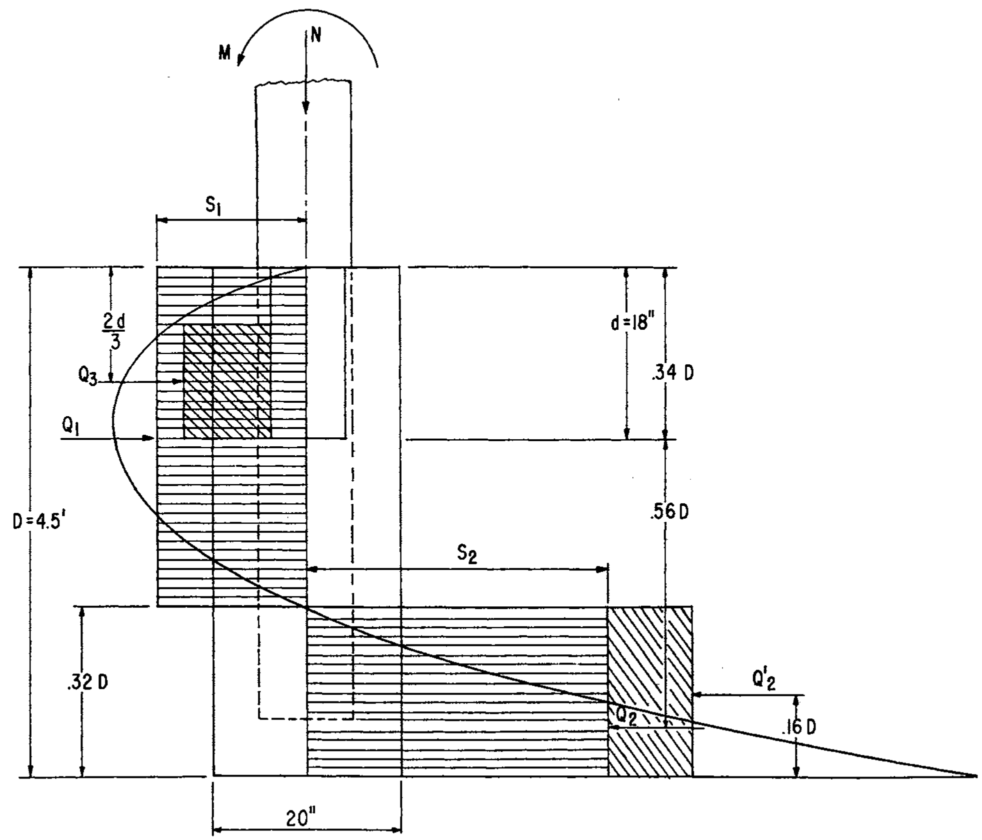


Figure 16. Soil Reactions to Footing and Wing Walls Against Overturning Moments.

h = Distance, in feet, from ground surface to the point of application of P.

d = Depth of embedment of post.

For the clay soils in this area, S_1 was assumed to be 3,500 psf, M was 24.530 kip-ft., h = 10 ft., P = 2.45 kips, and b_1 = 1.31 ft. Thus, $d = \frac{1}{2} \cdot \frac{2.34P}{S_1 \cdot b_1}\left[1 + \left(1 + \frac{4.36h}{\frac{2.34P}{S_1 \cdot b_1}}\right)^{1/2}\right]$

$$= \frac{1}{2} \frac{(2.34 \times 2.450)}{(3{,}500 \times 1.31)} \left[1 + \left(1 + \frac{4.36\ (10)}{\frac{(2.34 \times 2.57)}{(3{,}500 \times 1.31)}}\right)^{1/2}\right]$$

$= \frac{1}{2}\ (1.310) \left[1 + (1 + \frac{43.6}{1.310})^{1/2}\right] = (0.656)(6.86)$, or d = 4.50 ft. The depth of the foundation holes was approximately 4.5 ft.

The minimum recommended diameter for foundation holes, Figure 15, is b_1 + 4 in. = 15.6 in. + 4 in. = 19.6 in., or 20 in. The footing excavation was dug with a 16 inch rotary drill, then hand finished to a diameter of 20 inches.

To support the structure in bearing, the area of the footing base was determined and checked for adequacy. The total design weight at the bottom of the column was Wt./column = $W_{Quad.} + W_{Col.} + W_{Foot} + W_{Design}$ = 6,400 + 2,265 + 1,000 + 8,000 = 17,665 lbs. Assuming the soil bearing capacity, P = 5,000 lbs./sq. ft., the required bearing area was $A_B = \frac{W_T}{P} = \frac{17{,}665 \text{ lbs.}}{5{,}000 \text{ psf}}$ = 3.53 sq. ft., which left an additional bearing area required of 3.53 - 2.18 = 1.35 sq.ft., where the area of the 20 inch diameter hole was 2.18 sq. ft.

The additional bearing area was supplied by two cantilever wings 8 inches wide, 12 inches long, and 18 inches deep, which were placed parallel to the column tie bar along the centerline of the structure. These two wings furnished a bearing surface of $\frac{(8 \text{ in.} \times 12 \text{ in.})2}{144 \text{ sq. in./sq. ft.}} = 1.33$ sq. ft., which was sufficient.

The equation used earlier to determine the required depth of the footing is an empirical expression which accounts for the overturning moment. However, the wing walls will also resist overturning by developing the passive earth pressure of the soil. Assuming the top 6 inches of soil was disturbed and, therefore, not effective, the overturning resistance developed by the wing walls was (Figure 16) $Q_3 = S \times A = 3{,}500 \text{ lb./ft.}^2 \cdot \frac{(12 \text{ in.} \times 12 \text{ in.} \times 2)}{144 \text{ in.}^2/\text{ft.}^2}$ $= 7{,}000$ lbs. Assuming that Q_1 acted at $\frac{2d}{3}$ and the overturning moment, M caused pivoting at .68 D, which was approximately $\frac{2D}{3}$, the force to be resisted by the wings was $F = \frac{M}{h} = \frac{24{,}530}{3{,}0-1.0} = 12{,}265$ lbs. The maximum force, Q, which had to be developed by the footing was 12,265 lbs. - 7,000 lbs. = 5,265 lbs. or a moment of 10,530 ft.-lbs. The addition of the wing walls to the circular footing gives a conservative value of resistance of overturning moment of $M_T + M_F + M_W = 24{,}530 + 14{,}000 = 38{,}530$ ft.-lbs.

The wing wall must withstand moments in two directions; when loaded, the wall must act as a cantilever beam in the vertical direction, and when acted upon by overturning moments, it must act as a cantilever beam in the horizontal plane.

Checking the moment in the horizontal or overturning plane, $F_1 = \frac{Q_3}{2} = \frac{7,000}{2} = 3,500$ lbs. (maximum horizontal force), and $M_1 = \frac{F_1 \times L_1}{2} = 3,500$ lbs. $\times \frac{1}{2}$ ft. $= 1,750$ ft.-lbs. The maximum moment which the reinforced wall was capable of developing was $M_{c_1} = Kbd^2 = 236 \times \frac{18}{12} \times (5)^2 =$ 8,850 ft.-lbs., which was greater than $M_1 = 1,750$ lbs. Thus, the required area of steel, $A_{s_1} = \frac{M_1}{ad} = \frac{1,750}{1.44 \times 5} =$ .243 sq. in. Checking the moment in the vertical plane for bearing, $F_2 = P \times A = 5,000 \times \frac{8 \times 12}{144} = 3,330$ lbs. $M_2 = F_2 \times \frac{L_2}{2} = 3,330 \times \frac{1}{2} = 1,665$ ft.-lbs., $M_{c_2} = Kbd^2 =$ $236\frac{(8)}{12} \times (15)^2 = 35,400$ ft.-lbs. $> 1,665$ ft.-lbs., and $A_{s_2} = \frac{1.665}{1.44 \times 14} = .076$ sq. in. Two No. 4 bars top and bottom, $A_s = .40$ sq. in., were used to satisfy A_{s_1}.

The A_{s_1} was the governing value of steel area, so this area was checked for bond. For $f'_c = 3,000$ psi, and $V_{max} = S_1 = 3,500$ lbs., $u = \frac{V_{max}}{\Sigma_o \cdot jd} = \frac{3,500}{3.1 \times 7/8 \times 6} =$ 215 psi $<$ 300 psi. Four No. 4 bars were used for the wing wall steel, Figure 15.

CHAPTER V

DEVELOPMENT OF ASSEMBLY COMPONENTS

Introduction

The development of several assembly components and techniques, in addition to the structural elements themselves, was necessary for final assembly of the precast elements. This involved (1) the design and construction of shell and column forms, (2) the footing reinforcing cage, (3) the assembly support system, (4) the lifting frame, and (5) a means for supporting and stabilizing the column while casting the footings. Also involved were (6) the casting and curing operations for the shell quadrants and columns.

The design procedures involved in developing these components were mainly investigations of maximum stresses to assure safe construction conditions during the erection of the structure and were not intended to be a complete and detailed design.

Formwork and Precasting

Column Forming

The column forms were constructed from 2 in. by 12 in.

Douglas fir lumber. This type of lumber was rigid enough to maintain form dimensions with a minimum number of braces. Disassembly time of the forms was reduced by the use of double headed forming nails which allowed workmen to pull all nails with nail bars.

Steel fabrication was complicated by the special haunch at the top of the column. This was formed by bending the column steel from one side of the column to form the top steel in the opposite haunch, Figure 14. To keep the bending process as simple as possible, the column steel was offset one-half the width of the bars, 1/2 inch on each side of the column to allow the cross-over bars to pass without special bending.

The bottom haunch steel was formed by No. 7 bars four feet long with a 60° bend approximately 18 inches from the end. Three bars were placed on each side. The 18 inch leg of each bar was spotwelded to the three column steel bars on the opposite side of the column. This placed each lower haunch bar directly below the top bar.

By welding the lower haunch bars to the column steel and spotwelding the ties, the steel was formed into a cage which could be handled and moved about as a unit. This was helpful when moving the steel from its construction location to the casting site. Also, a minimum of supports and attachments were needed to keep the steel properly spaced when placing concrete.

An outdoor concrete floor slab was used for a casting bed. The column forms were blocked up to facilitate lifting with a fork truck. Each form was coated with a bond breaker compound prior to placing the reinforcing steel. The column steel cage, which weighed approximately 360 pounds, was moved to the casting site by a small hoist on a farm tractor and lowered into the forms. The steel reinforcing cage was supported on the haunch end of the form by the 3 in. x 5 in. steel angle cast into the haunch to serve as the shell connecter, Figure 14.

The 3,000 psi concrete was delivered to the casting site by a ready-mix truck to simulate prototype casting procedures. A three-man crew cast the concrete using an electric vibrator for uniform placement. The entire casting operation including the finish troweling of the surface lasted approximately 1 hour and 30 minutes.

Shell Forming

The forms used to precast the shell quadrants were previously used to cast an inverted umbrella shell 20 feet square in plan. The forms were made up of four feet square modular units on a metal framework. Four of these units were combined to form an 8 ft. x 8 ft. form unit. A 2 ft. section was added between each 8 ft. quadrant unit to form the interior edgebeam section, and a one ft. extension section was added to the outer edge to form the horizontal edge.

This set of forms was modified by deleting the two form extensions and adding a new 2 foot extension to the 8 foot base to form the outer edge beams. The slope of the form extension was tapered to a $3\frac{1}{2}$ inch edge depth to provide a thickened edge beam section without altering the shell's top slope, Figure 6.

Side forms were made from 1 in. x 8 in. yellow pine lumber. The edge height was adjusted vertically to $3\frac{1}{2}$ inches and holes were drilled through the boards to match pre-located holes in the steel angles; then, one-fourth inch bolts were inserted and tightened. Braces were bolted to the forms at the corners to increase form stiffness.

The 2 in. x $2\frac{1}{2}$ in. x $\frac{3}{8}$ in. angles, which made up the interior edge beams, were cut to length. The corners were cut at 45° angles to form a 90° corner angle between the horizontal and sloped edge beam; both angles were bent down slightly along the 45° cut for welding. Ten inch long No. 6 dowels were welded to the edge beam angle to bond the shell and edge beam. A 90° bend was made 2 inches from one end of each dowel to provide a welding edge. The two end dowels were tackwelded in place parallel to the form slope. When the edge beam angle was removed from the form, a straight steel bar was clamped to the two end dowels to act as a welding guide for the rest of the dowels.

The two No. 6 bars which comprised the exterior edge beam steel were heated and bent around the corners. The bar ends were buttwelded to the interior edge beam angles at each end. Thus, continuity, with rigidity and effectiveness of stress transfer was obtained. The two No. 6 bars were spaced at 2 inch centers with the outside bar centered 2 inches from the edge of the concrete. The bars were placed at the center of the edge beam depth for efficient stress transfer.

The reinforcing mat in the corner of the quadrant above the column was constructed from 18 inch long No. 5 bars, which replaced the 8 inch No. 6 bars in the lower corner of each quadrant. They were spaced at 4 inch centers along the edge beam and the 23 inch angle cast into the exterior edge beams. The bars welded to the short angle were bent upward 4 inches from the angle to conform to the slope of the shell. The short angle was notched to fit against the end of the edge beam angle for welding.

Preparation of the shell steel consisted of cutting the No. 2 bars into 10 foot lengths and making a 180° bend in the bars 4 to 5 inches from one end. The other end of the bars were left straight to overlap the 8 inch dowels of the interior edge beams.

Prior to placing the steel in the forms, all cracks between sections of the form plywood were covered by strips of plastic stretched tight and stapled to the form. Then, the form surfaces were given a heavy coat of form oil to keep the concrete from bonding to the forms.

After spraying the forms with the bond breaker, the steel angles were placed in position and clamped to the side forms. Then, the four quadrants were placed together and leveled. The edge beam angles between quadrants were checked and all corners found to be within 1° of 90° angles. The edge beams were aligned with a maximum allowable clearance gap of one-half inch, which was the design tolerance.

The steel was placed by a five-man crew, consisting of a foreman, two welders, and two laborers. The shell steel spacings were marked off by one workman on the form sidewalls for rapid alignment of the steel. Two workmen placed the steel on the forms while one workman spaced and tied the steel, Figure 17.

One welder welded the corners of the interior edge beams and the short angles at the low corner. The second welder buttwelded the exterior edge beam bars to the interior edge beam, then tightened the shell steel and spotwelded the shell steel to the edge beam dowels to keep the steel network rigid. The shell steel was tied at alternate junctions in both directions. Four lift rings were placed under the shell steel at quarter points from the edges and tackwelded. Three-fourth inch thick wooden blocks were placed at various points under the steel junctions so that the steel would not be over one-fourth inch from the center of the shell thickness at any time. A length of wire was attached to each block so that they could be removed as the concrete was poured with the steel

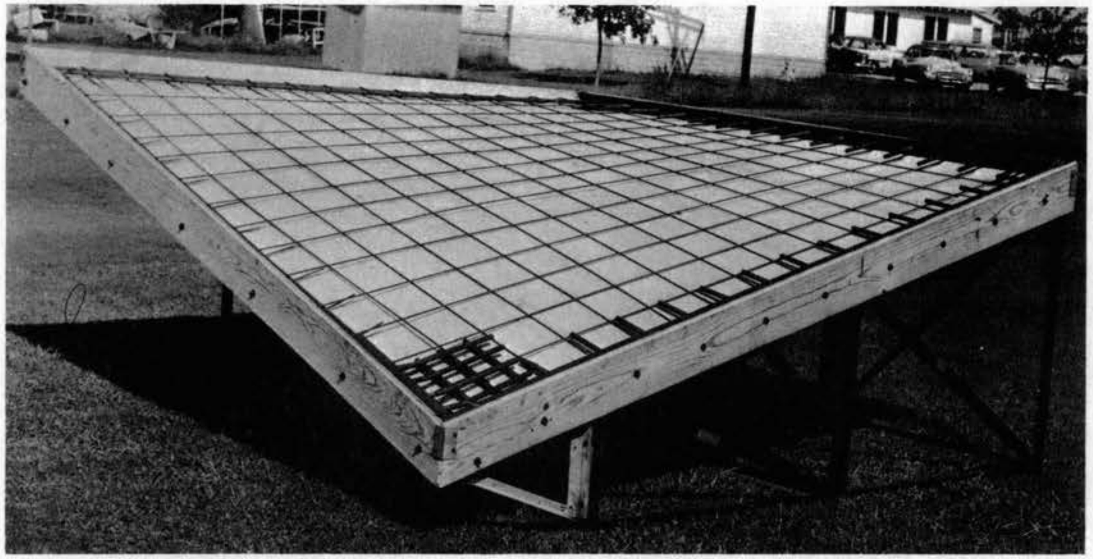


Figure 17. Shell Form Ready for Precasting.

supported by the fresh concrete.

Final steel forming was completed prior to the arrival of the concrete ready mix truck. A five-man crew, excluding the truck operator, was used to place and work the concrete. Two men began moving the concrete on the forms while the other three men worked the concrete around the steel. The concrete had to be rodded and vibrated under the angles and shell steel to reduce voids.

After the first quadrant was cast, one man placed the concrete as it came from the chute, one man worked the concrete under the angles and vibrated the forms, two men worked and screeded the concrete on the main part of the shell, and one man finished with a wooden trowel.

When approximately one-third of the shell was covered androughly smoothed by rake and shovel to the approximate depth, two workers began screeding the concrete with a 14 foot screed. The third man continued working the concrete under the edge beam angles and the shell steel. Just before the first quadrant screeding was completed, one worker moved to the next quadrant form and began placing the concrete on it. The finish man began wood troweling the first quadrant when the screeding was past the midpoint in the shell.

The concrete began to dry and became stiff by the time the third quadrant was cast and had to be tempered. This was due to the length of time required to cast all four shells. The concrete would have been more consistent and workable throughout the casting period if the load had been ordered on two trucks spaced at one hour intervals. The total time required for casting the four quadrants was two and one-half hours.

Curing Precast Elements

Column Curing

The curing operation on the columns was begun after the concrete had hardened for approximately two hours. Two layers of burlap material were placed over the top surface of the columns. A perforated sprinkler hose was laid down the center of each column. The sprinkler hose pressure was adjusted to keep the burlap continuously soaked. After the columns had cured under moist conditions for eight days, the sprinkler hoses were removed. The burlap material was left in place until the end of 14 days. Then the covering was removed and the columns cured in the forms with no covering.

Shell Concrete Curing

The shell curing process was initiated approximately one hour after the fourth quadrant was cast. Each quadrant was covered with two layers of burlap material and a four milli-inch thickness of clear plastic. The plastic was weighted down securely so that wind gusts would not blow it off. The quadrants were watered twice daily during the first four days, and in the mornings only during the next four days. No water was added after the eighth day. The sun shining through the plastic covering during the day raised the curing temperature and vaporized the moisture. This produced a curing condition similar to factory controlled curing. After 14 days, the plastic covering, burlap material and side forms were removed. Figure 9 shows a typical quadrant ready to be removed from the form.

Footing Steel

The footing steel was formed into a rectangular cage with the inside dimensions approximately 11 in. x 13 in. The cage was designed to allow a clearance of approximately one-half inch on all sides of the precast column as it was lowered into the footing excavation. This configuration of steel reinforcement was designed to give maximum anchorage and bond to the wing wall steel in order that the wall could develop its full potential in bending. Additional anchorage of the steel was also provided by hooking the ends of the footing steel.

Assembly Supports

The construction procedure selected for this study specified that a system of supports be developed to hold the precast roof quadrants in place during the erection process. The procedure also required that the quadrants be held rigidly, without uncontrolled movement or deformation of supports until the erection was completed. These requirements were met by the development of a rigid support frame or assembly jig, which was supplemented by three wooden supports.

Assembly Frame

The assembly jig, Figure 18, was developed to fulfill four basic requirements prior to and during the erection of the structure. These were:

(1) To rigidly support the corners of the shell quadrants during the assembly and final erection steps.

(2) To provide vertical adjustment of corner towers for precise control of the shell corner elevations.

(3) To provide a means of elongating or shortening the distance between the tower caps for ease of horizontal spacing.

(4) To provide a method of clamping the quadrants together for welding.

To satisfy these requirements, the rigid support framework illustrated in Figure 18 was designed to provide:

(1) A tower cap which could be adjusted to various slopes of exterior edge beams.

(2) A set of top and bottom horizontal braces to stabilize the corner towers.

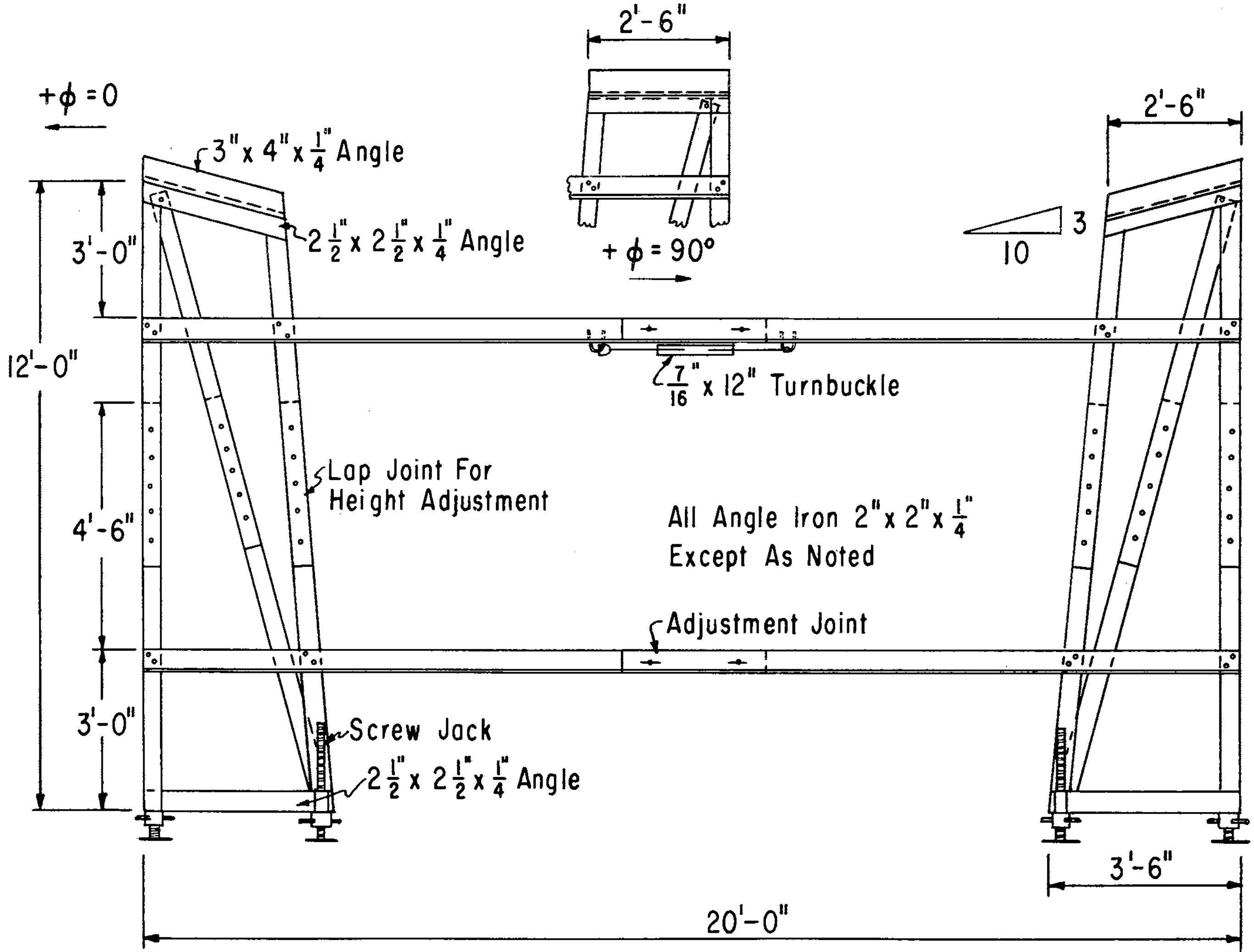


Figure 18. Assembly Jig.

(3) A turnbuckle mounted on the top horizontal brace which could be extended or retracted over a range of approximately 12 inches.

(4) Three screwjack legs in the base of each tower to provide fine adjustment vertically and to plumb the towers.

(5) Large increments of vertical adjustments in the corner towers by overlapped tower leg sections and braces with spaced bolt holes.

The design of the assembly frame was based on a weight per quadrant of approximately 3,200 pounds. By using a design factor of 2.0, the working load per quadrant was 6,400 pounds. During the assembly, each quadrant was supported by the assembly jig tower, the concrete column, and two wooden supports. The maximum stress condition for the tower would probably occur with the quadrant supported by the tower and the wooden support at the center. Then, the tower would support one-half of the working load, or 3,200 pounds.

The corner leg of the tower was designed to carry the full load of 3,200 pounds. The slenderness or l/r ratio governed the design; thus, for steel columns (26),

$$f_c = \frac{18{,}000}{1 \quad \frac{(84/.39)^2}{18{,}000}} = \frac{18{,}000}{3.58} = 5{,}030 \text{ psi, and}$$

$P = f \cdot A = 5{,}030 \times .94 = 4{,}730$ pounds was the safe load which the support could carry concentrically, compared to the applied load of 3,200 pounds.

The frame was welded at the corners where the legs joined the base and top angles of the towers. Each end of the diagonal brace was bolted to the top and base angles to allow rotation when the tower height was changed. The screwjack legs were fitted into one and three-fourths inch inside diameter pipe sleeves which were welded to the tower base. The tower cap was bolted to the top of the tower by eight $1\frac{1}{2}$ in. x $\frac{3}{8}$ in. counter-sunk headed bolts. Slope adjustment of the tower cap was made by shims or washers placed between the cap and the top of the tower. Slight adjustments in the slope of the cap could be made by adjusting the screwjack adjacent to the concrete column. The horizontal frame braces were bolted to the corner and outside leg of the tower and at the two center adjustment slots by $\frac{3}{8}$ inch bolts. The turnbuckles were connected to one section of the top brace by a welded ring; the opposite end of the turnbuckle was bolted to a ring on the overlapping brace section.

Temporary Wooden Supports

Three wooden supports were necessary to stabilize the roof quadrants vertically. Adequate horizontal stabilization was provided by the assembly frame and the two columns. The wooden supports were constructed principally for this project, therefore, they were designed to give small vertical adjustments.

From the topographic survey of the construction site, the ground elevations at the support points were determined. The distances from the ground to the lower side of the roof at these three points were then determined so that the towers could be constructed to the approximate heights necessary to maintain the proper horizontal interior edge beam elevation.

Center Support

The center support legs were constructed from two pieces of 4 in. x 4 in. x 14 ft. lumber spaced 12 inches apart, with 2 in. x 4 in. members for diagonal braces. Two 2 in. x 6 in. x 20 in. members were nailed horizontally on both sides of the support legs at the base.

The total calculated load carried by the center support assuming that one-third of the weight of each quadrant was supported by the center support, was $3{,}200 \times 1/3 \times 4 =$ 4,270 pounds. For a design factor of 2, the design load was 8,540 pounds. The cross-sectional area of the support legs was 26.28 in^2 which gave the support a load capacity of $P = A \cdot f = 26.28 \cdot c(1 - \frac{\ell}{80d}) = 26.28 \cdot 1{,}200(1 - \frac{36}{80 \times 3.66})$ $= 26.28 \cdot 1{,}050 = 27{,}590$ pounds.

Vertical movement of the support was supplied by two screwjacks on metal brackets bolted to the base of each leg. A 3 inch length of $1\frac{3}{4}$ inch inside diameter pipe was welded on the bracket to act as a sleeve for the screwjacks.

The top of the support was made up of a 4 in. x 4 in.

cap and a 2 in. x 12 in. scabbing plate on each side, Figure 19. A 4 in. x 4 in. member was bolted to each scabbing plate. The top surface of the support was planed down at a 17° angle to conform to the slope at the interior edge beam.

The placement and removal of the center support was complicated by the tie bar which it straddled. This problem was solved by the removal of an 8 inch section of one leg while the support was being placed over the bar or being removed. The short section was braced by 6 inch metal plates which were bolted in place.

End Supports

The two end supports were designed to support a maximum load of one-half the working load on the center support or $P = \frac{8{,}540 \text{ lbs.}}{2} = 4{,}270$ lbs. The construction of a satisfactory supporting surface was completed by using a 24 inch column cap held in place by a 2 in. x 12 in. scabbing plate on both sides of the cap. This gave the support top dimensions of approximately 7 inches by 24 inches. The legs were spaced 8 inches apart and braced at 24 inch intervals by a 2 in. x 6 in. member on each side. The base of the support was constructed so that small height adjustments could be made by wooden wedges. Larger adjustments were made by placing shims beneath the base of the support.

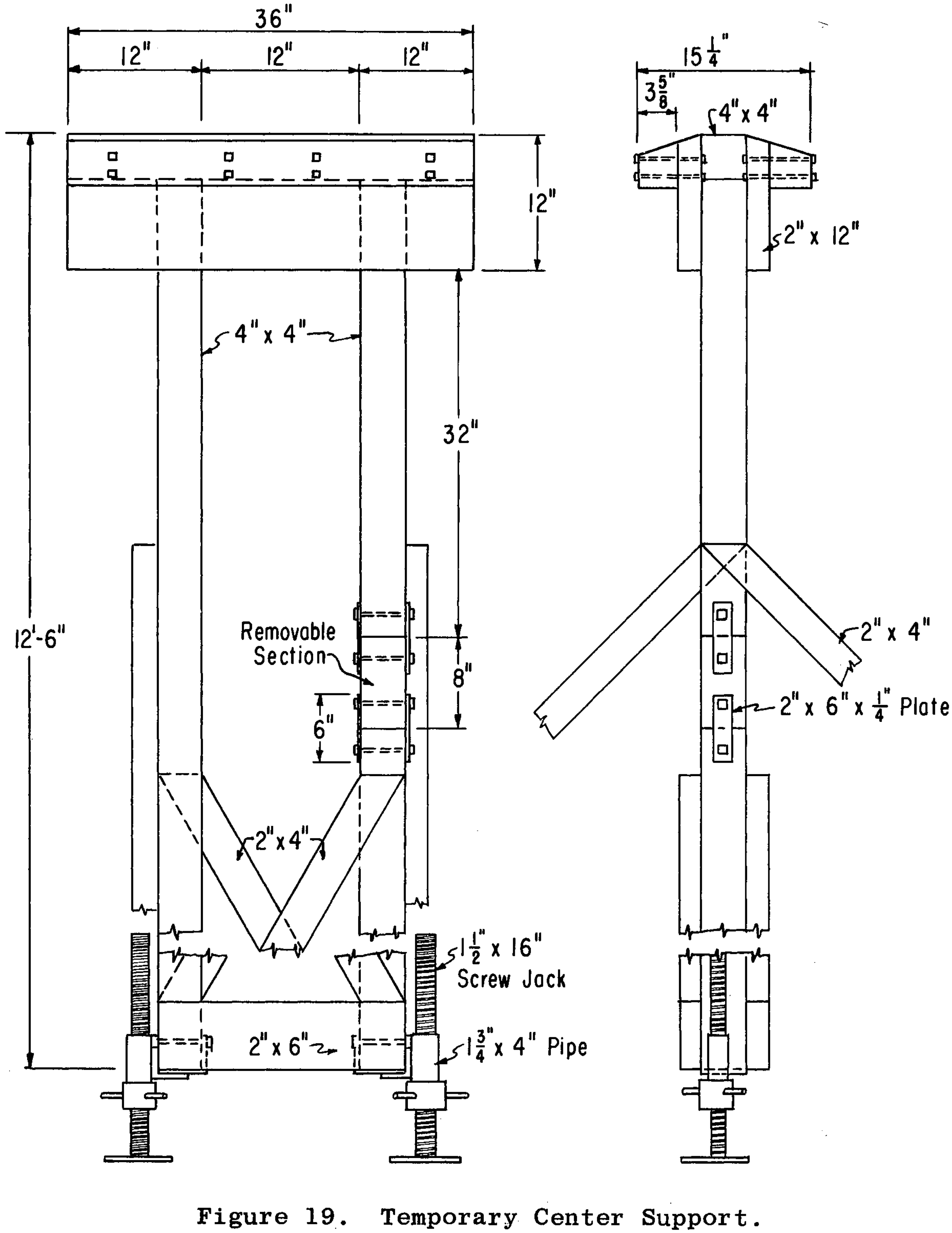


Figure 19. Temporary Center Support.

Lifting Frame

A special frame was developed to provide a vertical lift on all lift rings during removal of the quadrant from the forms and during erection, Figure 20. The lift frame also worked quite well when the quadrants were to be lifted with the surface sloped at various angles.

The frame had a square configuration with a diagonal brace. The frame sides and brace were constructed from 2 in. x 2 in. x 3/8 in. steel angles. Braces were placed across each corner at 45° angles. The main diagonal brace was welded to two of the corner braces. A 4 inch length of 3 inch pipe was welded flush with the top of the frame in each corner; one-fourth inch holes were drilled 3 inches from the top of the pipe section so that a one-fourth inch bolt could be inserted. These bolts were placed through one link of a three-eighths inch diameter chain to maintain the angle between the chain and lifting frame.

The lifting mechanism was completed by two lengths of three-eighths inch chain with hooks on each end. The chain ends were placed through the pipe sections in adjacent corners of the frame so that lifting stresses would be evenly distributed into both chains. The vertical chain angle was adjusted to approximately 45°, then the chains were bolted at the corners. Each chain hook was passed through a lift ring on the shell, then hooked back to the chain to give the quadrant surface the desired slope.

The maximum compressive stress in the frame occurred in the diagonal brace. The calculated load in each of the four chain legs was $\frac{P}{4} \times \frac{1}{\sin 45°} = \frac{3,200 \text{ lb.}}{4(0.707)} = 1,130$ lbs. tensions under a static load (31). The dynamic load which would be developed by a dynamic load design factor of 3.0 was $P_w = 3,390$ lb. per chain section. The critical load condition would occur when the entire load was supported by a chain connected to diagonal corners of the frame parallel to the diagonal brace. This situation would produce a calculated dynamic load, $P_w = 6,780$ lbs. and a compression of $P_w \cos 45° = (6,780)(.707) = 4,800$ lbs. in the diagonal brace. Checking for buckling gives $P_{allow.} = A \cdot f = (.94)\frac{(18,000)}{1 + \frac{(\ell/d)^2}{18,000}} = \frac{16,920}{3.71} = 4,560$ lbs. For $P_{allow} = 4,560$ lbs., the calculated factor of safety under the critical condition for static loading, $F_s = \frac{P_{allow}}{P_{static}} = \frac{4,560}{1,600} = 2.85$; therefore, care was exercised in connecting the chains to the frame through adjacent corners and in lifting the quadrants.

The safe working chain load was $T = 8D^2 = 8(3/8)^2 = 1.125$ tons, or 2,250 lbs., there D = diameter of one side of the chain link in inches (31). The equation employs a F_s of 4.0; a factor of 3.0 would give an allowable stress of 3,000 lbs. This is close to $P_w = 3,390$ lbs. which was also computed with a design factor of 3.0.

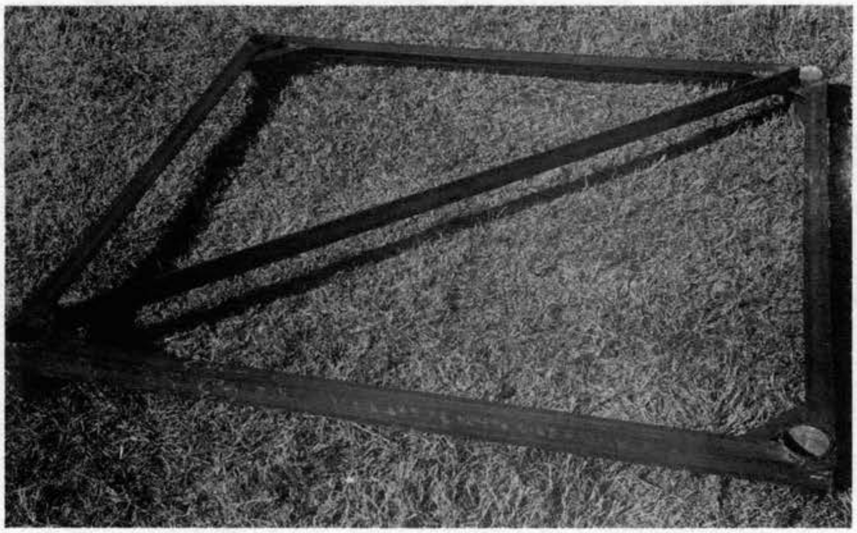


Figure 20. Lifting Frame.

Figure 21. Erected Column Showing Cribbing Clamps.

Column Supports for Foundation Casting

A special arrangement of cribbing was used to maintain the vertical position of the column during the time that the footings were cast and cured. This cribbing, Figure 21, consisted of 4 in. by 4 in. members, 36 in. long, clamped to the column in both directions to support the column weight. Additional vertical support was provided by 2 in. by 12 in. members clamped vertically to the sides of the column and butted against the bottom of the haunch at the column face.

The maximum stress in the cribbing was produced by a cantilever moment when each end of the cribbing in one direction supported the entire weight of the column. One-fourth of the column weight, 2,200 lbs., supported by each member, or 550 lb. per member, produced a calculated load on the end applied over a distance of 12 inches. The moment, $M = P \times L = 550 \times 12 = 6{,}600$ in.-lbs. and the shear force, $V = 550$ lbs. The allowable extreme fiber stress for Douglas fir (framing and joint grade) was 1,200 psi; the allowable shear stress perpendicular to the grain was 325 psi (26). Thus, $f = \frac{V}{A} = \frac{550}{13.14} = 41.85$ psi which is less than the 325 psi allowable. The section modulus required was $S = \frac{M}{f} = \frac{6{,}600 \text{ in.-lb.}}{1{,}200 \text{ psi}} = 5.5 \text{ in.}^3$ which was less than the value of 7.94 in.^3 for a 4 in. by 4 in. member.

CHAPTER VI

ERECTION PROCEDURE

The erection of the h-p shell from prefabricated elements required that five separate construction phases be integrated into a continuous operation. These phases consisted of site layout, column and tie erection, assembly of the support system, shell assembly, and final shear connections.

To make the study as realistic as possible, the construction of the quadrants and columns, the column and tie erection, and the assembly of the structure were carried out by an untrained crew with one of the departmental staff members acting as general contractor or foreman. The author was available for coordination with the foreman on construction procedures and plans, but did not actively supervise. In general, the entire construction phase was carried out as if this was a general contractor's crew, unfamiliar with the construction of an h-p shell.

The following paragraphs discuss the methods used to carry out each phase of the erection procedure. It should be noted that some of these steps were carried out concurrently as would be done on a prototype construction project.

Site Layout

The building site was on the Animal Husbandry farm, one and one-half miles west of the Agricultural Engineering Laboratory. The site was cleared and leveled by personnel from the Animal Science Department. After completion of this study, the structure was to be used as a machinery storage shelter.

The project foreman coordinated with the farm superintendent on the approximate location and the general orientation of the structure, and discussed the movement of the fences. Then, a two-man team surveyed the topography of the site and staked the principle building points. Figure 22 indicates the general layout of the structure, elevation points, and column locations. The site layout and construction staking required $3\frac{1}{2}$ hours for the two-man crew.

The foreman contracted a rotary drilling truck and operator to dig the foundation holes; 20 inch diameter holes were required, but the maximum bit size on the drill rig was 16 inches in diameter. The two holes were drilled to approximately 54 inches depth in one hour. The necessary reaming from 16 to 20 inches plus the excavations for the wing walls required two additional hours for a three-man crew. After the footing excavations were completed, the holes were covered to keep out moisture until the columns were erected.

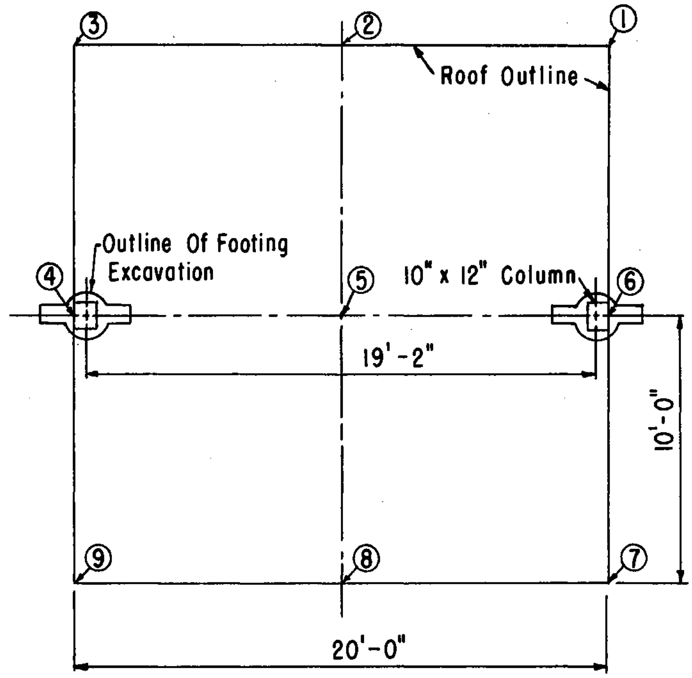


Figure 22. Worksite Layout Showing Construction Control Points.

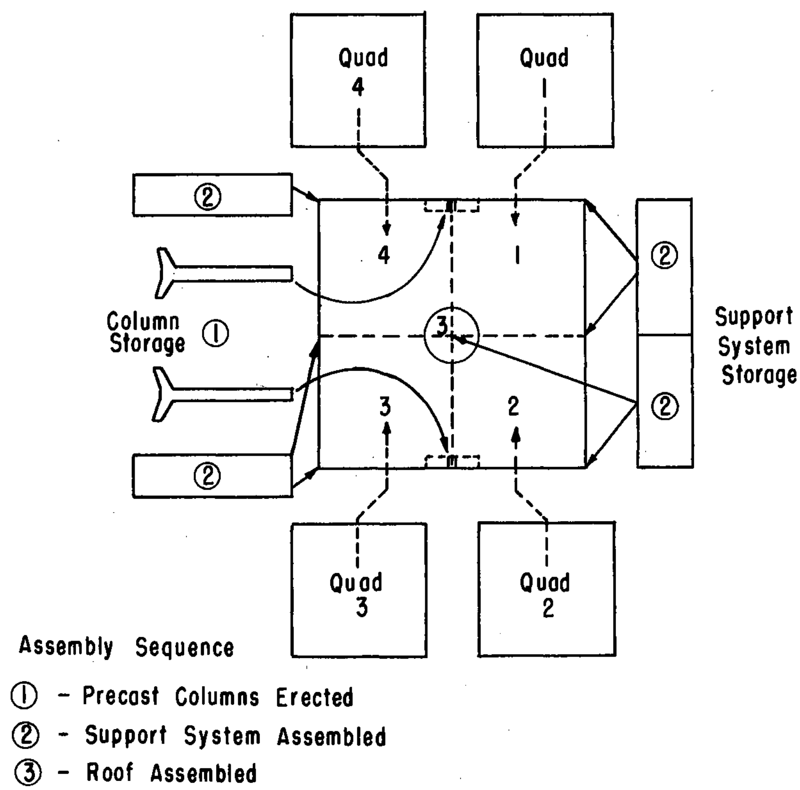


Figure 23. Material Layout for Assembly.

The columns were cured and erected in the forms to prevent damage to the concrete during lifting and moving. Both of the columns were loaded on a three-ton equipment trailer with a fork truck for transportation to the construction site. A three-man crew moved the columns to the site in one hour. The columns were stored at the site as indicated in Figure 23.

Because of the sequence for precasting the columns and shells, and constructing the assembly supports, all of the materials were not completely laid out at the worksite at the same time; however, this plan of material location was followed as closely as possible. The storage area indicated in Figure 23 for the rigid frame and temporary wooden supports was not utilized due to space limitations on the east side of the construction site. The north, south, and west sides were relatively unrestricted for locating and moving construction equipment and materials.

The shell quadrants were moved to the site after the columns and tie had been erected. Each quadrant was located as indicated in Figure 23. The assembly jig and wooden supports were moved to the site and assembled in their approximate locations.

No provision was made at the worksite layout for the parking or storing of major items of construction equipment. For this study, only one modular unit was erected, thus heavy equipment was required for short periods of time.

Column Erection

The column forms were used in the erection procedure in this study. Ordinarily, these forms would be removed by the second or third day for re-use in casting more column units. Two clamps, each made from two 21 inch double-threaded bolts and two 2 in. by 4 in. members were used to hold the side members against the column, Figure 8. The upper ends of these two members were placed against the lower surface of the haunches at the column face and were utilized as vertical supports during erection.

When the footing steel had been placed in the wing wall excavations, the airport crane lifted the column by a chain around the haunch arms, Figure 24. The crane lowered the column through the footing steel cage into the footing excavation until it was at the desired elevation. A target elevation had been marked on the column face five feet below the column top. By using the transit height of instrument reading from a temporary bench mark (used for the initial topographic survey), the exact elevation was determined.

The vertical support members on the sides of the column had been cut off to rest on the cribbing for column support. The outside face of the south column was aligned with the corner stakes and centered between them. The column was then plumbed in both directions when final elevation changes were made. Brace boards were placed in the

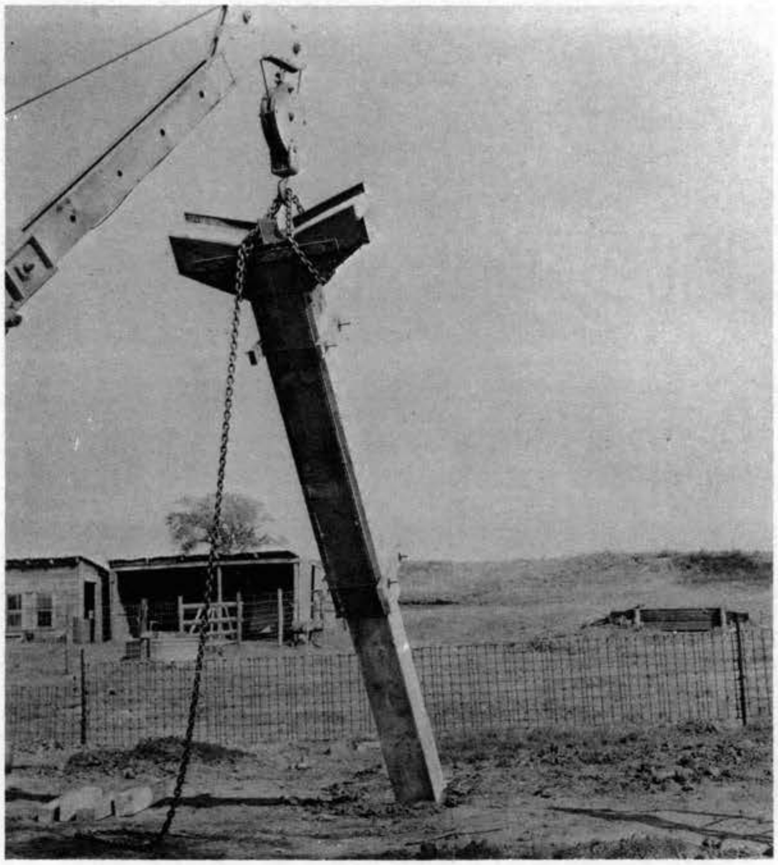


Figure 24. Method of Lifting Precast Column During Erection.

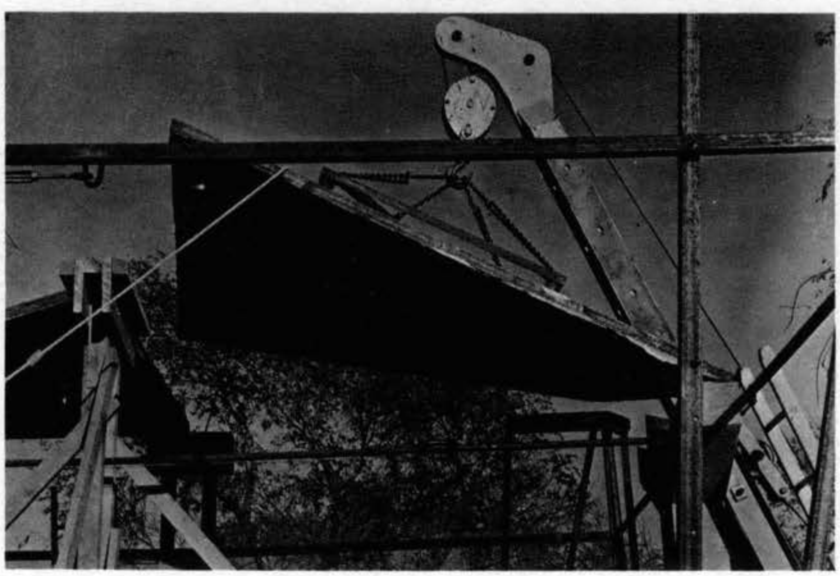


Figure 25. Second Quadrant Being Lowered During Assembly.

four principle directions and fastened securely before the lift chain was released.

The second column was lowered into the footing excavation and the cribbing was fastened in place. The distance between the outside column faces and the relative elevation of the second column was checked by transit. The column was then aligned between the corner stakes, set at the proper elevation and plumbed. Because of the dimensions of the precast elements, the distances between the outside column faces at the top of the column was 19 feet 11 inches.

When the second column was correctly aligned and plumbed, it was braced rigidly, Figure 21, and the crane support was released. The time required by a four-man crew plus crane operator to erect the columns, from the time the first column was ready for lifting until the column footings were ready for casting, was 2 hours and 50 minutes.

The footing concrete was delivered to the site by a ready-mix truck. The concrete was placed by a two-man crew and required approximately 30 minutes of working time. The footings were difficult to rod because of the small amount of clearance between column reinforcing steel and side of the excavation. A 22 or 24 inch diameter footing would have been easier to place, especially with an electric vibrator. No special curing procedures were used because of the small amount of surface area. The braces, cribbing, and vertical supports were removed after two days.

Tie Bar Connection

The tie bar was cut to a length of 18 feet 8 inches to allow a 4 inch overlap on each "T" section. The bar was laid in place and clamped to the "T" section at each end. The welder spotwelded the bar securely to one tie plate, then the bar was raised slightly at midspan and was spotwelded to the tie plate on the opposite column. The bar was then welded securely to the column tie sections on each column.

Support System

The assembly jig was assembled in its approximate location when it was moved to the building site. The corner towers were tilted up into position by the three-man assembly crew after the screw jack footings were inserted into the pipe sleeves. The towers needed no adjustment at the vertical lap joints, because the ground elevations did not vary by more than 8 inches. The horizontal braces had been previously marked according to their tower connection and frame position; these members were bolted to the towers and connected at the slotted overlap joint at midspans. The tower elevations were adjusted roughly for ease in connecting the horizontal braces. The tower assembly, not including final alignment, was completed by hand by a three-man untrained crew in three hours.

During the final tower alignment, the corner supports were centered on the columns, then the four tower caps were adjusted to 20 feet 2 inch horizontal spacings for assembling the quadrants. The lower horizontal frame members on the north and south sides were clamped to the concrete columns with 12 inch "C" clamps to prevent movement of the frame under an unsymmetrical load. Final adjustments were made in the corner elevations by checking the tower cap height with a survey rod and raising or lowering the towers by the screw jack legs to the correct heights.

The wooden end supports were placed in position at points 2 and 8 in Figure 22. Each end support was connected to the top and bottom horizontal frame members. This stabilized the support until the roof quadrant was lowered onto it. Small elevation changes were made by the use of shims beneath the base and by wooden wedges.

The center support was positioned over the tie bar, then the metal braces were fitted onto the removable 8 inch leg section. The vertical height was set by adjusting the two screwjacks at the base and checking the elevation with the transit and survey rod. Then, the support was centered horizontally and diagonal braces were set in the four principle directions, Figure 25. The time required for the alignment of the support system by a four-man crew was 2 hours and 45 minutes.

Shell Assembly

The procedure used during the shell assembly was to erect the quadrants in a pattern that would keep the support system stable during all phases of the assembly process. This requirement was met by erecting the quadrants in the sequence illustrated in Figure 23. The first and second quadrants were placed in positions adjacent to the column tie so that the low corner of each quadrant rested against the inverted "T" section which formed the column tie connecter; these column elements formed an effective guide during the assembly. The two adjacent quadrants rested against each other along the horizontal interior edge beam. The same procedure was carried out with the third and fourth quadrants so that the entire erection took place by rotating the crane's position in a clockwise direction around the structure to minimize crane movement.

The assembly crew consisted of the construction foreman, the crane operator, and three workmen. When lifting the quadrants, two workmen used tag lines to guide the quadrant into position on the frame. The movements of the crane were supervised by the foreman.

When the first quadrant was test lifted into position, the lower corner of the quadrant did not fit well on top of the column; close observation revealed that the quadrant was resting on the point of the shell corner. It was also noted that the tie rod would keep the third and fourth quadrants from fitting against the vertical web of the tie connection. The first quadrant was lowered and a portion of the corner was removed. The lower corners of the remaining three quadrants were also corrected in the same manner.

The first quadrant was lifted into position and the interior edge beam was visually aligned along the tie bar as it was lowered. The second quadrant was lowered into position in the same manner. Approximately 3/4 inch separated the two quadrants along the horizontal interior edge beam after they were initially set in place. The third and fourth quadrants were lowered onto the support system with their sloping edge beams against the first and second quadrants. The total assembly time required to connect the lifting frame to all quadrants and set them in position was one hour and twenty minutes. Figure 26 shows the shell immediately after assembly and prior to welding.

Welded Shear Connections

The quadrants had to be adjusted vertically and pulled together before the edge beam plates could be welded. After the quadrants were adjusted at the column top, a heavy weld was made connecting the sloped edge of the inverted "T" section to the edge beam angles, which fit directly against it. This weld on each end served to tie the shells solidly to the column top.

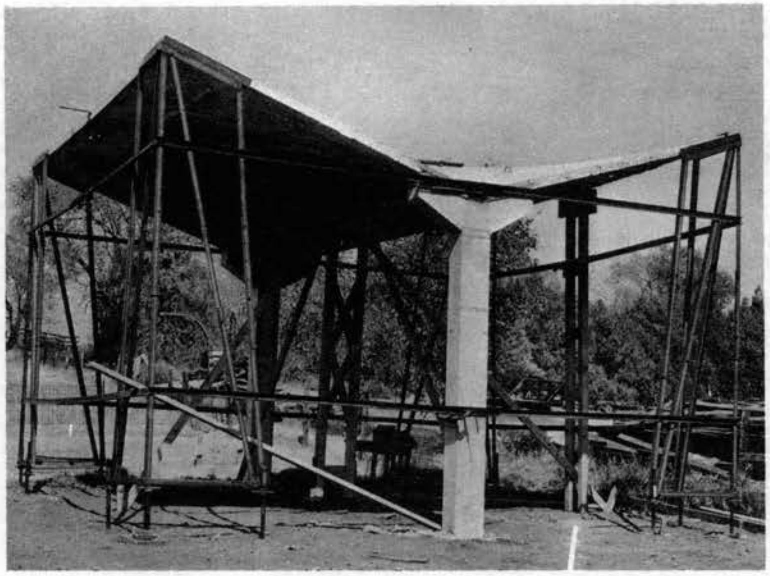


Figure 26. Support System Holding Shell Quadrants for Welding.

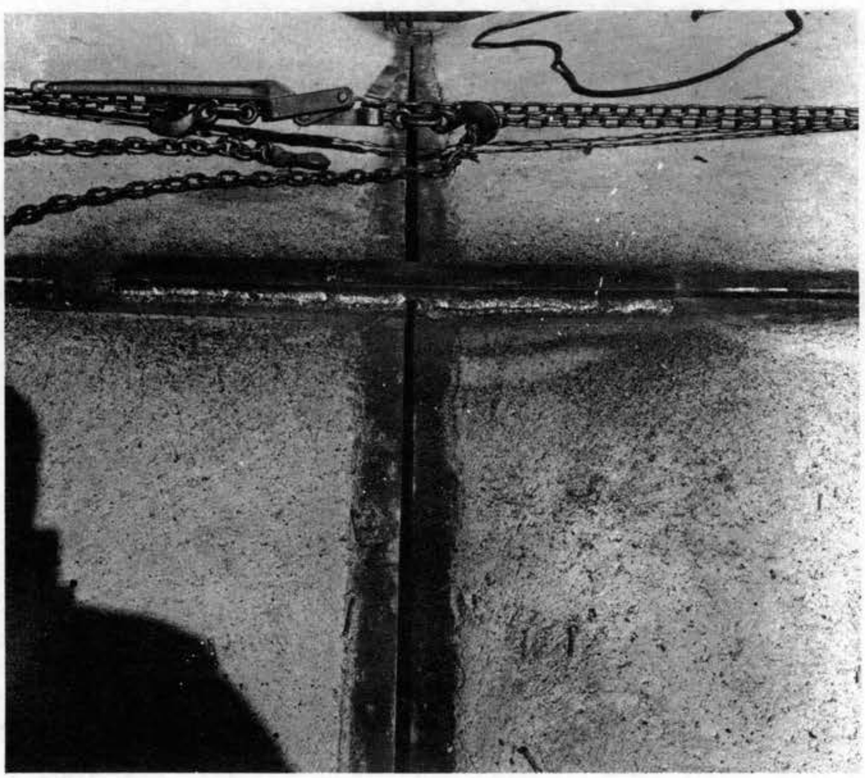


Figure 27. Roof Center Showing Method of Pulling Quadrants Together.

Initial closing along the horizontal interior edge beam was done by lowering the center and end supports. Instead of using the turnbuckles on the assembly jig to pull the quadrants together, a chain was connected between the two lifting rings, parallel to the horizontal edge beam and load binders were used to pull the sloped edge beams together, Figure 27. Because of the lack of complete uniformity in casting, the quadrants did not match at the center of the roof. The maximum desired allowance for misalignment and spacing between edge beams was one-half inch. This was the maximum that actually occurred due to warping of the edge beam angles when the dowels were welded to the edge beam, and due to the spacing induced by the web of the inverted "T" section on the column. As soon as the quadrants were bound together, the angle on the haunch was welded to the precast 23 inch angle in the lower corner of the shell. Two of the angles in the shell quadrants did not fit up against the haunch angle so a one-fourth inch steel bar was used as a filler and welded to the two angles.

Next, the 24 inch tension bar was centered over the intersection of the four quadrants and was welded in place. The 3/8 in. x 1 1/2 in. bars were then centered over the edge beam angles on the four edge beams and welded. A 1/4 in. to 3/8 in. leg fillet weld was used throughout the edge beam welding except on the center tension bar. The total time required by the welder to complete the shear and

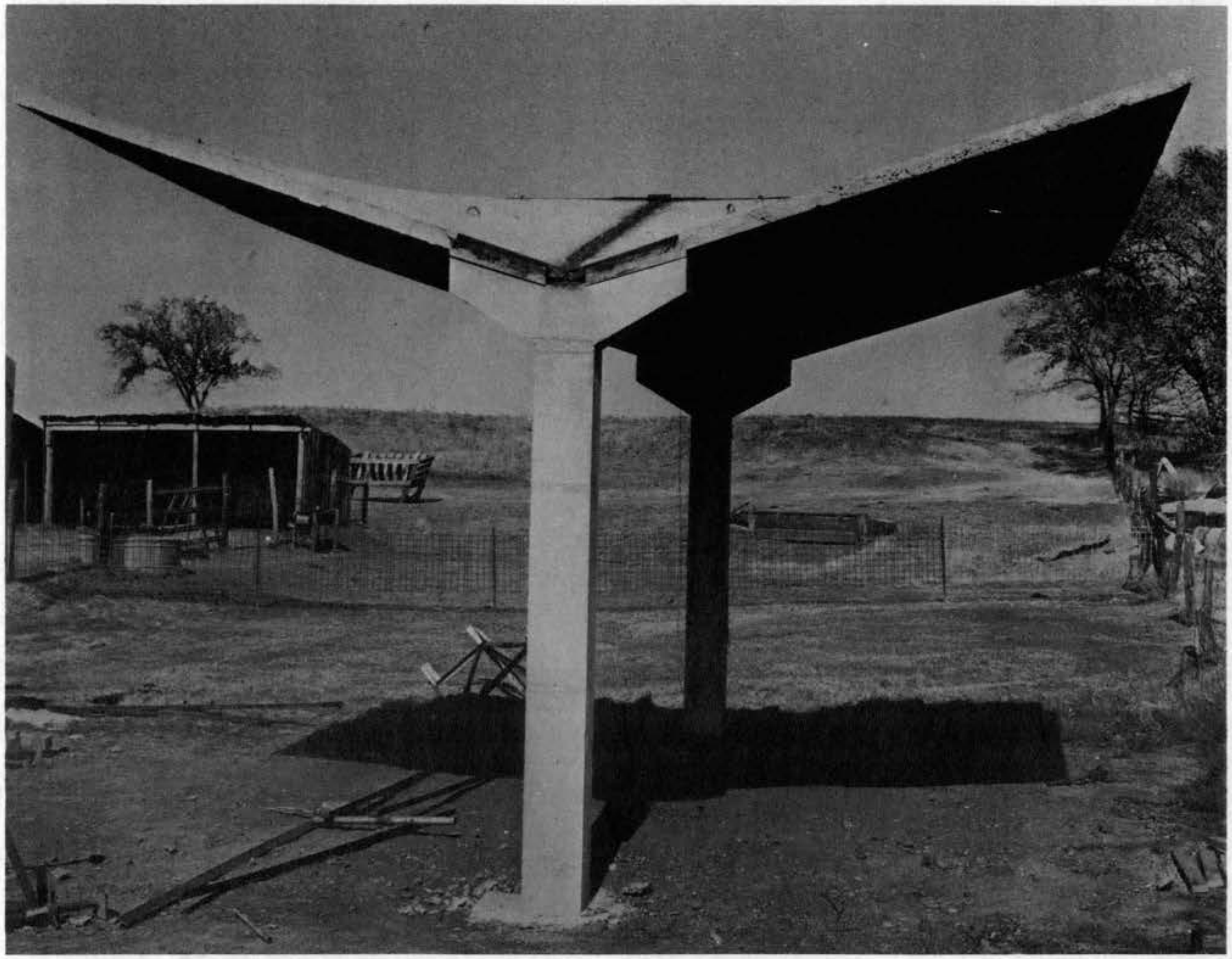


Figure 28. Completed Structure.

moment connections was 15 hours. As soon as the final welds were finished, the shell was structurally complete and the support system was removed, Figure 28.

CHAPTER VII

TESTING APPARATUS AND PROCEDURES

This phase of the study included experiments to analyze the properties of the structural elements, and tests of the structure under two types of static loads. First, the testing of the tie bar material and the concrete samples will be discussed. Then, the procedures and equipment used for load tests on the structure will be explained.

Tie Bar Calibration Tests

To determine the modulus of elasticity of the tie bar, two bar samples were tested in tension in the Riehle 100,000 Pound Testing Machine located in the Agricultural Engineering Laboratory.

A section at the center of each bar was ground down and smoothed on a belt sander. Two gage locations were marked 180 degrees apart near the center of the bar. By using an accelerator with the cement, the gages were bonded and ready for testing in approximately three minutes.

A Baldwin strain indicator and 10 channel Baldwin switching and balancing unit were used to indicate the strain. A temperature compensating gage placed on a bar sample was used to complete the external portion of the Wheatstone bridge.

Load was applied in increments of approximately 2,000 pounds. At each load increment, the scale was balanced and the load and strain readings were recorded simultaneously. The maximum loads placed on the two samples were 33,690 pounds and 35,230 pounds.

The recorded loads were converted to stress values by dividing each load by the cross section area of the bar. A plot of stress versus strain was made by regression analysis to determine the slope of the curve, which was the modulus of elasticity. An average value of the modulus of elasticity of $E_s = 30.48 \times 10^6$ psi was obtained. The observed data for the tie bar samples testing is in Appendix A.

Concrete Test Samples

Samples of the standard concrete in the precast columns and the lightweight aggregate concrete in the shell were taken during casting. These were cured under the burlap material with the columns and shells.

Three column test samples were cast in 3 inch diameter molds, 6 inches deep. These samples were tested in the Riehle 100,00 Pound Testing Machine after 14 days of curing time to determine the strength of the column concrete.

The average 14 day ultimate strength of the test cylinders was 3,086 psi. From the strength of these samples, the modulus of elasticity of the standard concrete was determined. The ACI recommendation for the modulus of elasticity for concrete was $E_c = 1{,}000\ f'_c$, providing the concrete was moist cured for 28 days (25). To adjust the results of the 14 day test, Figure 7 and Figure 9 from <u>Design and Control of Concrete Mixes</u> (32) were used. The adjusted 28 day ultimate strength of the column concrete, moist cured for 10 days, then air cured, was $f'_c = \frac{3{,}086\ \text{psi}}{(.90)} \times .95 = 3{,}260$ psi. Thus, $E_c = 1{,}000\ f'_c = 3.26 \times 10^6$ psi was used.

Three lightweight aggregate concrete samples were taken during the casting of the first, second, and fourth quadrants. The samples were removed from the mold after the first day and continued to cure under the plastic shell covering until the moist curing was completed at the end of 14 days. The three cylinders were tested at 21 days to check the strength of the quadrants for removal from the forms and movement to the site. The average 21 day strength of the samples was $f'_c = 4{,}480$ psi.

Structural Testing

<u>Tie Bar Testing</u>

The tie bars were tested by mounting two sets of foil strain gages approximately 3 feet from each end of the bar.

The bar was ground down to approximately $1\frac{1}{4}$ inches diameter and smoothed to present a uniform gage mounting surface. The gages were mounted 180 degrees apart longitudinally in the same manner that was used on the test samples. The tie bar was welded between the columns so that the gages were vertically opposite. A temperature compensating gage was mounted on $1\frac{1}{4}$ inch diameter steel bars which were located on the tie bar near each set of gages.

Tension Bar Testing

The steel bar connecting the four quadrants at the center of the roof was tested for tensile stresses by a strain gage centered on the bar over the edge beam gage. A temperature compensating gage was placed on a 6 inch length of the same material. During load testing, the gages were covered by a galvanized steel box formed to fit the roof slope and bar protrusion. It was bolted down to the concrete by $\frac{3}{8}$ inch diameter nail-set bolts. The lead wires were protected by a flexible conduit connected to the side of the box and extended to the edge of the roof.

Column and Haunch Testing

To determine the magnitudes of strains induced into the column and haunches under varied loading conditions, two gages were placed on each haunch arm along the centerline of the bottom side, one gage was centered 4 inches directly beneath the haunch on each side of the column, and one gage was centered approximately 6 inches from the base on each of the four sides, Figure 29. Six inch paper backed gages were used. Two compensating gages were mounted on a 10 inch concrete cube, cast to simulate the column dimensions; each compensating gage served the active gages on one column.

Strain Gage Equipment

The strain gage testing equipment consisted of a Baldwin strain indicator, one 20 channel, and one 10 channel Baldwin switching and balancing unit. The twenty gages mounted on concrete were connected to the 20-channel unit and the five gages mounted on steel were connected to the 10-channel unit.

The strain gage equipment was placed in a small wooden building approximately 12 feet from the south column. This building protected the instruments and did not interfere with the loading of test material onto the structure. The maximum lead wire distance was limited to approximately 50 feet while the shortest lead wires were approximately 25 feet long.

Deflection Apparatus

A manometer type deflection device was constructed to measure the vertical roof deflection at seven points and the relative vertical movement of the two columns. The reservoir of the manometer was a large coffee urn,

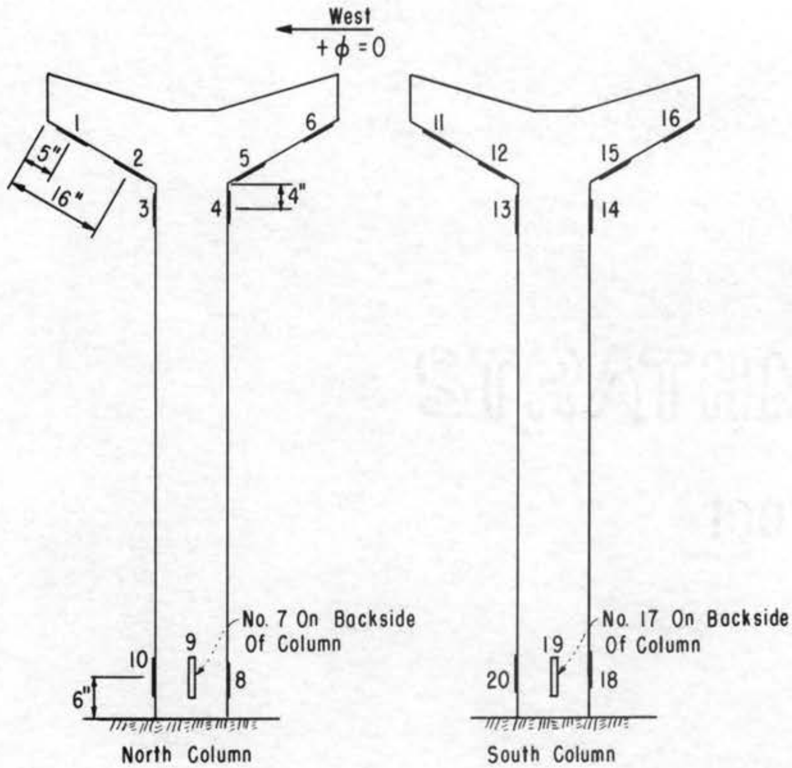


Figure 29. Positions of Strain Gages on Columns and Haunches.

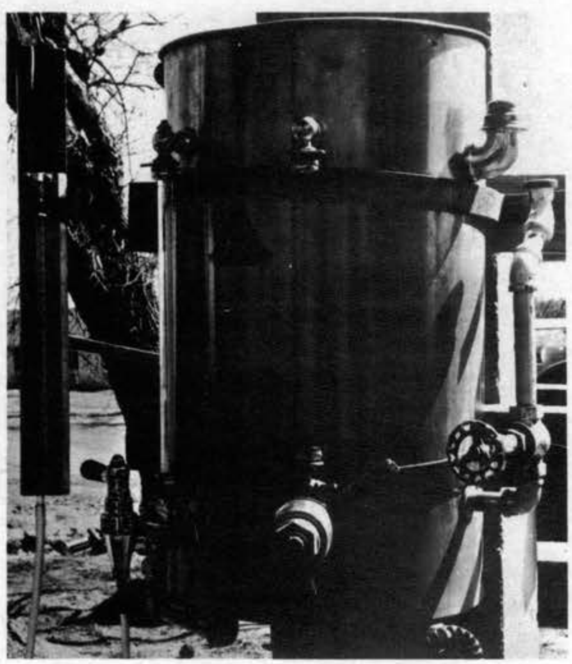


Figure 30. Manometer Deflection Apparatus Showing Reservoir and Moveable Section.

Figure 30. The urn was clamped to the inner face of the south column so that all movement of the structure could be related to one point.

A datum line was scribed around the glass water level tube on the urn. A 10 inch section from a scale with 50 divisions per inch was mounted vertically against the tube. A 3/8 inch outside diameter plastic tube was attached to the spigot directly below the water level tube to connect the manometer reservoir to the movable end of the manometer. The movable section of the manometer consisted of a glass tube clamped to a 1 in. x 2 in. x 18 in. board, a 50th scale which was attached to the board behind the glass tube, the 3/8 inch plastic connector tube, and a 1/4 inch steel rod approximately 7 feet long, Figure 30. The bottom end of the steel rod was rigidly attached to the top end of the board. The top end of the rod was formed into a ring to use in suspending the manometer board from hooks which were clamped to the edge of the shell.

The manometer was open to the atmosphere on both ends so that no pressure differences were developed. The manometer reservoir was filled with approximately 3 1/2 gallons of a water and alcohol mixture to prevent freezing during cold weather. During the testing period, water level readings were taken each day from the datum line on the water level tube to correct for evaporation. Readings were taken on the movable end of the manometer at all 8 points before and after each roof load change.

Testing Procedure

The structural load-testing consisted of three uniform load tests and one eccentric load test. Each test included one or more load increments.

The procedure used for each load increment was:

1. Zero the strain gages at 1,000 on the indicator.
2. Record the initial datum reading on the water level tube of the manometer.
3. Record the zero reading on the north column and at each of the four corners of the roof, at midspans of each horizontal edge, and the center of the roof.
4. Place the roof load increment on the shell by loading alternate quadrants on each side of the column tie, and spreading the load material uniformly.
5. Take depth measurements at 13 points on each quadrant to obtain an average depth. Average the four quadrant depths to obtain the average roof depth.
6. Take density samples during the loading in 12 inch square pans which were 2 inches, 4 inches, and 6 inches deep. Fill each pan in the same manner that the rest of the roof was loaded.

7. Weigh the density samples and average the densities to obtain an average density for the roof load.
8. Record the strain readings for all 25 gages after the instrument has warmed up for approximately 5 minutes.
9. Record vertical roof deflection data.

This sequence was repeated throughout the testing period with the exceptions of steps 6 and 7. The density of the gravel was checked periodically during each test phase, especially after a change of weather conditions.

To load the structure, a three-point hookup tractor slip mounted on the lift arms of a fork lift truck was used. The operator filled the slip by driving it into a gravel pile, raising the slip above the roof height, and dumping the load. The load material, consisting of 3/8 inch unwashed chat, was held on the roof by 8 inch depth wooden forms.

The initial loading phase consisted of a sustained uniform roof load of 21.6 lbs./ft.^{2} of horizontal projection. This load was placed on in one increment and served as a preliminary load to settle the structure. Deflection and strain readings were taken during a 72 hour load period, then the shell was unloaded and the final zero readings were taken.

The second load condition was a uniform roof load with increments of 25.0, 22.0, and 14.7 lbs./ft.^{2} to give a total load of 61.7 lbs./ft.^{2}. The maximum load which was approximately 1.5 times the design load, remained on the shell 114 hours.

The third load consisted of an eccentric roof load of 41.3 lbs./ft.^{2} on one-half of the roof surface which caused a cantilever load centered 5.0 feet from the column tie. This load, which was approximately design load, was placed on the roof in increments of 25.0 and 16.3 lbs./ft.^{2} and remained on the structure for approximately 45 minutes while the readings were taken.

The fourth load consisted of a uniform total load of 57.0 lbs./ft.^{2} placed on in progressive increments of 19.3, 16.1, 13.6, and 8 lbs./ft.^{2}; the duration of load for this test was 46 hours.

CHAPTER VIII

PRESENTATION AND ANALYSIS OF DATA

The data from this construction engineering study will be analyzed in two categories: (1) Analysis of construction costs, and (2) Analysis of load test data. The results of these analyses will be discussed in Chapter IX.

Analysis of Construction Costs

The analysis of construction costs will be divided into three sections: (1) labor costs, (2) material costs, and (3) equipment costs. The observed data from this study are valid only for this project. A set of skill or experience factors will be discussed in Chapter IX. These may be used to estimate actual construction labor costs by adjusting the observed data.

The labor wage scales were estimates from, Estimating Construction Costs (33), Table 1-2, "Union Wage Scale In The United States, In Dollars." This table lists an estimated average rate, and a range in rates. The average rate for building laborers, $2.18 per hour, will be used for unskilled labor costs, and a rate of $3.13 per hour, which is the average rate for carpenters, will be used for skilled labor and supervision.

Other costs for material or equipment were either actual costs incurred or estimates obtained from local sources.

Labor Costs

The labor costs will be tabulated for each phase of construction for unskilled labor and skilled labor (or supervision) on a man-hour basis. The final cost for labor will be computed from the total man-hours.

TABLE I

LABOR COSTS

Item	Skilled	Unskilled
1. Column Construction		
(a) Forms	2	40
(b) Steel Forming	5	54
(c) Casting and Curing	4	12
SUBTOTAL (Man-hours)	11	106
2. Shell Construction		
(a) Forms	16	96
(b) Shell Steel Forming	8	78
(c) Form Preparation and Casting	4	16
(d) Curing	3	10
SUBTOTAL (Man-hours)	31	200

TABLE I (Continued)

Item	Skilled	Unskilled
3. Support System Construction		
(a) Cutting Out Parts	1	7
(b) Welding Tower Frames	36	0
(c) Assembly of Bolted Components	0	3
(d) Wooden Supports	2	16
(e) Adjustments on Steel Frame	1	4
SUBTOTAL (Man-hours)	40	30
4. Lift Frame Construction	6	0
5. Site Preparation		
(a) Leveling and Smoothing	0	2
(b) Survey and Layout	2	2
(c) Foundation Excavation	2	9
SUBTOTAL (Man-hours)	4	13
6. Site Layout		
(a) Hauling Columns and Placing	2	2
(b) Construction of Shell Supports	2	4
(c) Removing Forms, Loading, and Transporting Shells to Site	4	7
(d) Moving Support System	1	4
SUBTOTAL (Man-hours)	9	17
7. Column Erection		
(a) Development of Column Support System for Stabilizing Column	4	25
(b) Cutting and Bending Footing Steel	0	4
(c) Column Erection and Plumbing	4	14
(d) Casting Column Footings	0	1
(e) Removal of Braces and Site Cleanup	0	4

TABLE I (Continued)

Item	Skilled	Unskilled
7. (Continued)		
(f) Welding Tie	2	2
SUBTOTAL (Man-hours)	10	50
8. Support System Erection		
(a) Initial Erection of Corner Towers	3	9
(b) Final Alignment of Towers for Shell Erection	4	12
SUBTOTAL (Man-hours)	7	21
9. Erection of Structure		
(a) Initial Assembly	4	12
(b) Preparation for Welding	2	4
(c) Welding Edge Beams and Column to Shell Connections	15	0
(d) Support Removal and Site Cleanup	0	6
(e) Grouting Top of Columns	0	4
(f) Waterproofing Interior Edge Beams	1	5
(g) Final Cleanup	0	4
SUBTOTAL (Man-hours)	22	35
FINAL TOTAL (Man-hours)	140	472

The total cost for labor was Labor Cost = 140($3.13) + 472($2.18) = 438.20 + 1,028.96 = $1,467.16.

Equipment Costs

Equipment charges were made for all equipment used, whether rented or obtained from the Agricultural Engineering Laboratory. Labor was included with rental equipment charges on the crane, tractor dozer, and rotary drill rig; all other equipment was laboratory property or operated by departmental personnel. Labor charges were shown in Labor Costs. Table II shows the types of equipment used and the number of equipment-hours for the specific jobs. The local electric welder rates varied from $2.00 in the shop to $3.00 for portable welders.

TABLE II

EQUIPMENT COSTS

Item	Hours	Cost
1. Acetylene Welder (Labor and material separate)		
(a) Heating and Bending Column Steel	13	
(b) Heating and Bending Shell Steel	6	
(c) Tower Support Frame	7	
(d) Footing Steel Cage	1	
(e) Lifting Frame	1	
SUBTOTAL (At $3.00/hr.)	28	$84.00
2. Electric Welder		
(a) Column Steel Forming, $2.00/hr.	3	6.00
(b) Tie Bar and Column Tie Plate, $3.00/hr.	2	6.00
(c) Shell Steel Forming		
(1) Welding dowels to edge beam, $2.00/hr.	10	20.00
(2) Corner reinforcing mat, $2.00/hr.	4	8.00

TABLE II (Continued)

Item	Hours	Cost
(3) Final welding on shell and edge beams, \$3.00/hr.	5	\$15.00
(d) Rigid Frame Supports, \$2.00/hr.	33	66.00
(e) Footing Steel Cage, \$2.00/hr.	1	2.00
(f) Lifting Frame, \$2.00/hr.	2	4.00
(g) Portable Welding on Edge Beams During Shell Erection, \$3.00/hr.	15	45.00
SUBTOTAL		\$172.00
3. Tractor and Equipment Trailer, \$2.50/hr.	9	22.50
4. Tractor With Drawbar Hoist, \$2.00/hr.	2	4.00
5. Fork Truck, 10 Ton Capacity, \$3.00/hr.	2	6.00
6. Crane, 10 Ton Capacity (With operator), \$6.00/hr.	8	48.00
7. Tractor Dozer for Site Leveling (With Operator), \$6.00/hr.	2	12.00
8. Rotary Drill Truck (With Operator), \$12.50/hr.	1	12.50
9. Power Hacksaw, 20 cuts per hour, \$0.10/ct, or \$2.00/hr.	9	18.00
FINAL TOTAL		\$379.00

Material Costs

The cost of materials was separated from labor and equipment to provide a clear outline of the expenditures charged to each part of the project. The material costs are listed in Table III.

TABLE III

MATERIAL COSTS

Item	Quantity	Cost
1. Welding Materials		
(a) Welding Rod	75 lbs., $.20/lb.	$15.00
(b) Acetylene	1-100 cu. ft. bottle	5.70
(c) Oxygen	1-224 cu. ft. bottle	5.65
SUBTOTAL		$26.35
2. Concrete		
(a) Standard Weight, 3,000 psi	3 cu. yd., $14.75	44.25
(b) Lightweight Aggregate, 3,750 psi	$3\frac{1}{2}$ cu. yd., $18.25	63.87
SUBTOTAL		$108.12
3. Steel Material		
(a) Assembly Support System		
(1) Steel	1,540 lbs., $.097/lb.	149.69
(2) Jacking Screws	12, $9.62 each	115.44
(3) Turnbuckles	4, $3.20 each	12.80
(4) Bolts and Pipe		14.10
(b) Lifting Frame		
(1) Steel	99 lbs., $.097/lb.	9.62
(2) Pipe	16 in., $.25/ft.	.33
(3) Bolts	4 - $\frac{1}{4}$ in. x 4 in., $.10	.40
(c) Shell and Column Steel	2,056 lbs., $.097/lb.	199.84
(d) Jacking Screws, Center Support		19.74
SUBTOTAL		$521.96
4. Lumber and Miscellaneous		
(a) Shell Forms		170.39
(b) Column Forms		24.15
(c) Assembly Supports		30.62
SUBTOTAL		$225.16
FINAL MATERIAL COST TOTAL		$881.59

From Tables I, II, and III, the total combined costs for labor, equipment, and material was determined. The total initial cost of precasting the 20 foot square h-p shell was $2,727.75. This would be an initial cost of $6.82 per square foot of horizontal projection, for one use of forms and erection apparatus.

Of the total cost, 53.8 per cent was for labor, 32.3 per cent for material, and 13.9 per cent of the total was charged to equipment. The multiple use of forms and erection equipment, and a discussion of the cost of constructing a 40 foot square prototype will be discussed in Chapter IX.

Analysis of Load Test Data

The analysis of the data from the load tests compares theoretical computations with the observed data from the structural tests. The data consists of observed strain and deflection readings recorded during uniform and eccentric roof load tests. The strain data were readings from strain gages on the steel tie bar and the horizontal interior edge beam at the roof center, and strain gages mounted on the columns. The deflection data were differential elevation readings taken at seven roof points and on the north column.

Although three uniform load tests were run, Table IV, only the strain data from Test IV are analyzed. The eccentric load data from Test III are also analyzed. The strain data are analyzed first, then the deflection data are presented.

TABLE IV

STRUCTURAL TESTS APPLIED TO SHELL

Test No.	Type of Loading	Maximum Load	Time Duration
I	Uniformly Distributed Gravity Load, 1 Load Increment	21.6 psf	73 1/2 hrs.
II	Uniformly Distributed Gravity Load, 3 Load Increments	61.7 psf	117 1/2 hrs.
III	Half-roof Eccentric Load, Uniformly Distributed, 2 Load Increments	41.3 psf	3 hrs.
IV	Uniformly Distributed Gravity Load, 4 Load Increments	57.0 psf	74 1/2 hrs.

The strain values of gages 1-20 for Test III were not adjusted due to the residual strain which remained in the structure after the test was completed. Strain values for gages 21-25 for Test III and gages 1-25 for Test IV were adjusted by using a ratio of the time of reading against the total time of the test and adjusting the final zero load values.

Properties of the Column Section

The properties of the column in both directions, $\varphi = 0$ and $\varphi = 90°$, Figure 4, were determined for the analysis. The values that were determined for each direction were (1) the column width, b, (2) the distance from the center of tension steel to the extreme compression face of the column, d, (3) the depth of column section, t, (4) the location of the neutral axis, N.A., which is the distance, kd, from the extreme compression fiber, (5) the distance from the extreme compression fiber to the center of the resultant compressive force, z, (6) the distance between the resultant tensile and compression forces in bending, jd, and (7) the moment of inertia of the transformed section, I_t.

The column section properties were analyzed for both the cracked and the uncracked sections, and the values are listed in tabular form in Table V.

The modulus of elasticity of the concrete was adjusted from the 28 day value, E_c, because of the influence of creep strain. The sustained modulus of elasticity, $E_{ct} = \frac{1 \text{ psi}}{\delta_t + \Delta_t}$, where:

δ_t = Axial creep strain (specific creep), the time-dependent unit creep strain of concrete per psi of sustained axial stress, in millionths.

Δ_t = Axial elastic strain = $\frac{1 \text{ psi}}{E_c}$.

From formula (10-1), (34), $\delta_t = c_1(t)^{1/r}$, where:

c_1 = A coefficient determined by tests, expressed in millionths, the first days creep strain under a stress of unity $= \frac{0.500}{(a)^{0.40}}$.

r = A root deduced from tests.

t = Time, the duration of the loading, in days.

a = Age when loaded, in days.

TABLE V

PROPERTIES OF COLUMN SECTIONS

Property	Values			
	Uncracked Section		Cracked Section	
	$\varphi = 0$	$\varphi = 90°$	$\varphi = 0$	$\varphi = 90°$
(1) b	10.0 in.	12.0 in.	10.0 in.	12.0 in.
(2) d (or d_{ave})	9.5 in.	6.25 in.	9.5 in.	6.25 in.
(3) t	12.0 in.	10.0 in.	12.0 in.	10.0 in.
(4) kd	6.0 in.	5.0 in.	4.15 in.	3.45 in.
(5) z	2.14 in.	1.81 in.	1.69 in.	1.35 in.
(6) jd	7.36 in.	4.40 in.	7.81 in.	4.90 in.
(7) I_t	1957.0 in.^{4}	1176.0 in.^{4}	921.6 in.^{4}	410.2 in.^{4}

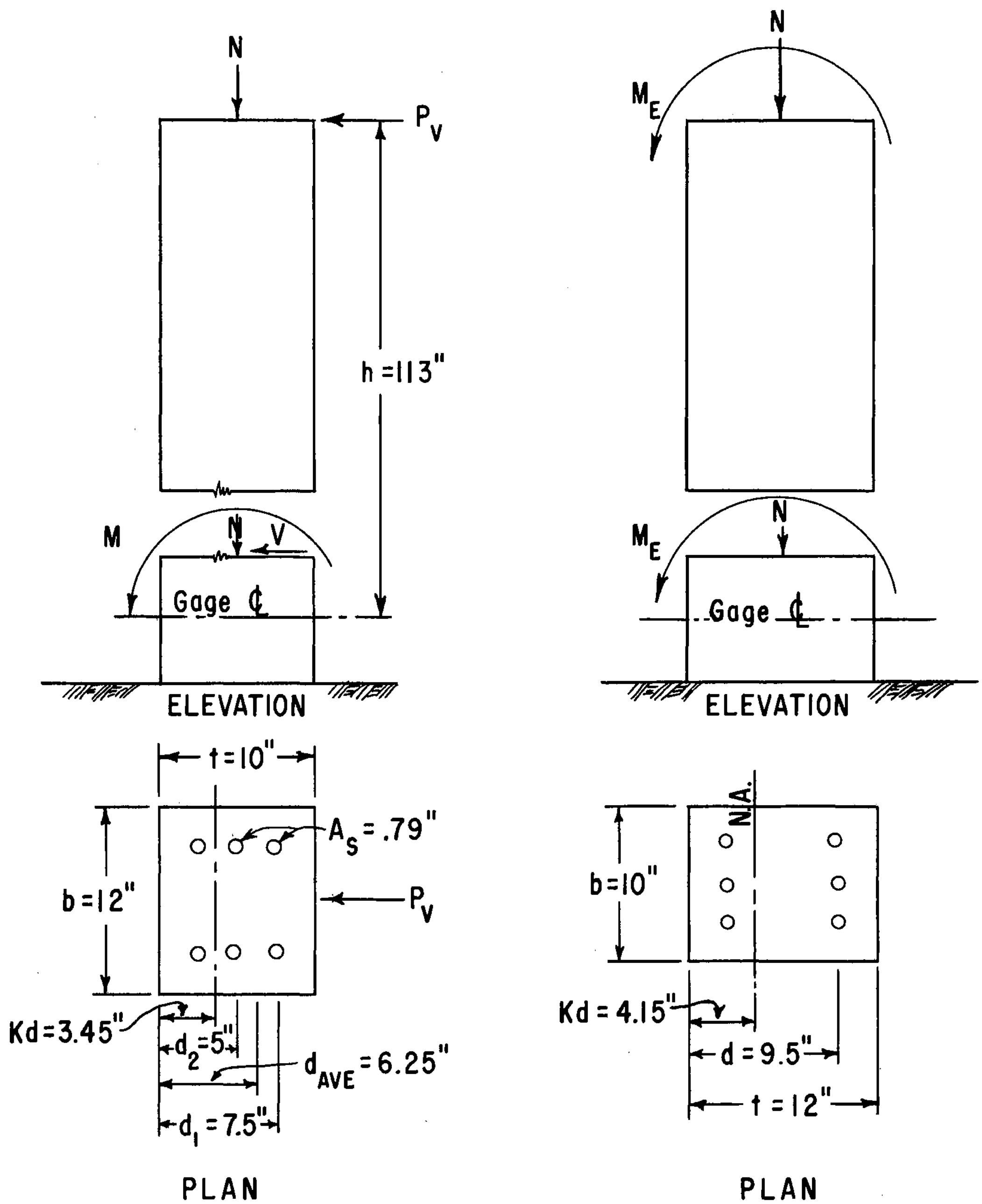
For Test III and Test IV, which were loaded on successive

TABLE VI

Test IV Strain and Stress Data

Load	19.3 psf		35.4 psf		49.0 psf		57.0 psf					
Time	$1\frac{1}{2}$ hrs.		$2\frac{1}{4}$ hrs.		$3\frac{1}{4}$ hrs.		$4\frac{1}{2}$ hrs.		20 hrs.		50 hrs.	
Gage No.	ϵ	σ	ϵ	σ	ϵ	σ	ϵ	σ	ϵ	σ	ϵ	σ
1	+12	+33.4	+13	+36.2	+24	+66.7	+42	+116.8	+18	+48.1	+15	+39.1
2	+2	+5.6	-6	-16.7	+5	+13.9	+28	+77.9	+4	+10.7	0	0
3	+10	+27.8	+20	+55.6	+41	+114.0	+81	+225.0	+84	+224.1	+50	+130.5
4	+27	+75.0	+24	+66.7	+45	+125.1	+28	+77.9	+2	+5.3	+10	+26.1
5	+7	+19.5	+3	+8.4	+14	+38.0	+27	+75.0	-2	-5.3	-5	-13.1
6	+12	+33.4	+3	+8.4	+14	+38.0	+26	+72.3	+9	+24.0	0	0
7	+25	+69.5	+27	+75.0	+40	+111.0	+55	+153.0	+37	+98.9	+60	+156.8
8	+32	+89.0	+24	+66.7	+45	+125.1	+28	+77.9	+22	+58.7	0	0
9	+8	+22.2	+14	+38.0	+25	+69.5	+34	+94.5	+16	+42.7	+50	+130.5
10	-6	-16.7	+14	+38.0	+8	+22.2	+57	+158.5	+52	+138.8	+120	+312.5
11	+11	+30.6	+12	+33.4	+13	+36.1	+24	+66.7	-12	-32.0	-15	-39.1
12	+11	+30.6	+7	+19.5	+13	+36.1	+24	+66.7	-22	-58.7	-25	-65.3
13	+3	+8.3	+18	+50.0	+17	+47.4	+45	+125.1	+5	+13.1	-30	-78.4
14	+22	+61.1	+23	+64.0	+45	+125.1	+37	+103.0	0	0	+10	+26.1
15	+12	+33.4	+3	+8.4	+14	+38.0	+17	+47.2	-12	-32.0	-5	-13.1
16	+7	+19.5	+3	+8.4	+14	+38.0	+17	+47.2	-2	-5.3	+5	+13.1
17	0	0	0	0	-10	-27.8	0	0	-20	-53.5	-40	-104.5
18	+16	+44.5	+9	+25.0	+22	+61.1	+3	+8.4	+118	+315.0	0	0
19	+22	+61.1	+53	+147.2	+54	+150.0	+67	+186.1	+133	+355.5	+120	+313.0
20	+4	+11.1	+37	+103.0	+27	+75.0	+56	+156.0	-4	-10.7	0	0
21	-35	-1067	-50	-1524	-65	-1982	-75	-2285	-62	-1890	-55	-1677
22	-160	-4880	-260	-7930	-360	-10,980	-422	-12,860	-445	-13,560	-405	-12,350
23	-110	-3350	-190	-5790	-270	-8240	-320	-9750	-320	-9750	-310	-9450
24	-130	-3960	-230	-7090	-310	-9450	-374	-11,400	-391	-11,920	-370	-11,280
25	-120	-3660	-210	-6440	-280	-8540	-340	-10,370	-358	-10,920	-340	-10,370

Remarks: (1) $\epsilon_c = 1 \times 10^{-6}$ in./in. (2) σ = psi. (3) $E_{ct(1)} = 2.78 \times 10^{6}$ psi at time, 0 through $4\frac{1}{2}$ hrs. (4) $E_{ct(2)} = 2.67 \times 10^{6}$ psi at time, 20 hrs. (5) $E_{ct(3)} = 2.61 \times 10^{6}$ psi at time, 50 hrs. (6) $E_s = 30.48 \times 10^{6}$ psi.



(a) Bending in $\emptyset = 90^{\circ}$ Direction (b) Bending in $\emptyset = 0$ Direction

Figure 31. Columns Showing Directions of Bending, Dimensions, and Reactions.

days, c_1 remained constant; $c_1 = 0.0533 \times 10^{-6}$. For Test III, $\delta_t = 0.0533 \times 10^{-6}$, $\Delta_t = 0.307 \times 10^{-6}$, and $E_{ct} = 2.78 \times 10^6$ psi. For Test IV, the values of $E_{ct(2)} = 2.67 \times 10^6$ psi, and $E_{ct(3)} = 2.61 \times 10^6$ psi.

Analysis of Load Strain Data

The axial load imposed upon each column was $N = 57.0 \times 200 = 11{,}400$ lbs. According to elastic theory, both the concrete and reinforcing steel would deform equally due to bond. Thus, $\Delta_t = e_s \cdot h = e_c \cdot h$, where h = the height of the column above the base gage centerline, Figure 31(a). From this, $\Delta_t = \frac{P_s \cdot h}{A_s \cdot E_s}$, and $P_s = \frac{P_T}{1+\frac{A_c}{n \cdot A_s}}$. The maximum axial load per column which was supported by the steel was $P_s = 3{,}180$ lbs. Thus, $\Delta_t = 2.48 \times 10^{-3}$ in. The axial deformation for each column was the average of the four strain readings at the base of each column. These values are tabulated in Table VI.

The axial strains for both columns are shown in Table VII for the entire load period during Test IV. The values for the bottom gages of each column are compared with the values of the other column. By comparing the values of axial strain for the north and south columns in Table VII, it is evident that the columns did not deform ideally. The differences in values between column (2) and column (3) in the table may be due to unequal settlement of the column footings. Thus, one of the columns resists

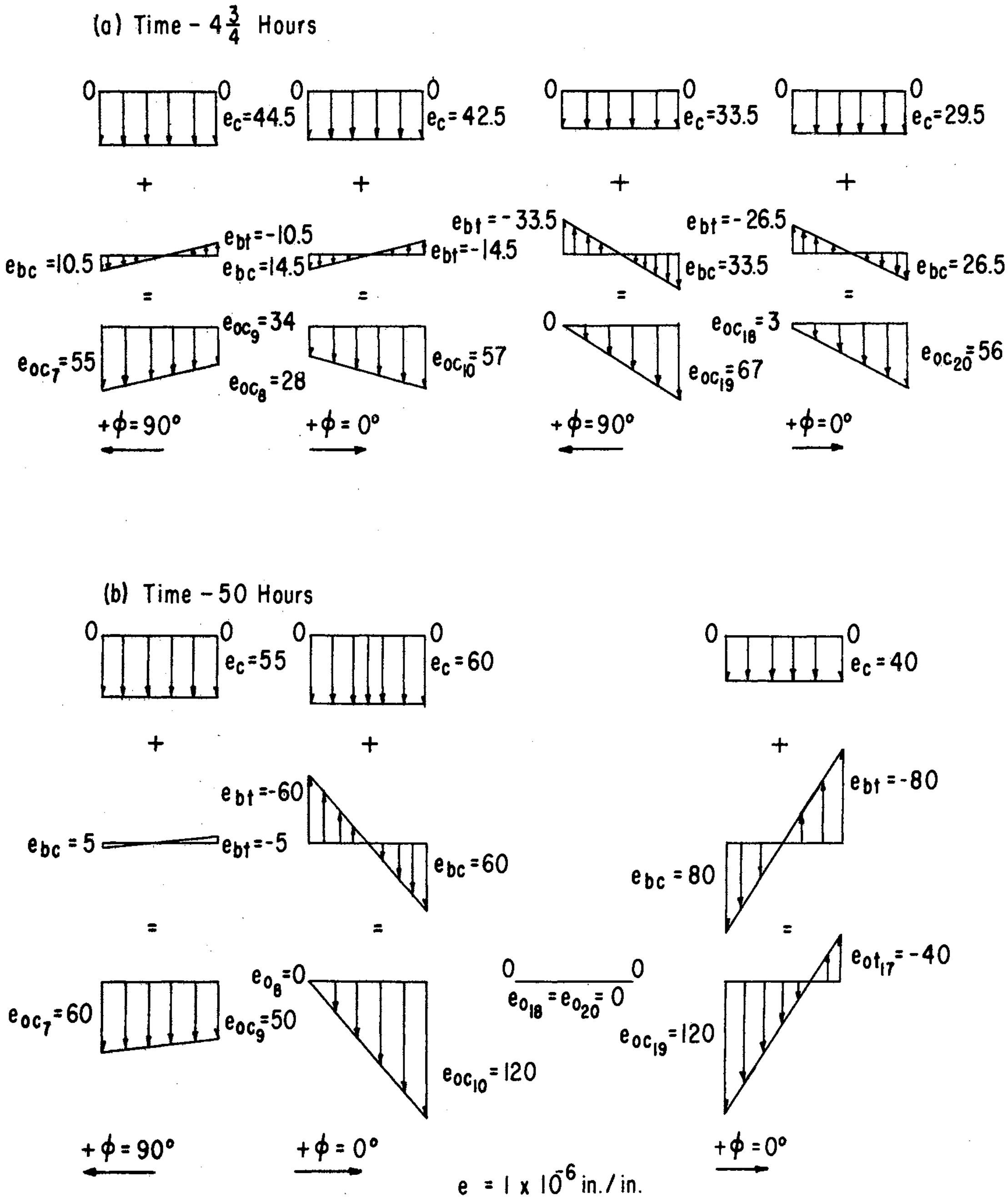
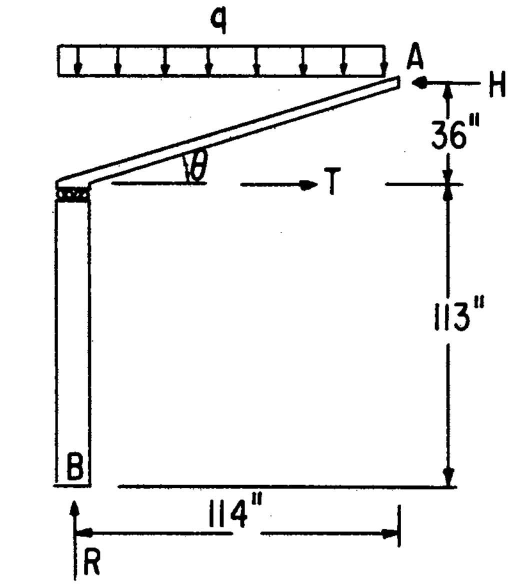
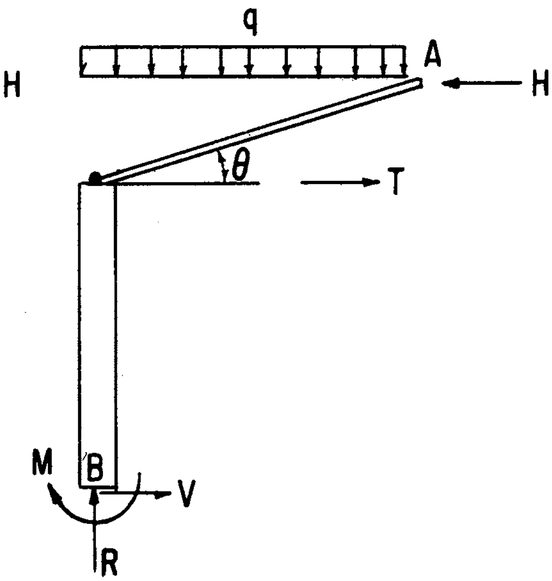
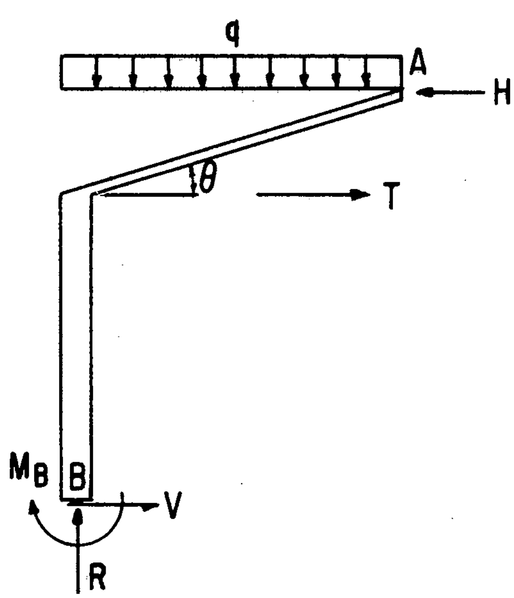


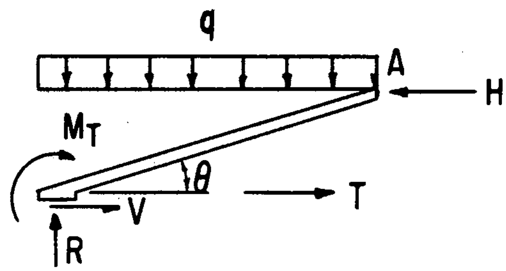
Figure 32. Axial and Bending Strain Distribution During Sustained Loading in Text IV.





(a) Condition 1., Table VIII (b) Condition 2., Table VIII





(c) Condition 3., Table VIII (d) Free Body at Base of Tie Connection

Figure 33. Free Body Diagrams of Theoretical Stress Conditions Acting on Column During Uniformly Distributed Roof Loading.

more of the load. The bending strains in both directions are shown in Figure 32 as an illustration of the change in loads resisted by each column. These plots show the variation in bending due to unequal settlement of the footings.

TABLE VII

AXIAL STRAIN UNDER UNIFORMLY DISTRIBUTED LOAD

Time (Load)	Strain (Micro-in./in.) North Column Base Gages (2)	South Column Base Gages (3)	Average, All Column Gages (4)	Theoretical Strain (5)
1 1/2 hours (19.3 psf)	14.75	10.50	13.58	8.96
2 1/4 hours (35.4 psf)	19.75	24.75	21.91	16.40
3 1/4 hours (49.0 psf)	29.50	23.25	29.92	22.70
4 1/2 hours (57.0 psf)	43.50	31.50	49.17	26.40
20 hours (57.0 psf)	31.75	56.50	37.00	27.50
50 hours (57.0 psf)	57.50	20.00	29.17	28.20

The values of strain in column (4) do not compare favorably with the theoretical strain values, column (5). This can be attributed to the fact that the footing on the prototype does not hold the column base rigid, thus, the footings deformed and the column could not resist the entire force in bending.

Table VIII summarizes the stress conditions of the columns under the uniformly distributed load during Test IV for several structural conditions, Figure 33. Condition 3 was the nearest to the actual conditions at the site. Condition 2 assumes pinned connections at the haunch and ridge. All of the values in Table VIII were determined by using the maximum values from the Test IV data.

TABLE VIII

STRESS CONDITIONS UNDER UNIFORMLY DISTRIBUTED GRAVITY LOAD FROM TEST IV DATA

Condition	Tie Bar Load (lbs.)	Shear at Column Base (lbs.)	Bending at Column Base (in.-lbs.)	Bending at Haunch (in.-lbs.)
1. Ideal Situation, Tie Bar Carries All Thrust, No Bending in Ridge, Haunch, or Column	19,000 (Calc.)	0	0	0
2. No Bending in Haunch or Ridge, Bending in Column	16,990 (Meas'd)	2,010 (Calc.)	227,000 (Calc.)	0
3. No Bending in Ridge, Bending in Haunch and Column	16,990 (Meas'd)	4,760 (Calc.)	21,900 (Meas'd)	520,000 (Calc.)
4. Bending in Ridge, Haunch, and Column	16,990 (Meas'd)	Indeterminate	21,900 (Meas'd)	Indeterminate

According to Portland Cement Association design procedure (24), the horizontal thrust acting at the top of the column was $P_h = 2 \cdot H \cdot a = \frac{2 \cdot w \cdot a^2 \cdot b}{2 \cdot h} = \frac{57.0\ (1{,}000)}{3} =$ 19,000 lbs., if no bending stress exists in the edge beams. This force is resisted by the columns in bending and by the tie bar. From Test IV, the average maximum stress in the tie bar under the 57.0 lb./ft.^{2} roof load was 11,540 psi, which produced a tensile force in the tie, $T = f_s \cdot A_s = 11{,}540 \times 1.47 = 16{,}990$ lbs. The shear resisted by the column at the top was $P_v = 19{,}000 - 16{,}990 = 2{,}010$ lbs. The calculated bending moment at the base gages' centerline due to shear at the top of the column was 2,010 x 113 = 227,130 in.-lbs. The moment derived from the observed strain data at the base gages' centerline was $M_b = \frac{f_c \cdot I_t}{c} = 21{,}900$ in.-lbs. for an uncracked section and 11,080 in.-lbs. for a cracked section.

Checking the north column base strain data for the second day of sustained loading, $e_b = \frac{e_7 - e_8}{2} = 10.5$ micro-in./in., which was the same reading obtained the previous day. For the south column, the observed bending strain, $e_b = \frac{e_{19} - e_{17}}{2} = 76.5$ micro-in./in. The theoretical value was $e_b = 48.1$ micro-in./in. for an uncracked section. By assuming the section was cracked, this value was $e_b = 33.2$ micro-in./in.

The values of bending strain during the third day were $e_b = \frac{e_7 - e_9}{2} = 5$ micro-in./in., and $e_b = \frac{e_{19} = e_{17}}{2} =$ 80 micro-in./in. for the north and south columns, respectively. The theoretical strain assuming first an uncracked section, then a cracked section, were e_b = 45.2 micro-in./in. and e_b = 31.2 micro-in./in.

A noticeable trend was developing during the sustained load period; this was indicated by the decrease in bending strain in the north column from 10.5 to 5.0 micro-in./in. and from 33.5 to 80 micro-in./in. in the south column. During this same time, the deflection data indicated a settlement of the south column of 0.04 in. between the first and second day readings.

Throughout the three-day period, gages 7 and 9 (Figure 29) on the north column indicated an increasing bending moment toward the $+\varphi = 0$ direction, while gages 17 and 19 on the south column indicated bending toward $+\varphi = 0$ on the first day but shifted to $-\varphi = 0$ on the second day and back to zero on the third day.

The average compressive strain values of the base gages on the north column during the two-day sustained load period were e_c = 43.5, 31.75, and 57.5 micro-in./in., while for the south column, e_c = 31.5, 56.75, and 20.0 micro-in./in. These values indicate a shift of the structural stresses.

The strain gage on the tension bar at the center of the horizontal interior edge beam did not develop the stresses for which the bar was designed. For the 57.0 lbs./ft.^{2} uniform load, the calculated tensile stress in the tension bar was 19,000 lbs. The maximum stress measured by the gage was 3,430 lbs. The remainder of the load, 15,570 lbs. was resisted in tension by the welded plates in the sloped edge beams on both sides of the horizontal edge beam.

Analysis of Eccentric Load Strain Data

The strain and stress data for Test III are shown in Table IX. The values of the moments calculated from the base gage strain data are tabulated in Table X. The calculated maximum overturning moment due to the eccentric roof load, Figure 31(b), assuming idealized conditions was M_o = 248,000 in.-lbs. for each column. Assuming an uncracked section, M_b = 360,000 in.-lbs. for the north column and M_b = 318,000 in.-lbs. for the south column. Assuming the section was cracked, M_b = 249,000 in.-lbs. for the north column and M_b = 220,000 in.-lbs. for the south column. A check of the $\frac{e'}{t}$ ratio indicates that the columns should be investigated for the cracked section condition. The comparison of the strain values against the theoretical value shows that the cracked section values check very closely with the idealized moment.

To compare the observed strain to the calculated values, the maximum bending strain in the direction of overturning for both columns was derived from the data. The values of actual bending strain were derived from the observed values, Figure 34, by the following relationships, Figure 35: (1) $e_{bc} = \frac{kd}{t-kd} \cdot e_{bt}$, (2) $e_{ot} = e_{bt} + e_{ac}$, and

TABLE IX

TEST III STRAIN AND STRESS DATA

Load	25.0 psf		41.3 psf		Remarks
Gage No.	ϵ	σ	ϵ	σ	
1	+30	+83.4	+20	+55.6	1. $\epsilon_c = 1 \times 10^{-6}$ in./in.
2	+60	+167.0	+65	+180.5	
3	+150	+417.0	+280	+778.0	
4	-70	-194.5	-420	-1168.0	2. σ= psi.
5	+10	+27.8	-10	-27.8	3. $E_{ct} = 2.78 \times 10^6$ psi.
6	0	0	-30	-83.4	
7	-25	-69.5	-185	-514.0	4. $E_s = 30.48 \times 10^6$ psi.
8	-250	-695.0	-550	-1529.0	
9	0	0	+40	+111.1	
10	+200	+556.0	+430	+1195.0	
11	-20	-55.6	+5	+13.9	
12	-10	-27.8	+5	+13.9	
13	+90	+250.0	+230	+639.0	
14	-120	-334.0	-350	-972.0	
15	-50	-139.0	-50	-139.0	
16	-40	-111.1	-50	-139.0	
17	-100	-278.0	-240	-666.0	
18	-230	-639.0	-470	-1308.0	
19	-40	-111.1	-90	-250.0	
20	-150	-417.0	+365	+1015.0	
21	-26	-792	-10	-304	
22	-78	-2378	-134	-4090	
23	-60	-1830	-110	-3358	
24	-64	-1950	-117	-3570	
25	-64	-1950	-117	-3570	

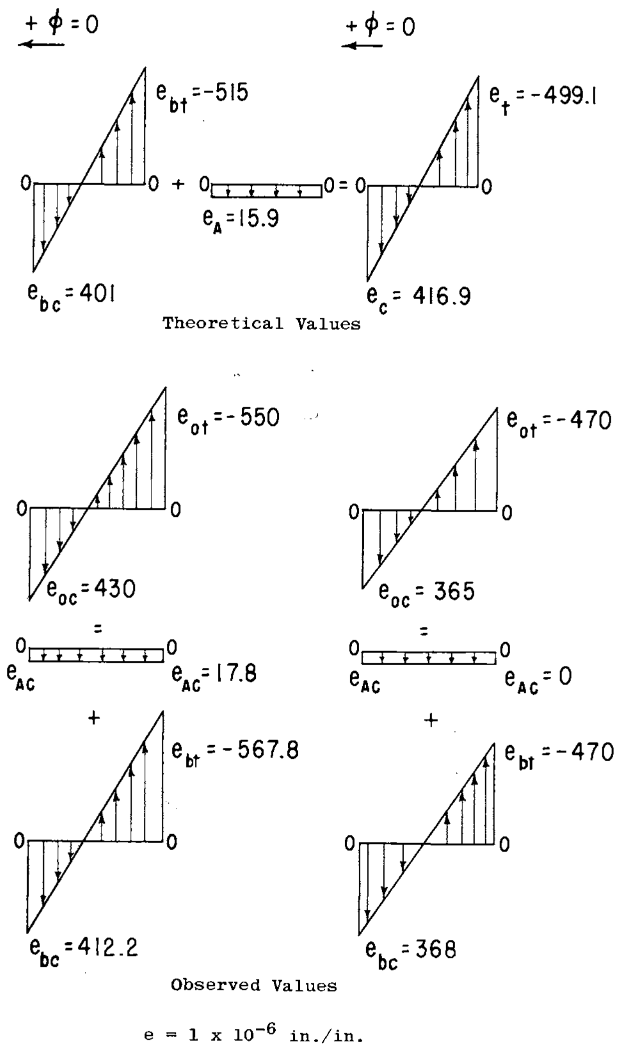
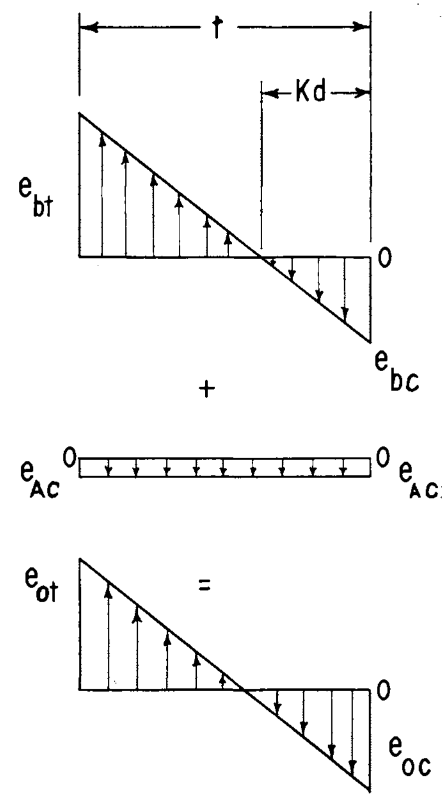


Figure 34. Axial and Bending Strains in Direction of Overturning During Test III.



$$\frac{e_{bt}}{t - Kd} = \frac{e_{bc}}{Kd}$$

or (1) $e_{bc} = \dfrac{Kd}{t - Kd} \times e_{bt}$

(2) $e_{ot} = e_{bt} + e_{AC}$

(3) $e_{oc} = e_{bc} + e_{AC}$

Figure 35. Definition Sketch for Heavy Bending Strain.

(3) $e_{oc} = e_{bc} + e_{ac}$,

where:

e_{bc} = compressive bending strain (unknown)

e_{bt} = tensile bending strain (unknown)

e_{ac} = compressive axial strain (unknown)

e_{oc} = observed compressive strain

e_{ot} = observed tensile strain.

TABLE X

SUMMARY OF MAXIMUM STRESS VALUES FROM TEST III

Location (φ = 0)	Type of Stress	Idealized Analysis	Experimental Results North Col.	South Col.
Top of Column	Shear (lbs.)	0	0	0
	Compression (lbs.)	4,130	27,400	21,100
	Moment (in.-lbs.)	248,000	142,200	122,200
Base of Column	Shear (lbs.)	0	1,520	1,520
	Compression (lbs.)	4,130	5,850	1,955
Note (2)	Moment (in.-lbs.)	248,000	360,000	318,000
Note (3)		248,000	249,000	220,000
Tie Bar	Tension (lbs.)	6,880	5,360	5,360

Notes:
(1) Maximum load = 41.3 psf.
(2) Assume an uncracked section.
(3) Assume a cracked section.
(4) See Figure 36 for location of N.A. for eccentric loading.

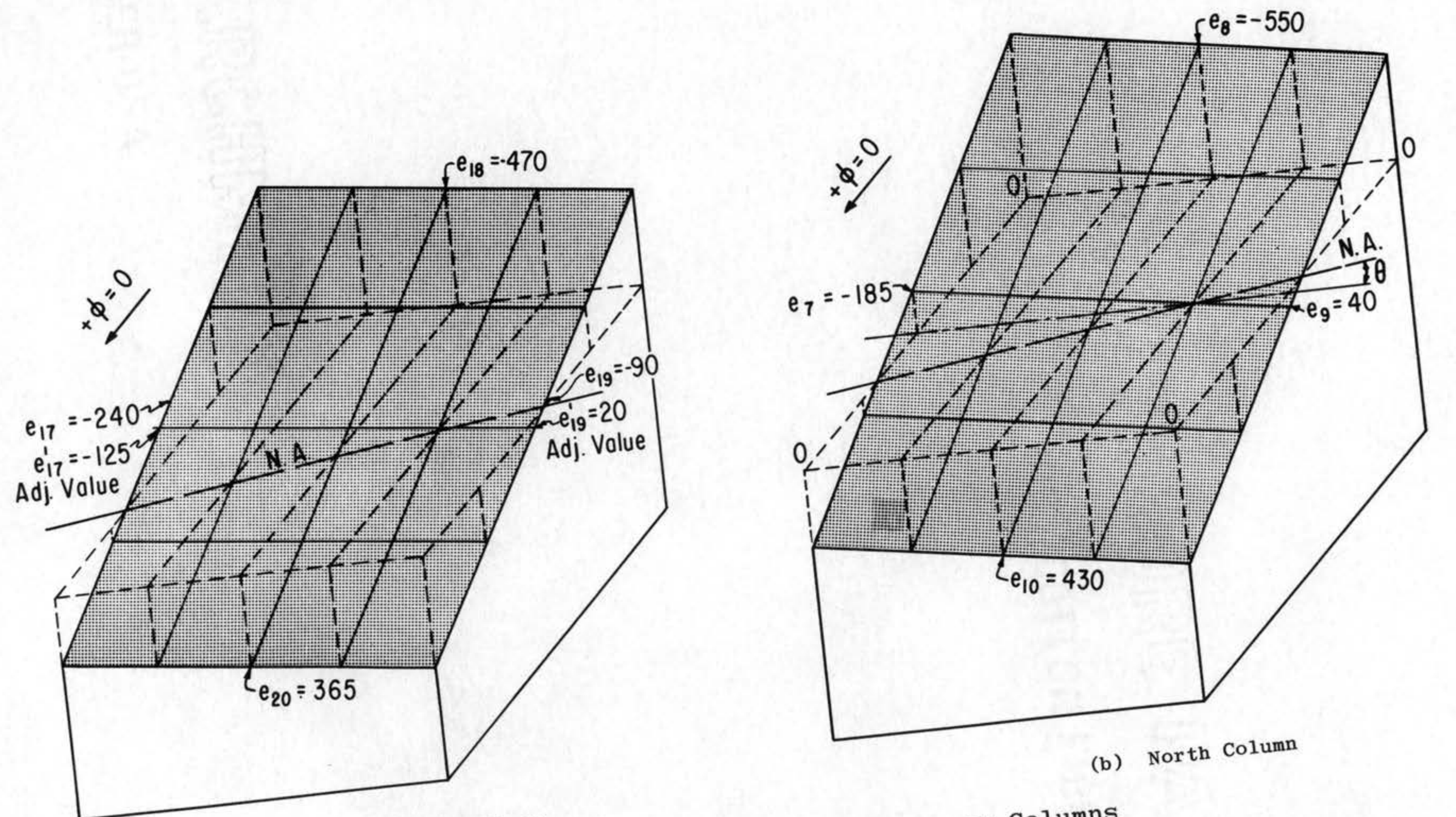
The maximum axial load per column during the eccentric loading was N = 4,130 lbs. This load was accompanied by a calculated horizontal thrust at the top of the column of $P = \frac{w \cdot a^2 \cdot b}{2 \cdot h} = \frac{41.3\ (1000)}{2 \times 3} = 6{,}880$ lbs., which was resisted by the tie bar, and by the column and haunch joint in bending. From the test data, the average stress in the tie bar was 5,360 lbs., which left 1,520 lbs. to be resisted by the column in shear and bending.

The orientation of the N.A., the axis along which strain is zero, Figure 36, shows the influence of the bending moments in the direction, $\varphi = 0$. If the tie bar resisted all of the horizontal thrust, the orientation of the N.A. would probably be in the $\varphi = 90°$ direction. Figure 36(a) shows that the N.A. has shifted far enough over from the column center to cause tension in gage 7. The south column, Figure 36(b) shows that tension existed in gages 17 and 19, which are on opposite faces of the column.

The complete strain relationship at the maximum eccentric load condition is illustrated by the three dimensional sketches in Figure 36. Because bending moments occurred in two directions, the resultant N.A. was located by plotting the known values of strain, which showed that the N.A. was skewed in the same direction for both columns.

The deflection data are presented in Figure 37 and Figure 38. The data for Test II were used to illustrate the uniform deformation of the roof. The deflections of the roof during Test IV were influenced by the residual strain remaining in the structure after Test III had been completed. The presence of strain is clearly apparent in the final unloaded condition at the conclusion of Test III, Figure 38(a). This may indicate that the column had exceeded the elastic limit during heavy bending under the eccentric loading and could not return to its normal state. Although residual strain was recorded by the column gages after unloading, part of the roof deformation may have been due to a slight yielding of the soil around the footings and wingwalls and tilting of the structure in the $+\varphi = 0$ direction.

The values of stress versus load and time for Test IV are shown in Figure 39. Values for Test III were not shown as there were only two load increments and no sustained loading.



(a) South Column

(b) North Column

Figure 36. Neutral Axis Locations in Columns Under Cantilever Loads.

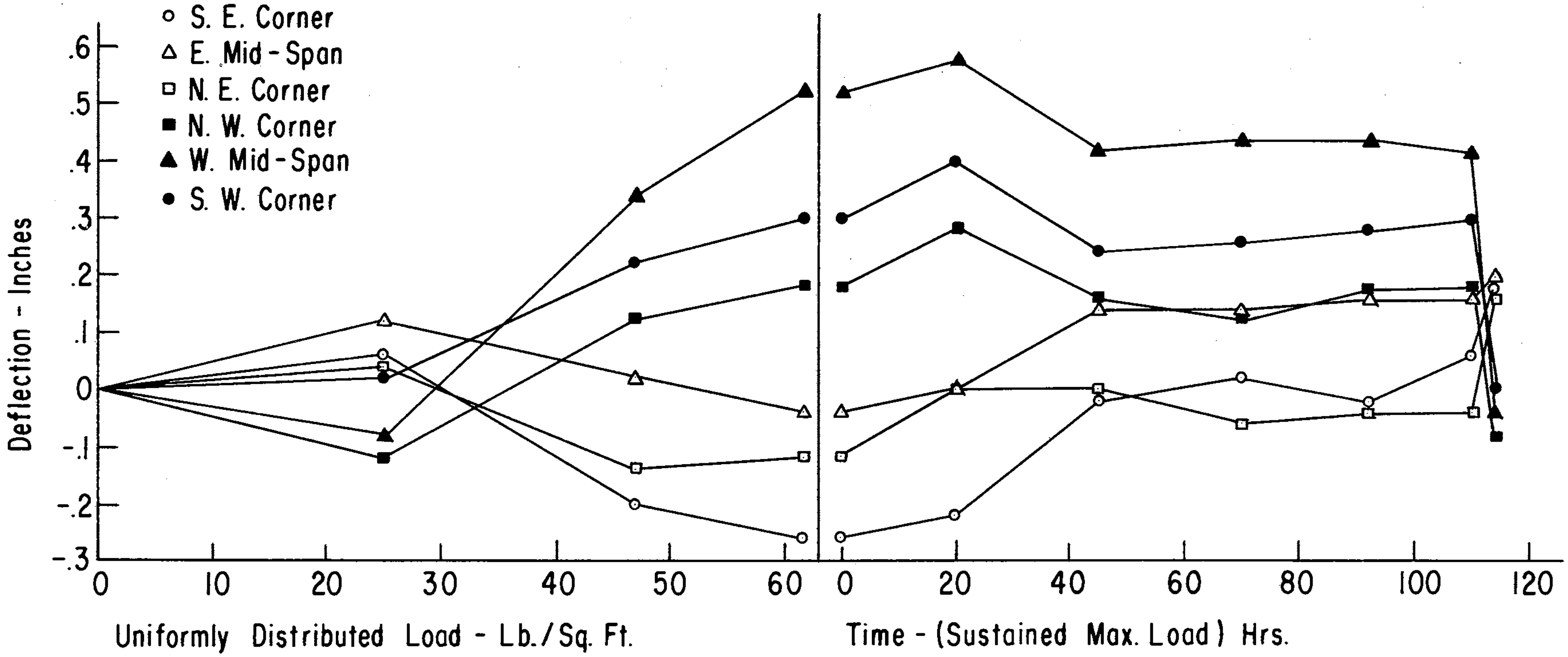
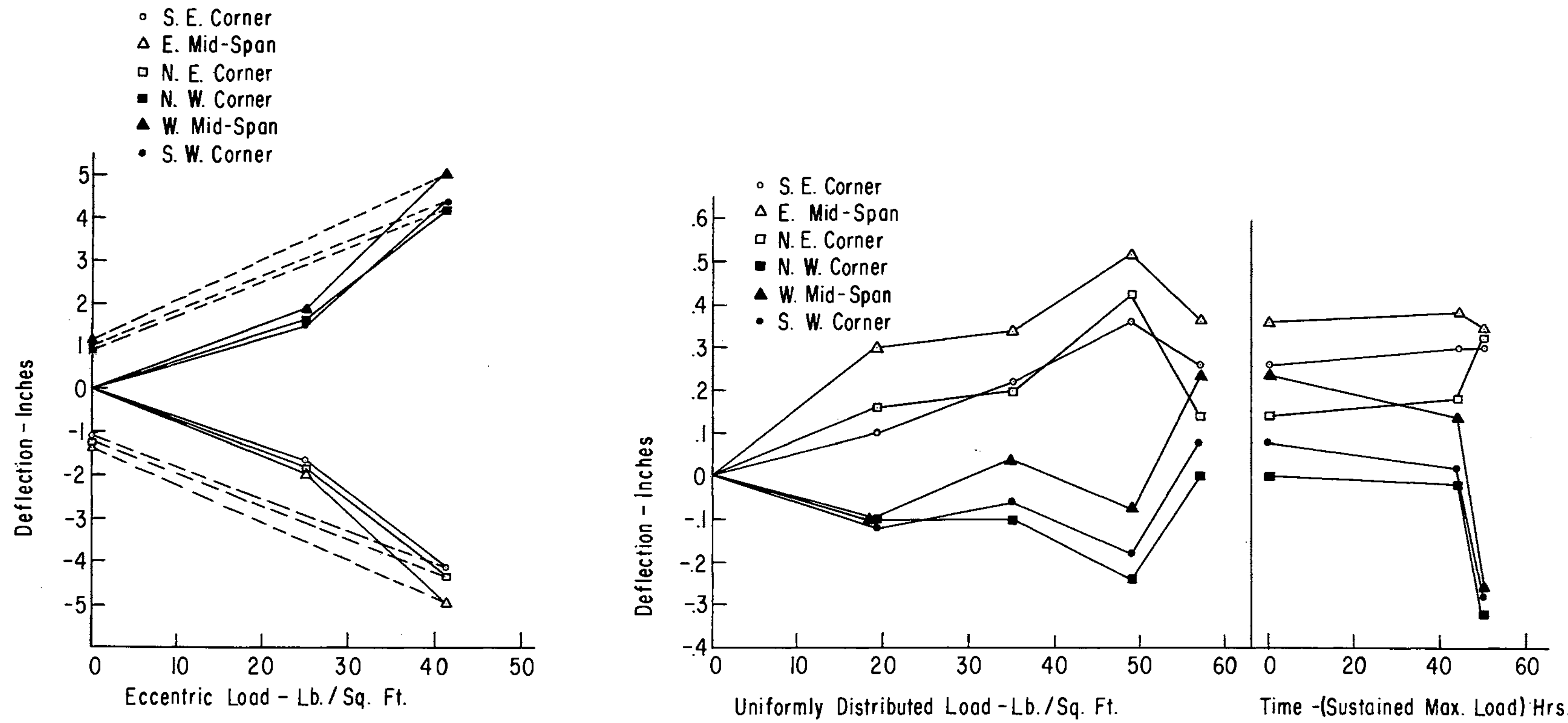


Figure 37. Roof and Column Deflection Curves For Test II.



(a) Test III Data

(b) Test IV Data

Figure 38. Roof and Column Deflection Curves for Test III and IV.

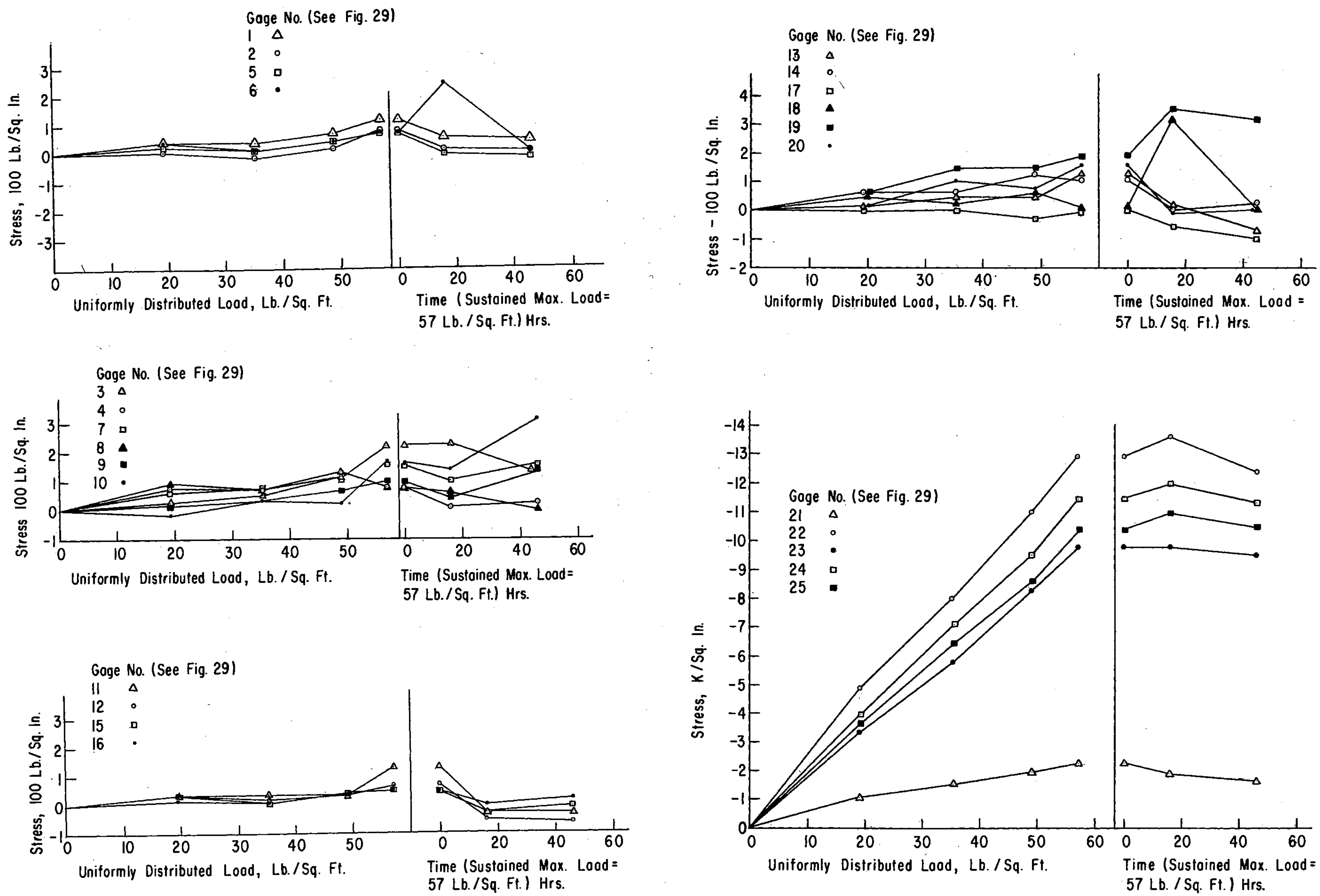


Figure 39. Stress Versus Load and Time Curves, Test IV.

CHAPTER IX

DISCUSSION OF RESULTS

The analysis of test results and a set of construction skill factors will be discussed in this chapter. The method of erection of the prototype structure used in this study will be examined and an erection procedure based on the research experience from this study will be recommended for use in the construction of h-p shells.

Assembly Components and Techniques

The discussion of assembly techniques includes the initial construction of forms and apparatus necessary for the assembly of the structure as well as the actual erection of the structural elements.

Column Forms

From the observations made during this study, the Douglas fir material used for the column forms would not be satisfactory for multiple reuse if extensive reuse was planned. After a period of approximately two weeks, the first 10 days of which the forms were constantly soaked, the side forms were warped to the extent that bracing or clamps would be necessary for reuse. It should be noted that under ordinary conditions, the forms would probably be removed after two to three days of curing; even then, this material, due to its non-homogeneous nature, would tend to warp unless well braced. A material which would provide adequate stiffness and strength for continuous reuse would be a 5 ply exterior grade or marine plywood. By using non-corroding hinges between the base and the sides of the form, and braces or stiffeners across the top face of the form, the column could be easily removed and the form could be quickly prepared for casting the next columns. By using a form which could be removed and prepared quickly, labor cost for forming could be reduced.

The haunches cast at the top of the column were developed for h-p structures consisting of only one unit with no walls or supports. This feature could be eliminated for structures composed of two or more shell units in which overturning moments were not acting upon the structure. However, the haunches provided a greater surface area for ease during the roof assembly. Eliminating the haunches would affect a savings in labor and equipment due to the large amount of special forming required by heating and bending the steel.

Even though the column may not be subjected to loads and moments as great as those applied in this study, the designer should consider transporting, lifting, and assembly loads which the column may be subjected to before it has been erected, as well as loads which it may receive during the roof assembly or before the individual structure is completed.

Shell Forming

The shell forms for this study were revised from a set used to cast an inverted umbrella shell at the Agricultural Engineering Laboratory in 1962. These form surfaces, which were used the second time during this study and had been stored out of doors, were showing signs of weathering. Covering the surface with a plastic coating, after the original surface was not usable, would permit additional uses to be obtained inexpensively. The metal base of these forms provided a rigid framework to keep the shell surface in its original shape and would stand the abuse of being transported to worksites for on-site casting. Due to the symmetry of the h-p shell, a minimum of two forms could be used if casting was done year-round and production demands were not excessive. Thus, material and labor costs of construction would be reduced.

Forming the shell steel for precasting required greater precision and more material than would be required in a cast-in-place shell. Lower design loads would allow the shell steel to be spaced wider. This would require fewer interior edge beam dowels to overlap with the shell steel. The reduction in the number of dowels used plus reducing the length of weld on the dowel base to approximately one inch per side of the dowel would reduce the warping problem which was encountered in this study. The shell steel forming could be further simplified by leaving out the reinforcing mat at the corner of the shell, used to resist local bending and radial stresses during eccentric loading. This mat would not be necessary for a shell loaded uniformly or connected to another shell or wall.

The shell casting operation would have been simplified by using steel chairs to support the shell steel instead of the three-fourth inch wood blocks. Several of these blocks were not removed during the casting operations. This could be critical in a building where several shell units were connected, and waterproofing and drainage were necessary. The concrete screeding operation was difficult in the area around the lift rings, but handworking around them was satisfactory. The lifting rings were easily installed and both the structural and functional design seemed to work satisfactorily.

At first, the low corner of each quadrant did not fit properly where it was seated on top of each column. This was detected during the test lift of the first quadrant onto the supports. The bottom tip of each corner had to be removed from each quadrant, which caused a delay of the assembly of approximately two man-hours. This method of connecting the quadrant to the column should be considered when designing the form surface in this area of the shell forms. A flat area could be formed easily by placing a wooden wedge in the low corner of each quadrant form.

Footing Methods

The footing used in this study performed satisfactorily, however, by not knowing the exact soil shear strength and bearing capacity, the footing may have been overdesigned both in regard to size of wing walls and the depth of the footing. The footing reinforcement cage used in this study functioned well in the assembly of the column but required special bending during construction. This probably could be eliminated by another type of footing. For a smaller design load and no overturning moments, a cylindrical footing with sufficient bearing area would be adequate.

Temporary Support System

The rigid assembly frame performed satisfactorily during the shell assembly process. For a larger shell, such as a 40 foot square structure, the corner towers would have to be braced so that the l/r ratio was less than the ratio used in this study and the possibility of buckling was reduced. The horizontal braces would have to be supported between the two towers. A metal or wooden post could serve the purpose of supporting the horizontal braces and also support the corners of the two roof quadrants; thus, the horizontal braces would reduce the unsupported length of the midspan supports.

The initial positions of the corner towers were not marked during the assembly of the rigid frame. If these positions had been established and marked and the towers positioned over the marks accordingly, time would have been saved during the initial frame assembly and much of the final frame adjustment.

The wooden center support was effective. However, improvements could be made in the bracing method used to keep the structure centered. Some methods which could be used are: (1) A metal stake driven into the ground with a metal brace from the tower to the stake. Bolt connections on each end of the brace with slotted adjustment holes for the lower end would provide the necessary adjustment. (2) A wooden member with a metal bracket bolted to the lower end to resist the wear of making repetitious connections. (3) A metal or wooden brace with a steel loop connected to the lower end to receive a steel stake. (4) A steel or wooden brace with a heavy-duty turnbuckle fixed rigidly to the lower end of the brace, adjusted to position or plumb the support. The particular method used would depend upon the amount of intended use and the relative cost of the alternative methods of bracing.

Lift Frame

The lift frame configuration used in this study would operate effectively on larger shells providing the unsupported length of the diagonal brace was not excessive. The addition of a second diagonal brace connected to the original brace at the center would make the frame more rigid and allow the use of materials of approximately the same dimensions that were used in this frame.

By using a three point arrangement of lift rings, a triangular shaped frame would work satisfactorily providing a standardized system of cables or chains was devised to complete the system. This configuration would lend itself well to a bolted frame assembly which could be assembled and dismounted rapidly.

A second alternative frame would be a simple "I" beam with a clevis on each end for the sling attachment. Balance of the quadrant could be maintained by two clamps on each edge of the shell parallel with the beam, connected to the center lift ring of the sling by small cables.

Column Erection

The column erection was costly in labor requirements as the complete erection required nine man-hours per column and was completed by a four man crew plus a crane operator. For structures with no heavy bending, a system of leveling bolts mounted on the base of the precast column would provide a satisfactory means of erecting and plumbing the column. This would require that the footings be located precisely before casting. After the column was plumbed by the bolts, the bolts would be welded, making the reinforcement continuous, and the joint would be completed by an expanding grout pack.

Other methods of providing rapid erection of precast columns by construction joints are presented by Rensaa (35), who used a precast footing socket; Naslund (15), suggests (1) a baseplate connection, (2) reinforcing bars from the column which extend into holes in the footing which are previously filled with grout, and (3) a slotted bar, cast into the footing and column, which is welded and the joint grouted. Cogan (36) developed an effective pipe connection by precasting into the footing a 4 inch pipe sleeve which was cut off at the correct elevation; the 4 inch pipe sleeve fit over a 3 inch pipe insert cast into the column. The insert had a steel ring or shoulder welded around it to give the exact elevation. The reinforcing bars were overlapped between the footing and column, and welded. The joint was completed by grouting with an expanding grout mix which prestressed the column reinforcement, thus giving a highly efficient joint.

Welded Connections

Four weld connections were made during the erection of the shell; these were: (1) the tie bar connection to column, (2) column and haunch to shell connection, (3) tension bar welded at center of roof, and (4) the interior edge beams.

The haunch to shell connection would be eliminated in a shell which was not designed for overturning moments or for a column with a different method of resisting moments. The connection on this shell worked adequately during eccentric loading. The column to shell connection was made quickly and efficiently by welding the interior edge beam angles and the web of the tie connector together. No change would be recommended for this erection step.

The tie connections performed satisfactorily in the structural sense but was not satisfactory functionally. During the erection of the roof quadrants, the tie bar protruded too far back along the web of the inverted "T" section, thus keeping the quadrants from seating properly; this caused approximately 3 man-hours delay while the quadrant corners were adjusted. This situation could be avoided by shaping the quadrant corners to compensate for the tie bar, or by a different method of attaching the tie bar. One method of adjusting the tie bar connection used in this study would be to use two 1 inch wide by 3/4 inch thick bars, welded on both sides of the web to the flange of the tie connection, with the 1 inch side placed horizontally. The tie bar could be placed between the two bars and fillet welded. A second method would be to notch the web of the "T" section from the flange up to a height equal to the tie bar diameter, and approximately 3 to 4 inches back from the end of the flange; the tie bar could be inserted and welded to the web on the top of the bar and to the flange on both sides of the bar at the base. Either method would have performed better functionally than the method which was used in this study.

Both the tension bar at the roof center and the edge beams should have been welded in sections or strips. This part of the shell design should have been examined more critically. The factor of safety of the weld on the tension bar was 5.0; this indicates that the welding should have been reduced to half of the amount used.

The edge beams could have been welded securely at the horizontal ends and at spaced intervals along the length. This would have reduced the welding time on the shell by at least one-half and saved on welding.

Recommended Shell Erection Procedure

The procedure for the erection of a two-column h-p shell of the configuration in Figure 2 could be carried out in the step-by-step procedure outlined below:

(1) Prepare site by clearing, leveling, construction staking, and excavating footings.
(2) Precast columns.
(3) Precast shell quadrants.
(4) Cast footings, if constructed separately.
(5) Move structural elements and construction apparatus to site and place in prescribed positions, Figure 23. (This should be accomplished while precast footings are curing for the first or second day.)
(6) Erect column and complete tie connection.
(7) Assemble and align the support system while columns are being erected, or while the footing or column construction joints are curing.
(8) Assemble the shell quadrants on the support system, adjust roof elevation, and pull quadrants together for welding.
(9) Connect quadrants to top of column by welded connections and weld tension bar at center of roof.
(10) Remove support system after tension bar at center of roof is welded and shell to column connection has been completed.
(11) Complete welding of edge beams.
(12) Waterproof steel edge beams.

This list of steps constitutes a procedure which can be utilized on one shell, or can be modified for use in erecting multiple shell structures; however, the erection of a number of shells to form a continuous structure is beyond the scope of this study.

DISCUSSION OF COST ANALYSIS

The observed cost data from this study have small significance in its present form; however, if this data can be adjusted by appropriate estimates based on the experience gained from this study, the adjusted data may serve as a useful guide for construction estimates on this type of shell.

The next four sub-sections will consider the data by (1) adjusting the material cost where appropriate, (2) adjusting the labor cost data by an appropriate skill factor based on the experience that a crew would have after becoming familiar with the construction routine, (3) adjusting the equipment costs that are related to the labor and material reductions, and (4) converting the values to a cost per square foot for the shell used in this study.

The final sub-section will be used to adjust the prototype data to a 40 foot square shell. Only the variable cost factors will be considered. Interest rates will not be considered in this study; the cost data will be considered as capital costs.

Material Cost Adjustments

By designing the columns by elastic analysis for a concentric load or small bending loads, the amount of steel reinforcement and the column size could be reduced. Table XI illustrates the reduction in material for a 20 foot square shell, which would result by designing for axial loads only.

TABLE XI

MATERIAL SAVINGS BY CONCENTRIC COLUMN DESIGN

Item of Material	Quantity	Unit Cost	Cost
No. 10 bar (Reduction due to No. 9 bar used for tie)	29.6 lbs.	$.097/lb.	$2.88
No. 8 bar (Column steel reduction to 4-No. 7 bars)	264.7 lbs.	.097/lb.	25.78
No. 6 bar (Haunch dowels)	24.0 lbs.	.097/lb.	2.33
No. 5 bar (Reinforcing mats and footing steel)	83.5 lbs.	.097/lb.	8.12
3 in. x 5 in. x 3/8 in. angle (Haunch-shell connector)	78.4 lbs.	.097/lb.	7.62
2 in. x 2 in. x 3/8 in. angle	37.6 lbs.	.097/lb.	3.66
Welding rod	15.0 lbs.	.20/lb.	3.00
Acetylene (Heating and bending)	1/4 bottle	5.70/bottle	1.42
Oxygen (Heating and bending)	1/4 bottle	5.65/bottle	1.41
	TOTAL		$56.22

From Table XI, the savings that could be realized between a structure which was subjected to overturning moments and one which was concentrically supported was evident. The savings of $50.39 on steel comprises approximately 25 percent of the shell steel costs. The concrete which would be saved on the haunch arms would probably be used to enlarge the footing diameter from 20 inches to 26 inches in order to provide adequate bearing area.

The reduction in equipment costs could be related to both the efficiency developed on each job and to the reduction in materials by a change in the support system design.

Another aspect of material costs is the cost for all shells after the construction of the first structure. By assuming that forms, the support system, lifting frame, and column cribbing will be reused indefinitely, the direct material cost for each future shell can be estimated from Table III. Table XII lists the costs of materials required for a 20 foot square h-p shell for (1) eccentric loading, and (2) for concentric loading. The adjustments which were made in the eccentric load costs are for deletion of costs for the erection apparatus. The adjustments in the concentric load cost include the deletion of costs of erection apparatus plus the items listed in Table XI. Thus, a savings in material costs of $56.18 would be made per 20 foot square shell by designing for concentric loading.

<u>Labor Cost Adjustment</u>

The labor costs which were tabulated in Table I are not usable except for estimating the time requirements of construction on a 20 foot square h-p shell with an

TABLE XII

ADJUSTED MATERIAL COSTS FOR TWENTY FOOT SQUARE H-P SHELL

Item	Quantity: Eccentric	Quantity: Concentric	Eccentric Cost	Concentric Cost
1. Welding Material				
(a) Welding rod	40 lbs., $.20/lb.	25 lbs., $.20/lb.	$8.00	$5.00
(b) Acetylene	7/8 bottle, $5.70 per bottle	5/8 bottle, $5.70 per bottle	4.98	3.56
(c) Oxygen	7/8 bottle, $5.65 per bottle	5/8 bottle, $5.65 per bottle	4.94	3.53
2. Concrete				
(a) Standard weight, 3,000 psi	3 cu. yd., $14.75 per cu. yd.	3 cu. yd., $14.75 per cu. yd.	44.25	44.25
(b) Lightweight Ag-regate, 3,750 psi	3 1/2 cu. yd., $18.25/cu. yd.	3 1/2 cu. yd., $18.25/cu. yd.	63.87	63.87
3. Steel Material	2056 lbs., $.097/lb.	1538 lbs., $.097/lb.	199.84	149.49
4. Form Oil	5 gal., $.80/gal.	5 gal., $.80/gal.	4.00	4.00
		TOTAL	$329.88	$273.70

unfamiliar crew. Table I was examined to arrive at appropriate skill factors for each type of operation which could be repeated.

The quality of labor used in the study was excellent, considering the attitude, education, and previous work experience. The productive working time per hour was estimated at 45 minutes per hour over all project operations. According to Dallavia (37) this would represent a working efficiency of 75 percent. Table XIII lists the operations with estimated skill factors based on expected construction efficiency during construction of the second h-p shell of this type.

TABLE XIII

LABOR ADJUSTMENTS BY SKILL FACTORS

Operation	Skilled Labor Factor	Unskilled Labor Factor	Adjusted Skilled Labor	Adjusted Unskilled Labor
1. Column Construction				
(a) Frame Assembly	.50	.85	1.0	30.0
(b) Steel Forming	.50	.80	2.5	43.2
(c) Casting and Curing	.50	.75	2.0	9.0
SUBTOTAL (Man-hours)			5.5	82.2
2. Shell Construction				
(a) Form Assembly	.25	.80	4.0	76.8
(b) Shell Steel Forming	.60	.80	4.8	62.4

TABLE XIII (Continued)

Operation	Skilled Labor Factor	Unskilled Labor Factor	Adjusted Skilled Labor	Adjusted Unskilled Labor
2. (Continued)				
(c) Form Preparation and Shell Casting	.60	.80	1.2	12.8
(d) Curing	.40	.75	1.2	7.5
SUBTOTAL (Man-hours)			12.4	159.5
3. Support System Construction				
(a) Fabricating Parts	.50	.80	0.5	5.6
(b) Welding Tower Frames	.80	---	28.8	---
(c) Assembly of Bolted Components	---	.85	---	2.5
(d) Wooden Support Fabrication	.40	.75	0.8	12.0
(e) Final Adjustments on Steel Supports	.50	.50	0.5	2.0
SUBTOTAL (Man-hours)			30.6	22.1
4. Lift Frame Construction	.70	---	4.2	---
5. Site Preparation				
(a) Leveling and Smoothing	---	.90	---	1.8
(b) Survey and Layout	.75	.75	1.5	1.5
(c) Foundation Excavation	.50	.50	1.0	4.5
SUBTOTAL (Man-hours)			2.5	7.8
6. Site Layout				
(a) Hauling and Placing Columns	.90	.90	1.8	1.8
(b) Wooden Support for Shell Transport	.50	.75	1.0	3.0
(c) Removing Shell Forms Transporting, and Placing Shells	.40	.50	1.6	3.5

TABLE XIII (Continued)

Operation	Skilled Labor Factor	Unskilled Labor Factor	Adjusted Skilled Labor	Adjusted Unskilled Labor
6. (Continued)				
(d) Transporting and Placing Supports	.50	.75	0.5	3.0
SUBTOTAL (Man-hours)			4.9	11.3
7. Column Erection				
(a) Construction of Cribbing for Column	.50	.75	2.0	18.8
(b) Cutting and Bending Footing Steel	---	.75	---	3.0
(c) Column Erection	.60	.75	2.4	10.5
(d) Casting footing	---	.85	---	0.8
(e) Removal of Braces and Site Cleanup	---	.85	---	3.4
(f) Tie Erection	.80	.80	1.6	1.6
SUBTOTAL (Man-hours)			6.0	38.1
8. Support System Erection				
(a) Initial Assembly	.50	.50	1.5	4.5
(b) Final Alignment	.50	.50	2.0	6.0
SUBTOTAL (Man-hours)			3.5	10.5
9. Erection of Shell				
(a) Initial Assembly	.40	.40	1.6	4.8
(b) Adjustment for Welding	.50	.50	1.0	2.0
(c) Welding	.50	---	7.5	---
(d) Support Removal	---	.70	---	4.2
(e) Grouting Haunches	---	.85	---	3.4
(f) Waterproofing Interior Edge Beams	.50	.75	0.5	3.8
(g) Final Clean-up	---	.75	---	3.0
SUBTOTAL (Man-hours)			10.6	21.2
FINAL TOTAL (Man-hours)			80.2	352.7

By adjusting the observed time data for the first construction cycle, the supervision and skilled labor cost was reduced to 57.1 percent of the initial supervision labor cost. The adjusted man-hours for the unskilled labor categories was 74.8 percent of the initial unskilled labor cost. The total adjusted man-hours for both labor categories was 70.7 percent of the combined man-hours of the initial construction phase. However, the final totals listed in Table XIII reflect data which would be reproduced only periodically; several operations which are shown would not be repeated in constructing the second shell. The items which would be deleted from Table XIII would be items 1(a), 2(a), 3, 4, 6(b), and 7(a). Thus, the revised total man-hours for supervision was 37.4 man-hours, while the total for unskilled labor was 202.0 man-hours. Comparing these values to the first data with the same sections deleted, the second combined man-hour total of 239.4 was 67.9 percent of the initial adjusted total. This total was only 39.1 percent of the initial phase combined total of 612 man-hours. Figure 40 shows the projected trends of the percentage of first unit man-hours under three conditions. For reuse of equipment, the repetitive operations are considered in Curve C. This is the estimated true situation; the trend of the percent of first unit labor would probably level out at around 25 percent of the FIRST UNIT values due to the high man-hour totals required to build the forms, lift frame, column cribbing, and support framework.

The estimated cost of labor for the second shell constructed would be \$120.19 for supervision and \$446.90 for unskilled labor, which would give a labor total cost of \$567.09. This would be 38.7 percent of the total cost. If the direct labor cost on this size of shell could be reduced to 25 percent of the original first unit cost, the projected cost would be \$367.00 which would be \$.92/ft.^{2} for labor. The labor cost for erection equipment which were constructed for multiple use, items 1(a), 2(a), 3, 4, 6(b), and 7(a) in Table I, was \$629.00. This cost is reduced for each reuse of the equipment by a proportional amount. For example, if a 200 foot by 80 foot warehouse were built using 20 foot square h-p shells, 40 h-p units would be required; thus, assuming all equipment was used 40 times, the equivalent cost per use would be \$15.72, or \$.0393/ft.^{2}

By considering a concentric load design, a further reduction in labor cost could be made by simplifying the column forming and footing, removing the shell angle used for the haunch connecter and removing the reinforcing mat in the corner of the shell. The labor reductions which this would create would be 5 man-hours for supervision and 58 man-hours for unskilled labor, Table XIV. These reduced items were taken from Table I. The labor savings obtained by the concentric design would be \$142.09. This would reduce the estimated second unit labor cost from \$567.09 to \$425.00, which is 28.9 percent of the original labor cost, \$1467.16.

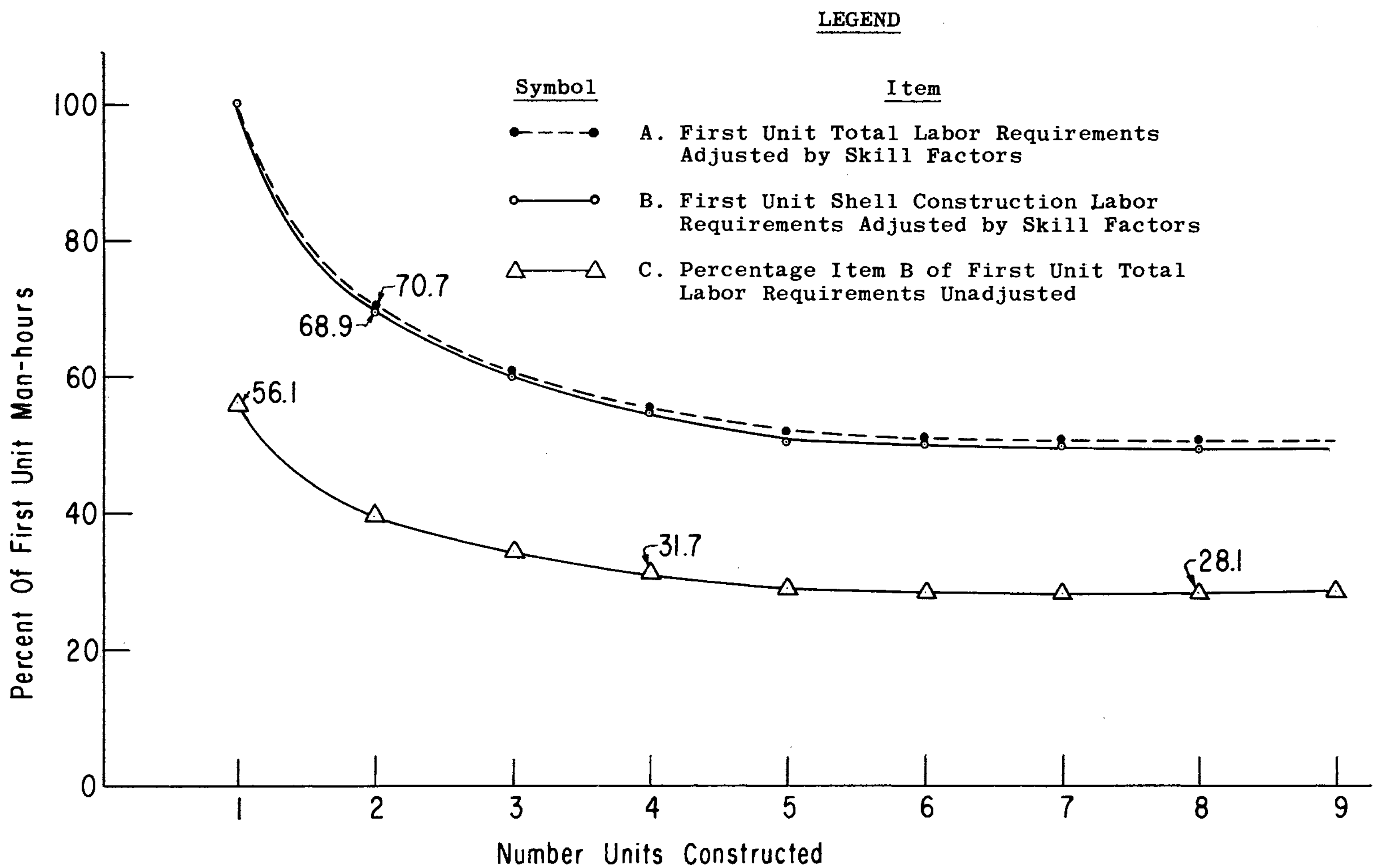


Figure 40. Labor Requirements Adjusted by Three Methods for Comparison.

TABLE XIV

LABOR REDUCTIONS DUE TO CONCENTRIC LOAD DESIGN

Items Reduced	Supervision Labor	Unskilled Labor
1. Column Construction		
(a) Forms	1	10
(b) Steel forming	2	30
2. Shell Construction		
(b) Steel forming	2	10
7. Column Erection		
(b) Cutting and bending footing steel	0	4
9. Erection of Structure		
(e) Welding	0	0
(d) Grouting haunches	0	4
TOTAL (Man-hours)	5	58

<u>Equipment Cost Adjustments</u>

The adjustment in equipment costs was dependent upon the adjusted labor and material costs. The equipment usage costs were adjusted on the basis of (1) eccentric design with labor adjusted for skill factors and reuse of erection apparatus, and (2) concentric design with labor adjusted and reuse of erection apparatus. The results of the equipment cost adjustments are tabulated in Table XV for the second shell unit.

TABLE XV

ADJUSTED EQUIPMENT COSTS FOR SECOND SHELL UNIT

ITEM	HOURS		COST	
	Eccentric Design	Concentric Design	Eccentric Design	Concentric Design
1. Acetylene Welder, $3.00/hr.				
(a) Forming column steel	10.4	0	$31.20	0
(b) Forming shell steel	4.8	4.0	14.40	$12.00
2. Electric Welder				
(a) Forming column steel, $2.00/hr.	2.4	1.0	4.80	2.00
(b) Tie bar erection, $3.00/hr.	1.6	1.6	4.80	4.80
(c) Forming shell steel,				
(1) Shop welding, $2.00/hr.	11.2	8.0	22.40	16.00
(2) Field welding, $3.00/hr.	3.2	2.7	9.60	8.10
(d) Footing cage, $2.00/hr.	0.8	0	1.60	0
(e) Portable welding on erected shell, $3.00/hr.	7.5	7.5	22.50	22.50
3. Tractor and Equipment Trailer, $2.50/hr.	6.0	6.0	15.00	15.00
4. Tractor and Lift Arm, $2.50/hr.	2.0	2.0	5.00	5.00
5. Fork Truck, $3.00/hr.	2.0	2.0	6.00	6.00
6. Crane, with Operator, $6.00/hr.	4.6	4.6	27.60	27.60
7. Tractor Dozer for Site Leveling, $6.00/hr.	1.8	1.8	10.80	10.80
8. Rotary Drill Rig, $12.50/hr.	1.0	1.0	12.50	12.50
9. Power Hacksaw, $2.00/hr.	3.5	2.5	7.00	5.00
TOTAL			$191.20	$137.30

From Table XV, an expected savings of $187.80 could be expected between the first and second units providing the equipment costs of the erection apparatus was included in the first unit cost for this comparison. A total equipment cost of $214.70 would be saved by constructing the second unit by concentric design compared to the eccentric design of the first shell. The difference of $53.90 between the costs in Table XV is attributed to the change in column and shell reinforcing with labor adjusted.

<u>Shell Costs Per Square Foot of Horizontal Projection</u>

By combining the data for labor, equipment, and materials for constructing a shell designed for eccentric loading with the costs of the erection apparatus adjusted for the number of uses, a cost of construction per square foot of horizontal roof surface can be obtained.

Assuming that the shell and column forms were used at least 10 times without repair or replacement of surfaces, the cost for the tenth shell could be estimated. From the original data, the costs of labor, materials, and equipment required to construct the support system, lifting frame, column cribbing, and forms was $644.20 for labor, $101.00 for equipment, and $556.07 for material; thus, the erection equipment total cost was $1,301.27. Pro-rating the total cost over 10 uses would give $130.13 per unit.

Figure 40 shows expected trends in labor man-hours.

requirements of future units based on increasing skills and job efficiency during the first few units constructed. The total labor requirement was described as First Unit Total Labor Requirements. This labor total included all labor used in fabricating the assembly components such as forms, steel and wooden supports, column support cribbing, and lifting frame, plus the precasting and erection of the structural elements. These labor requirements were adjusted by construction skill factors in Curve A.

The First Unit Total Labor Requirements were divided into two categories; these were labor for: (1) fabrication of casting and erection equipment (forms, column and shell erection supports, and lifting frame), (2) Column and shell casting, on-site assembly of supporting systems, and column and shell erection.

Curve B shows the labor requirements of item (2), in the previous paragraph, adjusted for increasing skill. Curve C shows item (2) adjusted for skill as a percentage of the First Unit Total Labor Requirements, unadjusted.

From Figure 40, the labor value could be estimated from Curve B to approach 50 per cent or slightly below after the 7th or 8th unit constructed. Subtracting the labor cost for forms and erection equipment, \$644.20, from \$1,467.00, the original cost, and multiplying by 0.50 gives a projected labor cost of \$411.40. The equipment costs would be proportional to the adjusted labor, except for the rotary drill truck; the adjusted equipment costs were \$153.25. The equipment costs due to the construction of the shell would be the original equipment cost, \$379.00, minus the cost due to forms and erection equipment, \$101.00 = \$278.00 for the construction of the first shell. The material cost was \$881.59 - \$556.07 = \$325.52.

The total cost for the tenth shell would be \$411.40 for labor, \$153.25 for equipment, \$325.52 for material, plus \$130.13 for each use of forms and equipment = \$1,020.30. This gives a cost of \$2.55 per square foot of horizontal projection, which is 37 per cent of the original cost for the first shell. Further uses of the erection apparatus could be readily projected as the data for the time, equipment, and material will not be expected to change appreciably.

Estimated Variable Costs for Forty Foot Square Shell

The material costs of a forty foot square shell could be considered to be directly proportional to the cost for material of a prototype of the same characteristics. The labor, material, and equipment costs, however, would vary from one size of shell to another. Table XVI lists estimated labor costs for a 40 foot square shell with a six foot rise and a column height of 10 feet.

The adjusted supervision labor averaged 1.21 more for the 40 square shell than the prototype, while the adjusted unskilled labor was 1.298 greater. The total adjusted labor cost for shell production was \$242.58 plus

$712.00 = $954.58. These costs do not include cost of construction of erection apparatus. The adjusted labor values for a 40 foot square shell were projected over several units constructed, Figure 41. The 100 per cent value represents the total of Table XVI.

The values for steel forming were estimated to take approximately the same time for a 40 foot square shell as a 20 foot square shell. The steel would be placed in longer lengths. However, the forming and tieing would require more time due to the larger number of junctions in the shell steel of the 40 foot square shell.

TABLE XVI

PROJECTED LABOR ESTIMATES FOR FORTY FOOT SQUARE H-P SHELL

Operation	Labor for 20 ft. x 20 ft. Shell		Adjustment Factor		Adjusted Labor for 40 ft. x 40 ft. Shell	
	Super.	Unsk.	Super.	Unsk.	Super.	Unsk.
1. Column Construction						
(a) Steel Forming	5	54	1.0	1.0	5	54
(b) Casting and Curing	4	12	1.2	1.4	4.8	16.8
2. Shell Construction						
(a) Steel Forming and Tieing	8	78	1.1	1.3	8.8	101.4
(b) Form Preparation and Shell Casting	4	16	1.0	2.5	4.0	40.0
(c) Curing	3	10	1.0	1.5	3.0	15.0

TABLE XVI (Continued)

Operation	Labor for 20 ft. x 20 ft. Shell Super.	Labor for 20 ft. x 20 ft. Shell Unsk.	Adjustment Factor Super.	Adjustment Factor Unsk.	Adjusted Labor for 40 ft. x 40 ft. Shell Super.	Adjusted Labor for 40 ft. x 40 ft. Shell Unsk.
3. Site Preparation						
(a) Leveling and Smoothing	0	2	-	1.5	0	3.0
(b) Survey and Layout	2	2	1.3	1.3	2.6	2.6
(c) Foundation Excavation	2	9	2.0	2.0	4.0	18.0
4. Site Layout						
(a) Hauling and Placing Columns	2	2	1.0	1.0	2.0	2.0
(b) Removing Forms, Loading, and Transporting Shells to Site	4	7	1.2	1.2	4.8	8.4
(c) Moving Support System	1	4	1.1	1.1	1.1	4.4
5. Erection of Supports	7	21	1.1	1.1	7.7	23.1
6. Erection of Structure						
(a) Initial Assembly	4	12	1.0	1.0	4.0	12.0
(b) Preparation for Welding	2	4	1.1	1.1	2.2	4.4
(c) Welding Time	15	0	1.5	-	22.5	-
(d) Support Removal and Site Cleanup	-	10	-	1.1	-	11.0
(e) Grouting Haunches	-	4	-	1.0	-	4.0
(f) Waterproofing Edge Beams	1	5	1.0	1.3	1.0	6.5
TOTAL (Man-hours)	64	252			77.5	326.6

The shell form costs were adjusted by determining the

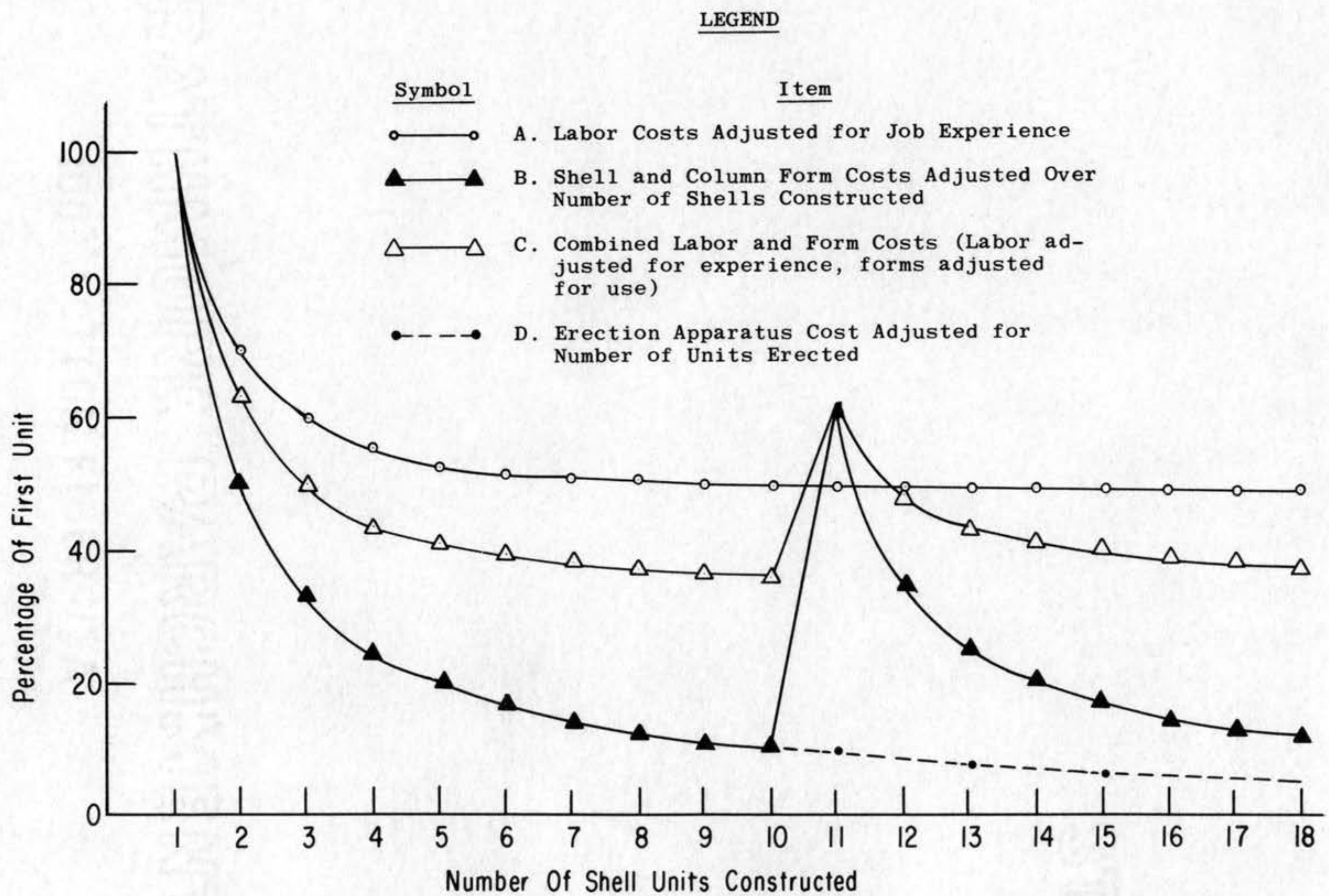


Figure 41. Variation of Labor and Erection Apparatus Costs With Increased Number of Shells Constructed.

amount of increase in material costs and adjusting the labor costs of the 20 foot square shell forms. The column forms were not adjusted as the same column size and shape could be used for the larger shell. Table XV lists the pertinent variable costs for the 40 ft. x 40 ft. shell. These values are used in Figure 41 to show the cost variation adjusted for experience and use. By adjusting the material costs in direct proportion to the prototype and increasing the equipment costs, the costs of constructing a 40 foot square shell can be readily estimated.

TABLE XVII

VARIABLE COSTS FOR FORTY FOOT SQUARE SHELL

Item	Cost
1. Total Form Costs	
(a) Form Material	$528.00
(b) Form Labor	501.00
2. Labor Cost for First Shell Constructed	954.58
3. Total Cost of Refinishing Column and Shell Forms	531.50
4. Labor Costs for Constructing Erection Apparatus	291.50
5. Material Costs for Erection Apparatus	388.00
6. Equipment Costs for Erection Apparatus	108.00

Discussion of Load Test Results

The analysis of the test data in Chapter VIII was made to determine whether the shell and columns reacted according to the design. A discussion of factors which were not covered in Chapter VIII will be presented in the following paragraphs.

The maximum shell stress at the center of the interior horizontal edge beam was only 18 per cent of the design load, 19,000 lbs., therefore, it was assumed that the sloped interior edge beams in the local area surrounding the tension bar at the roof center actually took 82 per cent of the tension.

From the strain readings in the tie bar, an account was maintained of the horizontal shear forces acting at the intersection of the column and roof. The construction joint at the haunch and column absorbed much greater bending stresses than the design indicated. This was pointed out by the fact that the maximum tensile force in the tie bar was 16,990 lbs. for a live roof load of 57.0 lbs./ft.^{2} compared to the design load in the tie of 19,000 lbs. The maximum total force that the tie measured was composed of the load of the shell, approximately 32 lbs./ft.^{2}, plus a uniformly distributed gravity load of 61.7 lbs./ft.^{2}, or 93.7 lbs./ft.^{2}; this gave an observed tensile force which was slightly less than the allowable load for the tie bar. This load was computed from the original strain readings on the tie bar just after the shells had been welded together and the supports lowered, which averaged 218.5 micro-in./in., plus a maximum strain of 405 micro-in./in. measured during Test II.

By recalculating the value of tensile force which should have been taken by the tie according to design, using the actual dead load of 32.0 lbs./ft.^{2}, the tensile force should have been 32,540 lbs. Assuming that the strain of 218.5 micro-in./in. had not changed during the four months period between the shell completion and the testing period, the difference between 32,540 lbs. and 27,955 lbs., 4,585 lbs., was the calculated maximum shearing force resisted by each column. The calculated unit shearing stress in the concrete by the method for flexural members was 43.8 psi, which was approximately one-half of the allowable shear stress.

An interesting correlation was noticed between the observed strains in the tie bar after the quadrants were pulled together before welding, and after welding was completed and the supports were lowered, compared to the maximum strains during Test II. The first two average readings were 165 micro-in./in. before welding of the edge beams and 218.5 after lowering the supports; both of these readings were taken after the column to shell connection had been made so the difference in strain was considered to be induced by lowering the supports. The static load of 61.7 lbs./ft.^{2}, which was nearly double the dead load, was placed on the roof after the column haunches had been grouted. If the load-strain trend had continued linearly, and the columns had not been grouted, an expected strain value of 422 micro-in./in. would have been obtained. The difference between 422 and the observed strain value of 405 micro-in./in., or 17 micro-in./in. was considered to be a measure of increased stiffness of the joint due to grouting. The bending resistance was increased by approximately 4 per cent by grouting the haunch.

It was believed that the rigidity exhibited by the construction joint was due to the wide haunch and shell connection. This connection including the edge beam to column weld spanned a horizontal distance of 4 feet and was "V" shaped with a side slope of 3 to 10. Thus, the joint could not react as a pinned or simple connection, which was the design assumption.

The tie bar area could be reduced by 14.0 per cent because of the difference between the design tensile stress and the measured stress. However, changes in soil conditions due to ground water and moisture infiltration should be considered before changing the design.

The shell quadrants were assumed to transfer all roof loads, including eccentric loads, as shearing forces through the parabolic arches into the edge beams of the quadrant. However, from visual observation during the cantilever load of 41.3 lbs./ft.^{2}, the shell was subjected to bending stresses. Cracks perpendicular to the edge beam centerline extended to the midpoint of the edge beam on the loaded side of the structure. This indicated heavy bending stresses in the beam, causing the section to crack. Three cracks were observed approximately 12 to 15 inches apart, starting about 12 inches past the end of the haunch on the south column. A similar pattern was observed on the north edge of the shell.

During the maximum cantilever load, the shell raised off both haunches on the unloaded side by approximately 1/16 to 1/8 inch. This occurred due to the slack in the welded connection. Two hairline cracks appeared in a radial direction around the outside of the reinforcing steel mat location on the north unloaded quadrant. These cracks had radii of approximately 2 feet and $2\frac{1}{2}$ feet from the column center. No cracks or other visible stress were noticed around the column in the quadrants which were loaded. However, after the shell had been loaded for 45 minutes, the roof edges were still deflecting slowly.

The maximum observed deflections were +5.0 inches and -5.0 inches at the west and east roof edge mid-spans. The weld connection at the mid-span of the roof edges showed greater deflections than the corners. The maximum observed deflections of the corners during the half-roof load were -4.16 inches at the southeast corner, -4.36 inches at the northeast corner, +4.18 inches at the northwest corner, and +4.34 inches at the southwest corner.

During the sustained uniformly distributed load test of 61.7 lbs./ft.^{2}, the average maximum deflection of corresponding roof points was 0.30 inches at the east and west roof edge mid-spans. Under the same load, the average deflections were 0.11 inches for the northeast and northwest corners and 0.18 inches for the southeast and southwest corners. On the basis of a deflection to span ratio, the mid-span deflection would be 0.9 in./360 in.

CHAPTER X

SUMMARY AND CONCLUSIONS

A construction engineering study was conducted on a two-column h-p shell to (1) develop a step-by-step precasting procedure, (2) test the structural design values, and (3) to study the costs of production of the shell and correlate the observed cost data to usable data for use in construction cost estimating on this type of shell.

A construction procedure was developed for trial during the study. This procedure was carried out to simulate prototype conditions by using an untrained crew with a staff member from the Agricultural Engineering Department acting as construction foreman. After the erection procedure was completed, the individual construction steps and methods were analyzed critically and alternate methods were suggested. These were discussed in Chapter IX.

The cost study was divided into three sections for analysis of the data; these were (1) labor costs, (2) material costs, and (3) equipment costs. After the raw data were analyzed in Chapter VIII, the labor data were adjusted by skill factors for each labor operation in Chapter IX. Estimates were made from construction experience for adjusting the variable cost data from a 20 foot square shell to a 40 foot square shell.

Curves were plotted showing the expected trend of labor, form, and erection apparatus costs as a function of number of units constructed.

The structural test data included strain data plus roof and column deflection data. The strain data were obtained from strain gages mounted on the columns, as illustrated in Figure 29, four strain gages mounted on the tie bar, and one strain gage mounted at the center of the horizontal interior edge beam. The deflection data were obtained from a differential water level manometer with the reservoir connected to to a moveable end by a plastic tube. Observations were taken at six points on the edge of the roof, at the roof center, and on the column opposite the reservoir.

RESULTS AND CONCLUSIONS

The following conclusions were drawn from the investigation:

(1) Precast h-p shells can be erected by rural builders and general contractors by following the simple erection sequence outlined in Chapter IX.

(2) Precast h-p shells can function as well structurally as conventional, cast-in-place shells.

(3) Prefabricated h-p shell elements can be stockpiled, transported to a building site, and erected efficiently.

(4) Precasting reduces the amount and complexity of formwork as compared to conventional h-p shell

construction, thus, reducing forming costs due to labor.

(5) The welded edge beams with reinforcing dowels functioned well as shear transfer members.

(6) Welding the shell quadrants together is a fast, effective means of obtaining an efficient construction joint.

(7) The cribbing used for supporting the precast columns during erection was an inefficient means of erection.

(8) The steel assembly jig effectively controlled the vertical and horizontal positions of the shell quadrant corners.

(9) The configuration of the haunch connection to the shell produced a joint capable of resisting moment stresses between the columns and edge beams. Grouting the haunches increased the rigidity by 4 percent.

(10) The exterior sloped edge beams on the loaded quadrants were subjected to bending stresses when the shell was eccentrically loaded. Bending cracks occurred along the edge beam under the maximum cantilever load of 41.3 lbs./ft.^{2}

(11) The measured tensile force in the horizontal interior edge beam was approximately 18 percent of the design stress, 19,000 lbs. for a uniform roof load of 57.0 lbs./ft.^{2} The remainder of the stress was resisted by the splice plates joining the sloped interior edge beams.

(12) The bending moments from observed column bending strains analyzed by the cracked section method were 249,000 and 220,000 in.-lbs. for the north and south columns compared to the theoretical value of 248,000 in.-lbs. during eccentric loading in the direction of overturning.

(13) The shell was subjected to radial bending stresses around the reinforcing mat, cast into the corner of each shell, during eccentric loading; this was indicated by hairline cracks in a radial direction around the column in the unloaded quadrants.

(14) The haunch arms were effective in resisting overturning moments. The welded angles connecting the shell and haunch allowed 1/16 to 1/8 inch deflection of the shell above the haunch arms on the unloaded side of the shell during eccentric loading.

(15) The steel edge beams and column haunches plus the wingwall footing increased the shell material costs by \$56.18 above the cost for a similar shell design for only axial loads. However, this additional expense can be offset by reduced labor costs.

(16) The calculated difference in total cost between the prototype shell and a shell designed for concentric loading was \$252.17 or \$0.63/ft.^{2}

(17) The maximum roof deflection under the cantilever

roof load was 5.0 inches at the mid-span of the horizontal exterior edge beam.

(18) Under a sustained uniformly distributed roof load of 61.7 lbs./ft.^{2}, the average maximum deflection of corresponding roof points was .30 inches for the east and west midspans. The same deflection was observed for a sustained load of 57.0 lbs./ft.^{2} This gave a deflection to span ratio of 0.9 in./360 in.

(19) After ten uses of forms and erection apparatus, the cost of erecting a 20 foot square precast h-p shell can be reduced to \$2.55/ft.^{2}, or 37 percent of the first cost total.

(20) The h-p shell roof quadrants were assembled on the support system in 1 hour and 20 minutes.

(21) The 40 foot square shell could be erected with an estimated 21 percent increase in supervision and 29.8 percent increase in unskilled labor costs based on estimates from the construction experience of this study.

SUGGESTIONS FOR FURTHER STUDY

Investigate further the stresses under uniform and eccentric loading by mounting strain gages on the reinforcing steel in the columns and haunches.

Fabricate an efficient deflection measuring apparatus for future testing using the principle of the system used in this study. Hook gages could be used for accuracy.

Experimentally investigate the stresses in the edge beam steel for a prefabricated h-p shell with welded steel edge beams.

Conduct a construction engineering study for a prefabricated h-p shell utilizing new methods of column erection which were suggested in Chapter IX.

Analyze the stresses in the edge beams, tie, and column on an h-p shell to determine the stresses resisted by each member.

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APPENDIX A

TIE BAR CALIBRATION TEST DATA

TABLE A-1

OBSERVED STRAIN DATA FROM TESTS

OF TIE BAR SAMPLES

Reading No.	Load (lbs.)		Strain (in./in. x 10^{-6})			
	Bar #1	Bar #2	Bar #1		Bar #2	
			Gage #1	Gage #2	Gage #3	Gage #4
1	2,100	1,830	-190	+105	0	-70
2	3,950	3,990	-310	+145	-10	-155
3	6,100	5,640	-300	+150	-15	-230
4	7,980	8,070	-480	+135	-30	-330
5	9,680	9,820	-525	+100	-35	-400
6	11,540	13,000	-560	+50	-75	-510
7	17,640	15,180	-675	-120	-110	-580
8	18,860	17,880	-690	-145	-160	-650
9	22,800	19,500	-770	-260	-190	-700
10	23,840	22,520	-785	-280	-255	-775
11	25,070	25,100	-810	-310	-310	-840
12	26,090	27,230	-840	-340	-360	-890
13	28,000	30,020	-870	-390	-420	-960
14	29,210	32,200	-895	-425	-470	-1020
15	31,480	35,230	-920	-500	-535	-1095
16	33,690	----	-930	-550	----	----

APPENDIX B

SHELL ERECTION STRAIN GAGE DATA

TABLE B-1

STRAIN GAGE DATA FROM TIE BAR

DURING SHELL ERECTION

Condition	Observed Strain (in./in. x 10^{-6})			
	Gage #22	Gage #23	Gage #24	Gage #25
1st Quadrant Erected (4:00 p.m.)	0	-15	0	-25
2nd Quadrant Erected (4:25 p.m.)	-5	-10	-50	-35
3rd Quadrant Erected (4:45 p.m.)	-30	+10	-20	-40
4th Quadrant Erected (5:20 p.m.)	-50	0	-15	-80
Quadrants Drawn Together	-200	-120	-80	-260
Quadrant Edge Beams Welded, Column Welded to Shell Angles	-135	-85	+30	-130
Support System Lowered	-250	-225	-115	-275

Remarks:
(1) Shell erection was begun on October 8, 1963.
(2) Supports were lowered on October 25, 1963.
(3) Tension is indicated by minus (-) signs.

APPENDIX C

DEFLECTION DATA FROM LOAD TESTS I-IV

REMARKS AND DEFINITIONS

Defl. = Deflection
Acc. Defl. = Accumulated Deflection
S.E.C. = Southeast Corner of Roof
E.M.S. = East Mid-span of Roof Edge
N.E.C. = Northeast Corner of Roof
N. Col. = North Column
N.W.C. = Northwest Corner of Roof
W.M.S. = West Mid-span of Roof Edge
S.W.C. = Southwest Corner of Roof
C. of R. = Center of Roof
Deflections were measured in inches.
Positive (+) deflection was measured downward.

TABLE C-1

TEST I ROOF AND COLUMN DEFLECTION DATA

Load	21.6 psf		21.6 psf		21.6 psf		21.6 psf		0	
Time	1 hr.		24 hrs.		48 hrs.		72 hrs.		74.5 hrs.	
Location	Deflection (Inches)									
	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.
S.E.C.	+.08	+.08	0	+.08	0	+.08	0	+.08	-.10	-.02
E.M.S.	+.10	+.10	0	+.10	+.04	+.14	0	+.14	-.20	-.06
N.E.C.	+.10	+.10	-.02	+.08	-.04	+.04	+.04	+.08	-.06	+.02
N.W.C.	+.08	+.08	-.04	+.04	+.02	+.06	+.02	+.08	-.02	+.06
W.M.S.	+.08	+.08	-.02	+.06	+.04	+.10	-.06	+.04	-.10	-.06
S.W.C.	+.02	+.02	+.06	+.08	-.02	+.06	-.10	-.04	+.02	-.02
C. of R.	+.06	+.06	+.06	+.12	-.18	-.06	+.02	-.04	-.08	-.12

TABLE C-2
TEST II ROOF AND COLUMN DEFLECTION DATA

Load	25.3 psf		46.9 psf		61.7 psf		61.7 psf		61.7 psf		61.7 psf		61.7 psf		61.7 psf		0	
Time	$1\frac{1}{2}$ hrs.		3 hrs.		4 hrs.		24 hrs.		49 hrs.		74 hrs.		96.5 hrs.		115 hrs.		$117\frac{1}{2}$ hrs.	
Location	Deflection (Inches)																	
	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.
S.E.C.	+.06	+.06	-.26	-.20	-.06	-.26	+.04	-.22	+.20	-.02	+.04	+.02	-.04	-.02	+.08	+.06	+.12	+.18
E.M.S.	+.12	+.12	-.10	+.02	-.06	-.04	+.04	0	+.14	+.14	0	+.14	+.02	+.16	0	+.16	+.04	+.20
N.E.C.	+.04	+.04	-.18	-.14	+.02	-.12	-.02	-.14	+.14	0	-.06	-.06	+.02	-.04	0	-.04	+.20	-.16
N.Col.	+.06	+.06	-.02	+.04	-.02	+.02	0	+.02	+.02	+.04	-.02	+.02	-.06	-.04	-.02	-.06	+.06	0
N.W.C.	-.12	-.12	+.24	+.12	+.06	+.18	+.10	+.28	-.12	+.16	-.04	+.12	+.02	+.14	+.02	+.16	-.26	-.10
W.M.S.	-.08	-.08	+.42	+.34	+.18	+.52	+.06	+.58	-.16	+.42	+.02	+.44	0	+.44	-.02	+.42	-.46	-.04
S.W.C.	+.02	+.02	+.20	+.22	+.08	+.30	+.10	+.40	-.16	+.24	+.02	+.26	+.02	+.28	+.02	+.30	-.30	0
C.ofR.	+.14	+.14	+.04	+.18	-.10	+.08	+.06	+.14	0	+.14	+.10	+.24	-.02	+.22	-.04	+.18	-.14	+.04

TABLE C-3

TEST III ROOF AND COLUMN DEFLECTION DATA

Load	25.0 psf		41.3 psf		0	
Time	$1\frac{1}{4}$ hrs.		2 hrs.		3 hrs.	
Location	Deflection (Inches)					
	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.
S.E.C.	-1.66	-1.66	-2.50	-4.16	+3.06	-1.10
E.M.S.	-2.00	-2.00	-3.00	-5.00	+3.66	-1.34
N.E.C.	-1.86	-1.86	-2.50	-4.36	+3.16	-1.20
N.Col.	-.04	-.04	0	-.04	-.02	-.06
N.W.C.	+1.60	+1.60	+2.58	+4.18	-3.26	+.92
W.M.S.	+1.84	+1.84	+3.16	+5.00	-3.90	+1.10
S.W.C.	+1.56	+1.56	+2.78	+4.34	-3.36	+.98
C.ofR.	0	0	-.02	-.02	+.02	0

TABLE C-4

TEST IV ROOF AND COLUMN DEFLECTION DATA

Load	19.3 psf		35.4 psf		49.0 psf		57.0 psf		57.0 psf		0	
Time	$1\frac{1}{2}$ hrs.		$2\frac{1}{4}$ hrs.		$3\frac{1}{4}$ hrs.		$4\frac{1}{2}$ hrs.		20 hrs.		51 hrs.	
Location	Deflection (Inches)											
	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.	Defl.	Acc. Defl.
S.E.C.	+.10	+.10	+.12	+.22	+.14	+.36	-.10	+.26	+.04	+.30	0	+.30
E.M.S.	+.30	+.30	+.04	+.34	+.18	+.52	-.16	+.36	+.02	+.38	-.04	+.34
N.E.C.	+.16	+.16	+.04	+.20	+.22	+.42	-.28	+.14	+.04	+.18	+.14	+.32
N.Col.	0	0	-.02	-.02	0	-.02	+.02	0	-.04	-.04	0	-.04
N.W.C.	-.10	-.10	0	-.10	-.14	-.24	+.24	0	-.02	+.02	-.30	-.32
W.M.S.	-.10	-.10	+.14	+.04	-.12	-.08	+.32	+.24	-.10	+.14	-.40	-.26
S.W.C.	-.12	-.12	+.06	-.06	-.12	-.18	+.26	+.08	-.06	+.02	-.30	-.28
C.ofR.	0	0	+.04	+.04	+.10	+.14	+.04	+.18	-.06	+.12	-.10	+.02

APPENDIX D

STRAIN GAGE DATA FROM LOAD TESTS II-IV

TABLE D-1

TEST II OBSERVED STRAIN
(Inches x 10^{-6})

Load	0	25 psf	47 psf	61.7 psf						0
Time	0	$1\frac{1}{2}$ hrs.	3 hrs.	4 hrs.	24 hrs.	49 hrs.	74 hrs.	96 hrs.	$115\frac{1}{2}$hrs.	117 hrs.
Gage No.										
1	0	-20	+20	+60	+50	-40	+10	+35	+20	+10
2		-35	+10	+45	+20	-65	-10	+20	0	0
3		-40	+25	+70	+40	-40	+10	+30	+10	-10
4		-40	-5	+40	+10	-50	-10	+15	+15	+5
5		-30	+10	+40	+30	-50	0	+30	+120	+90
6		+250	+220	+280	+335	+220	+220	+185	+100	+115
7		-50	-20	+15	+10	+220	-25	-40	-50	-85
8		-40	-20	+5	0	-45	+10	-10	0	-5
9		+10	+10	+50	+35	+55	+25	+10	+20	+10
10		-5	+20	+60	+55	+50	+20	0	0	-40
11		-10	+10	+50	-10	-45	-10	-10	-30	0
12		-20	-10	+20	-10	-80	-20	-20	-25	0
13		+5	+60	-85	0	-30	-45	-50	-60	-60
14		+20	-80	-15	-90	-160	+30	+15	+30	+60
15		-20	-20	+10	-20	-50	-10	-15	-15	0
16		-10	0	+30	-10	-35	0	0	-15	0
17		-25	-20	+30	+60	-65	+10	-65	+5	+10
18		-30	-40	-5	+5	-50	0	-30	+15	+10
19		-20	-40	-20	-45	-40	-90	+20	-130	-110
20		-5	+30	+70	+80	+15	+20	+20	+15	-10
21		---	---	---	---	-315	-320	-310	-285	-190
22		0	-320	-415	-415	-420	-430	-440	-440	0
23		-15	-265	-365	-340	-370	-370	-380	-370	-30
24		-5	-290	-395	-420	-420	-415	-420	-425	-10
25	0	-5	-270	-360	-380	-380	-370	-380	-385	-10

Remarks: (1) Test dates were February 24-29, 1964. (2) Loading, uniformly distributed gravity load. (3) Steel gage factor, F=2.11. (4) Concrete gage factor, F=2.13. (5) Steel gage resistance, 120 ohms. (6) Concrete gage resistance, 300 ohms. (7) Strain data is uncorrected. (8) Minus (-) sign indicates tension.

TABLE D-2

TEST III OBSERVED STRAIN
(Inches x 10^{-6})

Load	0	25.0 psf	41.3 psf	Unloaded
Time	0	$1\frac{1}{4}$ hrs.	2 hrs.	3 hrs.
Gage No.				
1	0	+30	+20	0
2	↓	+60	+65	-10
3		+150	+280	+20
4		-70	-420	-30
5		+10	-10	0
6		0	-30	-10
7		-25	-185	-50
8		-250	-550	0
9		0	+40	+30
10		+200	+430	+60
11		-20	+5	+15
12		-10	+5	-30
13		+30	+230	0
14		+120	-350	-40
15		+50	-50	-25
16		+40	-50	-15
17		-100	-240	-85
18		+230	-470	-60
19		-40	-90	-110
20		+150	+365	+85
21		-30	-15	-10
22		-70	-120	+20
23		-60	-110	0
24	↓	-60	-110	+10
25	0	-60	-110	+10

Remarks:
(1) Test date, March 2, 1964.
(2) Load, uniformly distributed on half of the roof.
(3) Same as Remarks (3) through (8), Test II.

TABLE D-3

TEST IV OBSERVED STRAIN

(Inches x 10^{-6})

Load	0	19.3 psf	35.4 psf	49.0 psf	57.0 psf	61.7 psf		0
Time	0	$1\frac{1}{2}$ hrs.	$2\frac{1}{4}$ hrs.	$3\frac{1}{4}$ hrs.	$4\frac{1}{2}$ hrs.	20 hrs.	50 hrs.	51 hrs.
Gage No.								
1	0	+10	+10	+20	+35	-10	-55	-70
2		0	-10	0	+20	-30	-85	-85
3		+10	+20	+40	+80	+80	+40	-90
4		+25	+20	+40	+20	-30	-70	-80
5		+5	0	+10	+20	-30	-75	-70
6		+10	0	+10	+20	-15	-60	-60
7		+20	+20	+30	+40	-25	-95	-155
8		+30	+20	+40	+20	-10	-80	-80
9		+5	+10	+20	+25	-20	-40	-90
10		-10	+10	0	+45	0	-10	-130
11		+10	+10	+10	+20	-30	-60	-45
12		+10	+5	+10	+20	-40	-70	-45
13		+5	+20	+20	+40	+25	+20	+50
14		+20	+20	+40	+30	-30	-65	-75
15		+10	0	+10	+10	-40	-75	-70
16		+5	0	+10	+10	-30	-65	-70
17		0	0	-10	0	-20	-40	0
18		+20	+15	+30	+15	+170	+130	+130
19		+20	+50	+50	+60	+105	+50	-70
20		+5	+40	+30	+60	+20	+40	+40
21		-35	-50	-65	-75	-60	-50	+5
22		-160	-260	-360	-420	-430	-380	+25
23		-110	-190	-270	-320	-320	-310	0
24		-130	-230	-310	-375	-390	-380	-10
25	0	-120	-210	-280	-340	-360	-345	-5

Remarks: (1) Test dates were 3-5 March, 1964. (2) Loading was by uniformly distributed gravity load. (3) Same as Remarks (3) through (8), Test II.

VITA

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