FOUNDATION
PERFORMANCE
COMMITTEE

# 1998 SYMPOSIUM

# BUILDING BETTER FOUNDATIONS FOR THE RESIDENTIAL HOME INDUSTRY

GEORGE R. BROWN
CONVENTION CENTER

HOUSTON, TEXAS

NOVEMBER 5, 1998

# FOUNDATION PERFORMANCE COMMITTEE

## 1998 SYMPOSIUM AGENDA

**BUILDING BETTER FOUNDATIONS FOR THE RESIDENTIAL HOME INDUSTRY**
**The George R. Brown Convention Center**
**Houston, Texas**
**November 5. 1998**

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| **INTRODUCTION** | |
| o Foundation Performance Committee 1998- Dick Peverley, PE | 8:40 - 9:00 |
| o Professional Ethics - Why They Are So Important - Bill Lawson, PE | 9:00 - 9:30 |
| **LEGAL ISSUES** | |
| o The RCLA as It Applies to Foundation Work-Jack Rose | 9:30 - 10:00 |
| o A Review of Construction Litigation Cases-Leonard Meyer | 10:00 - 10:30 |
| Break | 10:30 - 11:00 |
| o Litigation from the Insurance Carrier's View-Bob Singleton | 11:00 - 11:30 |
| o Expert Witness Examination & Testimony-Dick Peverley, PE | 11:30 - 12:00 |
| o Forensic Materials Testing - David Eastwood, PE | 12:00 - 12:30 |
| Lunch: There will be a catered lunch from | 12:30 - 1:30. |
| **FOUNDATION DESIGN ISSUES** | |
| o State Subcommittee Report-Chairman, Kirby Meyer, PE | 1:30 - 2:00 |
| o A State of Art Review of Tract Home Foundations-Lowell Brumley PE | 2:00 - 2:20 |
| o A State of Art Review of Custom Home Foundations-Michael Skoller PE | 2:20 - 2:40 |
| o Soil Testing for 1998 and Beyond-David Eastwood, PE | 2:40 - 3:00 |
| Break | 3:00 - 3:20 |
| **FOUNDATION CONDITION SURVEY** | |
| o State Subcommittee Report-Bill Lawson, PE | 3:20 - 3:40 |
| **CONSTRUCTION ISSUES** | |
| o Construction Testing-Jack Spivey | 3:40 - 4:00 |
| o Building Code Integration- Joe Edwards | 4:00 - 4:20 |
| **SEWER LEAK ISSUES** | |
| o Subcommittee Report - Bob Newman, PE | 4:20 - 4:40 |
| **FOUNDATION REPAIR** | |
| o Subcommittee Report - Ann Nelson | 4:40 - 5:00 |
| **ROUND TABLE DISCUSSION** | 5:00 - 5:30 |

**The theme of this meeting will be to explore ways in which the Foundation Performance Committee can be more responsive to filling the needs of the residential home building business and the residential real estate business.**

# 1998 SOILS-STRUCTURE INTERACTION SEMINAR

## ABOUT THE

## FOUNDATION PERFORMANCE COMMITTEE

BY
RICHARD W. PEVERLEY

The Foundation Performance Committee was founded in 1991 initially through the efforts of David Eastwood. The work of the Committee was then continued through the participation of individuals who were involved with the design, construction, inspection and repair of residential and other forms of light construction. Meetings were held on a regular basis and subcommittees were formed to investigate such issues as deflection criteria, foundation failure criteria, the use of void cartons, etc. As a result of these activities, those of us who have participated have gained in knowledge and experience as well as having gained valuable communication interfaces with our contemporaries. In 1994, we were able to conduct our second seminar on the subject of soils-structure interaction.

Perhaps our most outstanding achievement, however, occurred 1996 when we became incorporated as a non-profit corporation known as the Foundation Performance Committee. The objectives of this Corporation, as stated in the by-laws, includes the following:

a) To serve the public by advancing the skill and the art of engineering analysis, investigation, and consultation in the design, construction, and repair of light structural foundations; primarily for residential buildings.
b) To engage in research through the conduct of seminars and the publication of technical papers, books, and articles on the science of residential design, construction, and repair of light foundations.
c) To maintain a library of information on the science of design, construction, and repair of light foundations.
d) To establish criteria for the preparation of specifications, geotechnical testing, design analysis, construction techniques, quality control, performance criteria, investigation and failure analysis, and repair techniques for light foundations; for the benefit of the public.
e) To elevate the standards and ethical concepts of those engaged in the light foundation industry.
f) To cooperate and share with other related professions engaged in related services information on the science of residential design, construction, and repair of light foundations

A slate of officers was elected which included Jack Deal as President, David Eastwood as Vice-President, Richard Peverley as Secretary/Treasurer, and Joe Edwards as Parliamentarian. Application forms have been mailed to some individuals who have participated in our past activities. Application forms were made available at this seminar for any others who wished to join our organization.

Our first official meeting of the Committee will be at the end of the 1996 seminar. The first order of business will be to elect a new President because Jack Deal has resigned because he was leaving the Houston. The Officers elected were David Eastwood as President, Richard Peverley as President Elect and Secretary/Treasurer, Joe Edwards, Michael Skoller, and Dan Jaggers as Board Members. Other business was also conducted.

At the beginning of the 1997/1997 year, elections were again held and the Officers elected included Richard Peverley as President, Michael Skoller as President Elect, and Ed Kile as Secretary/Treasurer. The Board of Directors included David Eastwood, Dan Jaggers, Ann Nelson, and David Grissom. During the 1997/1998 year, a significant amount of effort was expended to expand the work of the Subcommittees. The first subcommittee document to be published was issued by the Inspection and Assessment Subcommittee, under Don Lenert, PE. It was titled "Criteria for the Inspection of and the Assessment of Residential Slab-on-Ground Foundations." and was numbered FPC 201-97. The document was first issued for review and comment. After most of the comments were resolved, the document was issued for public use. In the fall of 1997, Texas Board of Professional Engineers formed a committee of Engineers to establish standards for the design, inspection and repair of residential foundations. David Eastwood, Bob Newman, and the author participated in this activity. The FPC 201-97 document was used by this committee. The Board has issued a policy document which is currently out for public review. It may possibly be issued in late November. There will be two papers on this subject in the 1998 Symposium. The following individuals were speakers at our 1997/1998 monthly meetings:

- Messrs Leonard Meyer and Jack Rose, Attorneys - Recent changes in the law which affect the residential foundation industry.
- Mr. Dean Parker of Pro-Chemical of Texas who spoke on chemical soil stabilization.
- Mr. Michael Turner of Sure Void of Colorado on void cartons.
- Messrs David Eastwood & Dick Peverley on the Engineering Board requirements for foundation designs and inspections.
- Dr. Karl Norman of the University of Houston who discussed geological faulting in the greater Houston area.
- Mr. Jack Spivery who provided a status report on the Inspection Subcommittee.

The annual meeting of the Foundation performance Committee was held in October. The new officers were elected included Michael Skoller as President, Joe Edwards As President-Elect, and Ed Kile as Secretary/Treasurer. Board Members include David Eastwood, Richard Peverley, Dan Jaggers, an Ann Nelson.

As the departing President, I wish to thank each and every person who participated in the Committee activities for making my tenure both enjoyable and successful. This organization is indeed beneficial to its members and to the public. I urge any one in the residential foundation business or associated with it in any way to join and help promote the committee's activities

Richard W. Peverley, PE

## NOAH'S ARK -- IF IT HAPPENED TODAY

The Lord gave Noah design plans and six months to build an Ark before the great flood, but after the time had passed and the rain began to fall, the Lord saw that Noah was sitting in his front yard, weeping -- and there was no Ark.

"Lord, please forgive me!" begged Noah. "I did my best. But there were big problems! I had to get a building permit for the Ark construction project, and your plans didn't met code. So I had to hire an engineer to re-draw the plans. I got into a big fight over whether or not the Ark needed a fire sprinkler system. My neighbor objected, claiming I was violating zoning by building the Ark in my front yard, so I had to get a variance from the city planning commission. I had problems getting enough wood for the Ark because there was a ban on cutting trees to save the Spotted Owl. I tried to explain there'd be no owls at all if I didn't get that wood, but no dice. Then the carpenters formed a union and went on strike! Then I started gathering up animals and I got sued by an animal rights group that objected to me taking only two of each kind. Just when I got the suit dismissed, EPA notified me that I couldn't complete the Ark without filing an environmental impact statement on your proposed flood. The Army Corps of Engineers wanted a map of the proposed new flood plain, so I sent them a globe. Just recently, the IRS seized all my assets, claiming I'm trying to avoid paying taxes by leaving the country. And I just got a notice from the state about owing them some kind of use tax. At this rate, I really don't think I can finish the Ark for at least another five years!"

Suddenly, the sky began to clear. A rainbow arched across the sky. Noah looked up and smiled. "You mean you're not going to destroy the earth?" Noah asked hopefully. "No need," the Lord said. "The job's already done."

-- *Gleaned from the Internet*

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## LEGAL ISSUES

## The RCLA as It Applies to Foundation Work

Mr. Jack Rose

# RESIDENTIAL FOUNDATIONS AND THE R.C.L.A.

**John C. (Jack) Rose**
**General Counsel**
**BTH, Inc. d/b/a Brighton Homes**
**13101 Northwest Freeway, Suite 312**
**Houston, Texas 77040**
**(713) 460-0264**

**Prepared for:**

**Foundation Performance Committee**
**Houston Engineering Society**
**Houston, Texas**
**November 5, 1998**

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# RESIDENTIAL FOUNDATIONS AND THE R.C.L.A.

JOHN C. (JACK) ROSE

***"There are two types of foundations; those that have failed, and those that are going to fail."***

## I. INTRODUCTION

While this old saying is far from true, foundation disputes do make up the single most prevalent category of residential construction litigation. All homeowners fear having a foundation problem, and all homebuilders fear receiving notice of a foundation claim. This is because foundation disputes are difficult to evaluate and expensive to fix. Due to these uncertainties they often lead to litigation.

Once a foundation dispute becomes a lawsuit it becomes even more difficult to settle. Homeowners and homebuilders alike often dig in their proverbial heels and spend tens of thousands of dollars on hiring attorneys and foundation experts often with mixed results. It is not uncommon for homeowners to recover only a fraction of the cost of repair their foundation and for the homebuilder to spend more money in attorneys fees and costs than it costs to repair the foundation in the first place. Regardless of who wins, everyone loses.

The vast majority of foundation claims are brought by homeowners against the new homebuilders who constructed their home. This is the type of dispute covered in this discussion. However, the term "homebuilder" can apply to any residential construction contractor in almost all of the situations described in this paper.

## II. THE BEGINNING OF A FOUNDATION CLAIM

### A. What are those cracks?

Most foundation disputes begin when a home owner notices cracks in the sheetrock or trim inside the home. Many times these cracks are simply part of the normal "settlement" of the home, once the temperature inside the home is stabilized and the wood framing dries. However, if these cracks continue to appear, it may be evidence of a more significant problem. Homeowners also may notice cracks in the exterior brick, gaps in the expansion joint in the brick facade, and in some cases even separation of the breezeways between the home and a detached garage.

Upon noticing these problems, homeowners usually contact the customer service or warranty departments of their builders to request that they be repaired. Many times sheetrock cracks and other items are repaired and no further action is taken. However, if the cracks reappear and grow larger, the home owner may feel that there is a foundation problem occurring with his home.

The home owner then usually contacts an independent engineer to determine the cause of the cracks or other problems. Most qualified engineers inspect the home looking for clues indicating the cause of the symptoms being noticed by the homeowner. Many also take measurements of the relative elevations of various points on the foundation to determine whether the foundation is level. While there is no set standard regarding what is acceptable, it is commonly held that a variation of three inches (3") or less across the span of a foundation is acceptable. Anything more indicates significant foundation movement and possible damage.

### B. What are you going to do about it?

If the engineer's report indicates that there is some possible foundation damage, the homeowner typically sends a copy of the report the homebuilder and demands that the foundation be repaired or sends it to an attorney who typically demands that the homebuilder pay hundreds of thousands of dollars to the homeowner for his damages and his attorneys fees. Either way, the stage is set for what may be a long and costly legal battle.

## III. APPLICATION OF THE RCLA

To address construction disputes like those involving foundations and the inherent difficulties in litigating them, the Texas legislature has enacted Chapter 27 of the Texas Property Code, known as the Residential Construction Liability Act (RCLA). The purpose of the RCLA is to help homeowners, homebuilders and residential contractors settle construction disputes before they become lawsuits. It serves this purpose by setting up a procedure for each party to follow when a residential construction dispute arises. It also establishes a time frame during which a homeowner and homebuilder must respond to one another during the resolution process. Both parties receive significant benefits if they follow the procedure and both can incur substantial penalties if they do not.

The RCLA applies to "any action to recover damages resulting from a construction defect,..." Tex. Prop. Code Ann. §27.002 (Vernon Supp. 1996). A "construction defect" means a matter concerning the design, construction, or repair of a new residence, of an alteration of or addition to an existing residence, or of an appurtenance to a residence. Id at §27.001. In almost all foundation related claims brought by homeowners an allegation is made that there is a defect in the design, construction or repair of their home. Those allegations trigger the application of the RCLA.

### A. The Certified Letter

On the sixtieth (60th) day before filing suit against a contractor, a homeowner must give written notice by certified mail to the contractor specifying in reasonable detail the construction defects that are the subject of the complaint. There is no set form for this notice other than it must be sent via certified mail, return receipt requested. It does not have to mention the RCLA. It can be a letter or even a handwritten memo or repair request.

If a homebuilder receives any certified letter from a homeowner which in any way involves the construction of his home, the builder should treat it as an RCLA notice letter. For example, if a homebuilder receives a warranty claim through its customer service department or even if one is sent to a construction superintendent and it is sent by certified mail, always treat it as though it were an RCLA notice from an attorney. It just might be one. Sometimes clever lawyers will have homeowners write a handwritten note to the construction superintendent requesting repairs which is then sent via certified mail to the homebuilder which ends up being considered by a court as an actual RCLA notice. Again, there is no set form for an RCLA notice letter. A letter from the homeowner does not need to even mention that the home owner is making a claim for a construction defect.

### B. The Inspection

During the next thirty-five (35) days after the builder receives the notice from the homeowner, the builder may request a reasonable opportunity to inspect the property which is the subject of the complaint. The request must be in writing. The purpose of the inspection is to determine the nature and the cause of the defect and the extent of repair necessary to fix it.

The contractor should <u>always</u> request and conduct an inspection. Even if the construction superintendent or other personnel have previously looked at the problem, it is still important to take this opportunity to inspect the property for any changes in the home's condition, and even more important to document the inspection through the use of photographs and video tape including evidence of sheetrock cracks, nail pops, brick separation, etc. This will assist in establishing a point of reference for any future foundation movement.

Also, ask questions of the home owner as to when the cracks began to appear, and whether they have grown larger. The home owner must provide notice of the construction defects the subject of the complaint in reasonable detail. It is also important for the builder to notify insurance companies and attorneys that an RCLA notice has been received.

### C. The Offer

Then, within forty-five (45) days after the builder receives the notice, the builder may make a written offer of settlement. The offer can be either an agreement by the builder to repair the home, an agreement to have an independent contractor repair the home at the builder's expense, or an offer of money to settle the claim. The offer should always address each claim mentioned by the homeowner in the notice letter even if no repair or money is being offered. The offer should also include an offer to pay some amount of money for the reasonable attorneys' fees incurred by the homeowner up to the time the offer is made. It is important for the builder to document what was found during the inspection and the reasons for either offering or failing to offer a settlement.

The hardest part for many builders is to determine what, if anything, to offer to settle a foundation claim. It is certainly true that many builders can have the repair work done by their chosen contractors at a cost far less than what it would cost the homeowner to hire an independent contractor to perform the repairs. However, this means that the builder and the homeowner will continue to have close contact during the repair process. This process may take as little as a few days, or as much as several months.

### D. The Sensitive Homeowner

Many home owners are not aware of various methods commonly used to repair foundations such as the installation of interior or perimeter piers under a foundation, epoxy injections, or the cosmetic repairs necessary after these types of work are done. If the homeowners will not tolerate the inconveniences these repairs may cause, the homebuilder should consider carefully whether to perform the repairs itself.

Sometimes it may be appropriate for a builder to offer to buy back a home, repair it themselves, and then place the home back on the market with the appropriate disclosures. This puts the home owner back to where he was before purchasing the home, and simplifies the repair process for the builder. Finally, the builder can simply offer a cash settlement.

### E. It Pays To Be Reasonable

In any case, the builder should <u>always</u> make an offer of settlement, even if it is no more than offering a couple hundred dollars for reasonable attorneys' fees. Even a limited offer may help the homebuilder limit its damages if the case is ultimately heard by a jury. Pursuant to the RCLA a homeowner may recover only the following damages caused by a construction defect:

1. The reasonable cost of repairs necessary to cure any construction defect that the contractor failed to cure;

2. The reasonable expenses of temporary housing reasonably necessary during the period;

3. The reduction of market value, if any, to the extent the reduction is due to structural failure;

4. Reasonable and necessary attorneys fees.

The RCLA also limits the total damages awarded to the homeowner to the purchase price of the residence.

These are extremely important limitations. If the RCLA is found not to apply to a particular claim because a builder failed to make a reasonable offer to repair, other applicable statutes, including the Texas Deceptive Trade Practices-Consumer Protection Act may allow for the doubling or tripling of some damages incurred by the homeowner. Other statutes may also permit recovery of damages for mental anguish not found under the RCLA.

If the contractor fails to make a reasonable offer under the RCLA, or fails to make a reasonable attempt to complete the repairs specified in the accepted offer, or to complete the repairs in a good and workmanlike manner, the limitations on damages and defenses to liability available to the contractor under the RCLA will <u>not</u> apply.

## F. The Homeowner's Response

Once an offer is made, the homeowner may accept it, reject it, or do nothing. If the home owner fails to accept the offer before the twenty-fifth (25th) day after the date the settlement offer is received, then the offer is considered rejected. Once an offer is rejected, either in writing or by failing to accept the written offer of the contractor and a lawsuit is filed by the homeowner, the builder should always file an Affidavit certifying the rejection of the settlement offer. By doing so, the contractor ensures that either the judge or the jury will consider whether the homeowner unreasonably rejected the offer of settlement made by the homebuilder. If the homeowner is found to have unreasonably rejected the offer, he may only recover his attorneys fees incurred before the offer was rejected. Further, this evidence will go a long way to show the judge or the jury that the homebuilder has acted reasonably in dealing with the homeowner. This type of "good faith" may prove valuable as the case progresses.

## IV. TYPICAL FOUNDATION CLAIMS AND DEFENSES

There are several construction issues specific to foundation problems. Below is a brief description of some of these construction issues. Each of them individually or in combination has formed the basis of an alleged construction defect lawsuit filed in and around Harris County, Texas.

### A. PSI

A foundation's PSI, or pounds per square inch rating, is a measure that determines the foundation's ability to withstand the forces exerted against it by gravity, soil and the loads that it must bear. Most foundations in the Houston area are either rated at 2500 PSI or 3000 PSI. Some jurisdictions, such as the City of Friendswood, require no less than 3000 PSI concrete be used in residential foundations. In some foundation lawsuits, a core sample is taken by a qualified concrete testing firm to indicate the actual PSI of the foundation. If the PSI is found to be lower than acceptable levels, the foundation will be determined to be improperly constructed. Moisture content, the concrete mixture and other factors all affect the PSI rating of a concrete foundation.

### B. Steel Reinforcement

Conventional slabs on grade have customarily utilized some type of steel reinforcement to ensure the rigidity of the foundation. Post-tension foundations also use a variation of this approach to insure proper foundation performance. This reinforcement is usually in the form of wire mesh or steel rebar. While the steel rebar is preferable, wire mesh is still commonly used. A design of a foundation usually requires a certain type of reinforcement be used and dictates the actual placement of the reinforcement. For example, if wire mesh is placed too close to the bottom or the top of the foundation slab, it may do little to reinforce its rigidity. Some firms have used x-rays to determine the extent and location of steel or wire mesh reinforcement. However, these types of measurements have yet to be widely accepted.

### C. Design

The design of the foundation is another issue commonly debated in foundation lawsuits. The engineer's stamp is usually some indication to the homeowner and the builder that it was designed using commonly accepted design criteria. Such criteria has been established by the Post-Tension Institute (PTI), and other industry organizations. If these criteria are not followed, many home owners allege that the foundation is improperly designed.

**D. Inadequate Soil Investigation**

Soil investigation is a crucial factor to determine the suitability of a foundation for a residential structure. Soils in and around Houston are considered very active and commonly have plasticity index ratings ranging from the low 20's to the high 60's. In these types of conditions it is important for a builder or contractor to follow the recommendations of a qualified soils engineer in the construction of the residential foundation. Failure to do so can result in unexpected movement of the foundation and foundation damage. However, sometimes developers will place inadequate or unacceptable fill on the site after the soil investigation has been completed. Inadequate soil investigation and use of improper fill are common bases used by homeowners to allege improper foundation construction.

**E. Defenses**

The RCLA also sets forth various defenses available to a builder or contractor. These defenses are underutilized and often ignored in the defense of claims. Under the RCLA, a homebuilder is not liable for any damages caused by:

1. The negligence of a person other than the contractor or agent;

2. Failure of a person other than the contractor to take reasonable action and mitigate damages or to maintain the residence;

3. Normal wear and tear and deterioration;

4. Normal shrinkage due to drying or settlement of construction components within the tolerance of building standards.

In many cases, foundation damage claims are brought many years after the home was constructed. Sometimes a swimming pool has been installed and landscaping alterations have been made. These types of alterations have an impact on the moisture content of the soil around a foundation, and may cause much of the foundation damage exhibited alleged by the home owner. Unfortunately, many builders do not investigate evidence of these defenses thoroughly enough.

# Property Code

CHAPTER 27. RESIDENTIAL CONSTRUCTION LIABILITY

Sec. 27.001. Definitions.

In this chapter:

(1) "Appurtenance" means any structure or recreational facility that is appurtenant to a residence but is not a part of the dwelling unit.

(2) "Construction defect" means a matter concerning the design, construction, or repair of a new residence, of an alteration of or addition to an existing residence, or of an appurtenance to a residence, on which a person has a complaint against a contractor. The term "construction defect" may include any physical damage to the residence, any appurtenance, or the real property on which the residence and appurtenance are affixed proximately caused by a construction defect.

(3) "Contractor" means a person contracting with an owner for the construction or sale of a new residence constructed by that person or of an alteration of or addition to an existing residence, repair of a new or existing residence, or construction, sale, alteration, addition, or repair of an appurtenance to a new or existing residence. The term "contractor" also includes a risk retention group registered under Article 21.54, Insurance Code, that insures all or any part of a contractor's liability for the cost to repair a residential construction defect.

(4) "Residence" means a single-family house, duplex, triplex, or quadruplex or a unit in a multiunit residential structure in which title to the individual units is transferred to the owners under a condominium or cooperative system.

(5) "Structural failure" means actual physical damage to the load-bearing portion of a residence caused by a failure of the load-bearing portion.

Added by Acts 1989, 71st Leg., ch. 1072, Sec. 1, eff. Sept. 1, 1989. Amended by Acts 1993, 73rd Leg., ch. 797, Secs. 1, 2 eff. Aug. 30, 1993.

Sec. 27.002. Application of Chapter.

(a) This chapter applies to any action to recover damages resulting from a construction defect, except a claim for personal injury, survival, or wrongful death or for damage to goods. To the extent of conflict between this chapter and any other law, including the Deceptive Trade Practices-Consumer Protection Act (Subchapter E, Chapter 17, Business & Commerce Code), this chapter prevails.

(b) In this section:

(1) "Goods" does not include a residence.

(2) "Personal injury" does not include mental anguish.

Added by Acts 1989, 71st Leg., ch. 1072, Sec. 1, eff. Sept. 1, 1989. Amended by Acts 1993, 73rd Leg., ch. 797, Sec. 3, eff. Aug. 30, 1993.

Sec. 27.003. Liability.

(a) In an action to recover damages resulting from a construction defect, a contractor is not liable for any percentage of damages caused by:

(1) negligence of a person other than the contractor or an agent, employee, or subcontractor of the contractor;

(2) failure of a person other than the contractor or an agent, employee, or subcontractor of the contractor to:

(A) take reasonable action to mitigate the damages; or

(B) take reasonable action to maintain the residence;

(3) normal wear, tear, or deterioration;

(4) normal shrinkage due to drying or settlement of construction components within the tolerance of building standards; or

(5) the contractor's reliance on written information relating to the residence, appurtenance, or real property on which the residence and appurtenance are affixed that was obtained from official government records, if the written information was false or inaccurate and the contractor did not know and could not reasonably have known of the falsity or inaccuracy of the information.

(b) Except as provided herein, this chapter does not limit or bar any other defense or defensive matter or other defensive cause of action applicable to an action to recover damages resulting from a construction defect.

Added by Acts 1989, 71st Leg., ch. 1072, Sec. 1, eff. Sept. 1, 1989. Amended by Acts 1993, 73rd Leg., ch. 797, Sec. 4, eff. Aug. 30, 1993.

Sec. 27.004. Notice and Offer of Settlement.

(a) Before the 60th day preceding the date a claimant seeking from a contractor damages arising from a construction defect files suit, the claimant shall give written notice by certified mail, return receipt requested, to the contractor, at the contractor's last known address, specifying in reasonable detail the construction defects that are the subject of the complaint. During the 35-day period after the date the contractor receives the notice, and on the contractor's written request, the contractor shall be given a reasonable opportunity to inspect and have inspected the property that is the subject of the complaint to determine the nature and cause of the defect and the nature and extent of repairs necessary to remedy the defect. The contractor may take reasonable steps to document the defect.

(b) Within the 45-day period after the date the contractor receives the notice, the contractor may make a written offer of settlement to the claimant. The offer may include either an agreement by the contractor to repair or to have repaired by an independent contractor at the contractor's expense any construction defect described in the notice and shall describe in reasonable detail the kind of repairs which will be made. The repairs shall be made within the 45-day period after the date the contractor receives written notice of acceptance of the settlement offer, unless completion is delayed by the claimant or by other events beyond the control of the contractor. For the purposes of this section, "independent contractor" means a person who is independent of the contractor and did not perform any of the work complained of in the claimant's notice. The claimant and the contractor may agree in writing to extend the periods described by this subsection.

(c) If the giving of the notice under Subsections (a) and (b) within the period prescribed by those subsections is impracticable because of the necessity of filing suit at an earlier date to prevent expiration of the statute of limitations or if the complaint is asserted as a counterclaim, that notice is not required. However, the suit or counterclaim shall specify in reasonable detail each construction defect that is the subject of the complaint, and the inspection provided for by Subsection (a) may be made during the 60-day period following the date of service of the suit or counterclaim on the contractor, and the offer provided for by Subsection (b) may be made within the 60-day period following the date of service. If, while a suit subject to this chapter is pending, the statute of limitations for the cause of action would have expired and it is determined that the provisions of Subsection (a) were not properly followed, the suit shall be abated for up to 75 days in order to allow compliance with Subsections (a) and (b).

(d) The court shall abate a suit governed by this section if Subsection (c) does not apply and the court, after a hearing, finds that the contractor is entitled to an abatement because notice was not provided as required by Subsection (a). A suit is automatically abated without the order of the court beginning on the 11th day after the date a plea in abatement is filed if the plea in abatement:

(1) is verified and alleges that the person against whom the suit is pending did not receive the written notice as required by Subsection (a); and

(2) is not controverted by an affidavit filed by the claimant before the 11th day after the date on which the plea in abatement is filed.

(e) An abatement under Subsection (d) continues until the 60th day after the date that written notice is served in compliance with Subsection (a).

(f) If a claimant unreasonably rejects an offer made as provided by this section or does not permit the contractor or independent contractor a reasonable opportunity to repair the defect pursuant to an accepted offer of settlement, the claimant may not recover an amount in excess of the reasonable cost of the repairs which are necessary to cure the construction defect and which are the responsibility of the contractor and may recover only the amount of reasonable and necessary attorney's fees and costs incurred before the offer was rejected or considered rejected.

(g) If a contractor fails to make a reasonable offer under this section, or fails to make a reasonable attempt to complete the repairs specified in an accepted offer made under this section, or fails to complete, in a good and workmanlike manner, the repairs specified in an accepted offer made under this section, the limitations on damages and defenses to liability provided for in this section shall not apply.

(h) Except as provided by Subsection (f), in a suit subject to this chapter the claimant may recover only the following damages proximately caused by a construction defect:

(1) the reasonable cost of repairs necessary to cure any construction defect that the contractor failed to cure;

(2) the reasonable expenses of temporary housing reasonably necessary during the repair period;

(3) the reduction in market value, if any, to the extent the reduction is due to structural failure; and

(4) reasonable and necessary attorney's fees.

(i) The total damages awarded in a suit subject to this chapter may not exceed the claimant's purchase price for the residence.

(j) An offer of settlement made under this section that is not accepted before the 25th day after the date the offer is received by the claimant is considered rejected.

(k) An affidavit certifying rejection of a settlement offer under this section may be filed with the court. The trier of fact shall determine the reasonableness of a rejection of an offer of settlement made under this section.

(l) A contractor who makes or provides for repairs under this section is entitled to take reasonable steps to document the repair and to have it inspected.

(m) Notwithstanding Subsections (a), (b), and (c), a contractor who receives written notice of a construction defect resulting from work performed by the contractor or an agent, employee, or subcontractor of the contractor and creating an imminent threat to the health or safety of the inhabitants of the residence shall take reasonable steps to cure the defect as soon as practicable. If the contractor fails to cure the defect in a reasonable time, the owner of the residence may have the defect cured and may recover from the contractor the reasonable cost of the repairs plus attorney's fees and costs in addition to any other damages recoverable under any law not inconsistent with the provisions of this chapter.

(n) This section does not preclude a contractor from making a monetary settlement offer.

(o) The inspection and repair provisions of this chapter are in addition to any rights of inspection and settlement provided by common law or by another statute, including Section 17.505, Business & Commerce Code.

Added by Acts 1989, 71st Leg., ch. 1072, Sec. 1, eff. Sept. 1, 1989. Amended by Acts 1993, 73rd Leg., ch. 797, Sec. 5, eff. Aug. 30, 1993; Acts 1995, 74th Leg., ch. 414, Sec. 10, eff. Sept. 1, 1995.

Sec. 27.005. Limitations on Effect of Chapter.

This chapter does not create an implied warranty or extend a limitations period.

Added by Acts 1989, 71st Leg., ch. 1072, Sec. 1, eff. Sept. 1, 1989.

Sec. 27.006. Causation.

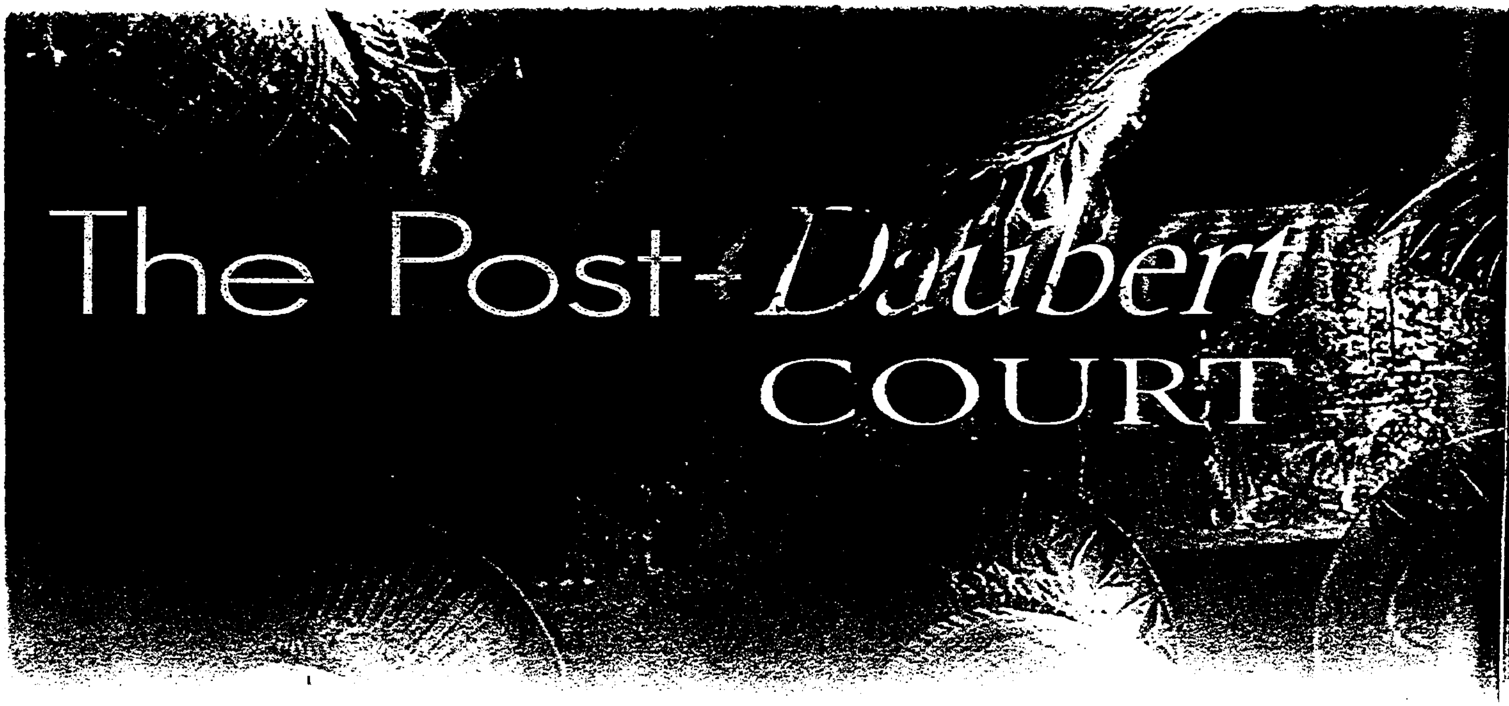
In an action to recover damages resulting from a construction defect, the claimant must prove that the damages were proximately caused by the construction defect.

Added by Acts 1993, 73rd Leg., ch. 797, Sec. 6, eff. Aug. 30, 1993

# LEGAL ISSUES

## A Review of Construction Litigation Cases

Mr. Leonard J. Meyer



# The Post-*Daubert* COURT

*By Sofia Adrogué*

## INTRODUCTION

The use of experts has generated controversy for centuries. Virtually 140 years ago, the Supreme Court, through Justice Grier, in *Winans v. New York & Erie R.R. Co.*,[1] expressed its displeasure at the proliferation of expert testimony in federal trials:

> Experience has shown that opposite opinions of persons professing to be experts may be obtained to any amount; and it often occurs that not only many days, but even weeks, are consumed in cross-examinations, to test the skill or knowledge of such witnesses and the correctness of their opinions, wasting the time and wearying the patience of both court and jury, and perplexing, instead of elucidating, the questions involved in the issue.

Writing for the Fifth Circuit in 1986, Judge Higginbotham, in *In re Air Crash Disaster at New Orleans, La.*,[2] articulated frustration with experts "for hire": "Our message to our able trial colleagues: it is time to take hold of expert testimony in federal trials."[3] Judge Weinstein of the Eastern District of New York advocated a similar view when he stated: "An expert can be found to testify to the truth of almost any theory, no matter how frivolous."[4]

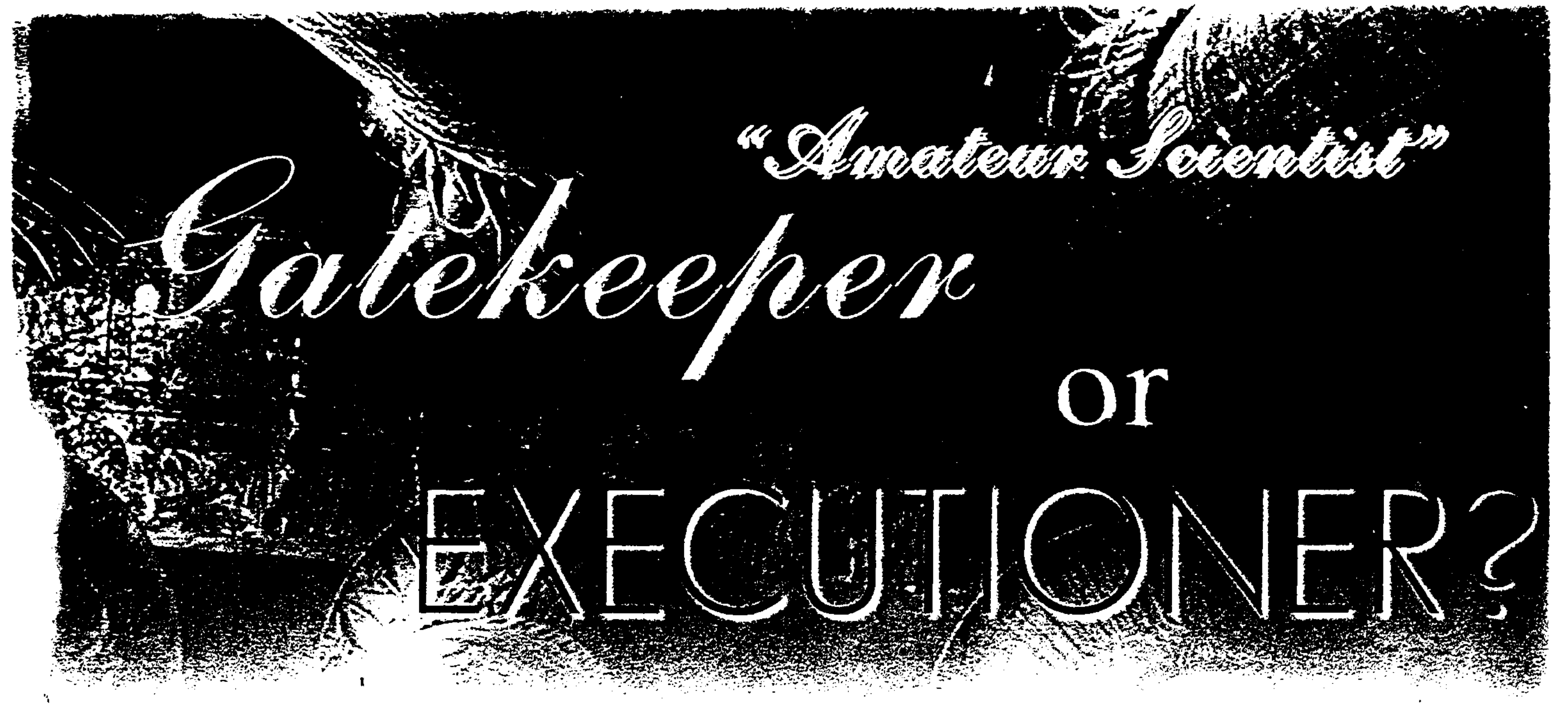
In *Daubert v. Merrell Dow Pharmaceuticals, Inc.*,[5] the United States Supreme Court heeded the admonitions of Justice Grier and Judges Higginbotham and Weinstein, rejected the Frye general acceptance test, and declined to adhere to the traditional protocol of "nose counting." Instead, the Court ostensibly opted to broaden the parameters of admissible scientific evidence to include novel opinions. As a result, under *Daubert* and its progeny, federal courts may not wait for the magical moment when a scientific principle or discovery "crosses the line between the experimental and demonstrable stages."[6] A district court judge must serve as the gatekeeper.

Its own brethren have admitted to the difficulty that emerges with the judge as gatekeeper. The Ninth Circuit on remand in *Daubert II*,[7] stated that "[f]ederal judges ruling on the admissibility of expert scientific testimony face a far more complex and daunting task in a post-*Daubert* world than before."[8] Justice Rehnquist, concurring in part, dissenting in part, in *Daubert I*, predicted as such, stating that the Court has forced trial judges to become "amateur scientists."[9]

*Daubert* expressly limited its discussion of Fed. R. Civ. Evid. 702 to the "scientific context" because that was the "nature of the expertise" at issue.[10] However, Rule 702 also applies to technical or other specialized knowledge, leading to the obvious question of what to do with the non-scientific expert witness. Thus, query whether *Daubert* extends beyond "scientific" evidence? For many courts, the answer appears a resounding yes.

The Fifth Circuit recently addressed this issue in *Watkins v. Telsmith, Inc.*,[11] a case in which the plaintiff alleged the improper application of *Daubert* to exclude the expert testimony as unqualified due to his training in civil engineering in contrast to mechanical engineering. The Fifth Circuit, with a panel comprised of Judges Jolly, Jones, and Wiener, found no abuse of discretion by the trial court and affirmed. Discussed in detail below, the Fifth Circuit reviewed other circuit opinions as to whether *Daubert* is limited to novel scientific techniques or mythologies, and agreeing with the rationale employed by the Seventh and Eighth Circuits, found the *Daubert* "criteria equally applicable to 'technical, or other specialized knowledge.'"[12] According to the Fifth Circuit, "[w]hether the expert would opine on economic valuation, advertising psychology, or engineering, application of the *Daubert* factors is germane to evaluating whether the expert is a hired gun or a person whose opinion in the courtroom will withstand the same scrutiny that it would among his professional peers."[13]

Post-*Daubert* jurisprudence has generated more confusion and a greater potential for the execution of one's expert as the judge seeks to reject the "dross" and retain the "pure and sound and fine."[14] The newly inaugurated Mealey's *Daubert* Report advertises that little is certain about this controversial ruling but that as a trial lawyer, you simply cannot ignore it. *Daubert* was ostensibly to assist with the ongoing controversy over "junk science," "hired gun experts," etc. The opinion, however, arguably contains few bright line tests that most experts pass. On the other hand, it does include plenty of quotable language to



# "Amateur Scientist" Gatekeeper or EXECUTIONER?

support almost any position.[15] Lawyers trying to exclude evidence should emphasize the vague "helpfulness" requirement and argue that the expert's method is so strained that it is not helpful. Lawyers urging admission of expert testimony should emphasize that the Texas Supreme Court, for instance, in at least one case, rejected general acceptance and expressed confidence in juries.

## *DAUBERT, ROBINSON AND MERRELL DOW*

The appropriate standard for determining the admissibility of scientific expert testimony is an issue that has long divided the federal courts.

### *DAUBERT*

In *Daubert v. Merrell Dow Pharmaceuticals, Inc.*,[16] the Supreme Court held that the Federal Rules of Evidence determined when expert testimony is admissible. The Court observed that Federal Rule of Evidence 702, in particular, places limits on the admissibility of scientific testimony.[17] Rule 702's reference to an expert's "scientific" knowledge suggests that the evidence must be based on "good science."[18] Rule 702's "helpfulness" requirement (evidence must help the jury to understand complicated issues) refers to materiality.[19] Of course, Rule 403 (probity vs. prejudice) further provides some limits on truly misleading "junk" science. In short, under the Rules, the trial judge must ensure that scientific testimony or evidence admitted is not only relevant, but reliable.[20]

Justice Blackmun identified four factors that a court should consider in determining whether the scientific reasoning or methodology underlying an expert's opinion is scientifically valid under Federal Rule of Evidence 702:

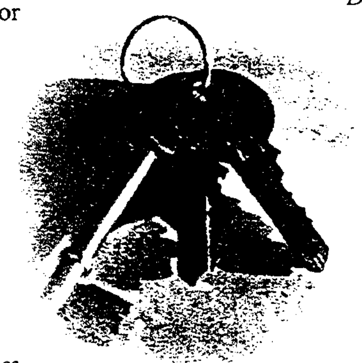
1. Whether the expert's theory or technique "can be (and has been) tested."[21]
2. "[W]hether the theory or technique has been subjected to peer review and publication."[22] While not a sine qua non of admissibility, "[t]he fact of publication (or lack thereof) in a peer reviewed journal thus will be a relevant, though not dispositive, consideration in assessing the scientific validity of a particular technique or methodology on which an opinion is premised."[23]
3. What the known or potential "rate of error" is for any test or scientific technique that has been employed and "the existence and maintenance of standards controlling the technique's operation."[24]
4. Whether the technique is generally accepted. "A 'reliability assessment does not require, although it does permit, explicit identification of a relevant scientific community and an express determination of a particular degree of acceptance within that community. . . Widespread acceptance can be an important factor in ruling particular evidence admissible, and a known technique which has been able to attract only minimal support within the community,' may properly be viewed with skepticism."[25]

On remand, the Ninth Circuit in *Daubert II* added the factor of "independence" concerning whether the testimony relates to matters growing naturally or directly out of research or whether the opinions were developed expressly for purposes of litigation.[26] In *E.I. du Pont de Nemours and Co. v. Robinson*,[27] the Texas Supreme Court articulated this issue as "the non-judicial uses which have been made of the theory or technique."[28] Independent research results are less likely to be biased and provide "important, objective proof that the research comports with the dictates of good science."[29] When this factor is not met, the proponent may overcome this prejudice by pointing to an objective source to demonstrate that the scientific method employed is practiced by at least a recognized minority of scientists in their field.[30]

### *ROBINSON*

Whereas *Daubert*'s starting point was the text of the Federal Rules of Evidence, *E.I. du Pont De Nemours & Co. v. Robinson*[31] began with the proposition that expert testimony is a threat to the justice system. To this end, the Texas Supreme Court stated: "Professional expert witnesses are available to render an opinion on almost any theory, regardless of its merit."[32] According to the Court, "[b]ecause expert evidence can be hard to evaluate, it can be both powerful and misleading."[33] While the Court expressed doubts about a jury's ability to identify even the most egregious abuses of expert testimony, the Court concluded, that "[j]udges are capable of understanding and evaluating scientific reliability" and "[do] not have to be trained in science to evaluate the reliability of a theory or technique."[34]

In analyzing the admissibility of "scientific" expert testimony, the Texas Supreme Court identified the *Daubert I* and *II* "factors" as the factors a trial court may consider in determining the reliability of expert testimony under Rule 702.[35] The Court similarly emphasized that the lists were both flexible and non-exclusive.[36] Moreover, the United States and Texas Supreme Courts stated that the scrutiny to be applied should be on the principles and methodology used by the expert, not the conclusions reached, which are within the province of the jury.[37] The only major difference was the "subjectivity" factor espoused by the Texas Supreme Court. This factor considers the extent to which the technique relies upon the subjective interpretation of the expert.[38] This factor seems quite similar in purpose and effect to the "testability" factor promulgated by the United States Supreme Court.



### *Merrell Dow*

On July 9, 1997, the Texas Supreme Court decided *Merrell Dow Pharmaceuticals, Inc. v. Havner.*[39] The case involved "no evidence" review of a jury verdict in favor of the plaintiffs who alleged that the drug Bendectin caused limb reduction birth defects in their child. Although the Supreme Court decided the case based on sufficiency of the scientific evidence of causation, the Court drew heavily on admissibility decisions such as *Robinson.*[40] According to the Supreme Court, "the same factors may be applied in a no evidence review of scientific evidence."[41]

The plaintiffs called five admittedly well-qualified experts on causation who testified that Bendectin causes limb reduction birth defects, and at least one testified that Bendectin specifically caused Kelly Havner's birth defects. Although the jury found for the plaintiffs, the Court disagreed. "The federal courts have dealt extensively with Bendectin litigation. To date, no plaintiff has ultimately prevailed in federal court... The federal decisions have discussed the substance of the evidence in detail, and often the testimony under scrutiny included that of Drs. Palmer, Newman, Glasser, Gross, and Swan, the Havners' witnesses."[42]

According to the Court, "We have held, however, that an expert's bare opinion will not suffice. . . The substance of the testimony must be considered."[43] This is true even if the expert uses the "magic language'" that the opinion is based on "reasonable medical probability."[44] "If the expert's scientific testimony is not reliable, it is not evidence."[45] "The underlying data should be independently evaluated in determining if the opinion itself is reliable."[46] "While Rule 702 deals with the admissibility of evidence, it offers substantive guidelines in determining if the expert testimony is some evidence of probative value."[47]

Finally, according to the Court, on "no evidence" review, a court can reject expert opinion if it disagrees with the expert's reasoning. Even if the expert's data is reliable, "[a] flaw in the expert's reasoning from the data may render reliance on a study unreasonable and render the inferences drawn therefrom dubious. Under that circumstance, the expert's scientific testimony is unreliable and, legally, no evidence."[48]

## *DAUBERT'S* PROGENY IN THE FIFTH CIRCUIT

In the year after the *Daubert* decision, more than forty appellate decisions cited the case. Only a few reversed the trial court.[49] In the Fifth Circuit, the early cases after *Daubert* almost uniformly affirmed the trial court's ruling.[50]

However, *Daubert* did not create a broad mandate to exclude more expert testimony. Approximately a year after *Daubert*, several federal appellate courts issued opinions reversing the exclusion of expert testimony under *Daubert*. In general, these cases instruct district courts that Daubert did not effect a wholesale change in federal evidence law and did not issue a mandate to exclude expert testimony. Most notably, in the Fifth Circuit, was the decision in *U.S. v. 14.38 Acres of Land*,[51] in which the Fifth Circuit reversed the district court's exclusion of a land appraisal expert, stating that *Daubert* established standards for evaluating reliability, but "did not otherwise work a sea change over federal evidence law," and emphasizing that the alleged weaknesses in the expert's opinion were matters for cross-examination.

Subsequently, the Fifth Circuit applied the *Daubert* test to non-scientific expert testimony in two significant cases.

### *Watkins*

In *Watkins v. Telsmith, Inc.*,[52] the plaintiff alleged that *Daubert* should not have been applied to exclude her expert's testimony. The plaintiff argued that *Daubert* applied only to "scientific knowledge" and expert testimony "based on 'novel' scientific evidence."[53] According to the plaintiff, the case presented no such novelty, "but merely the application of [the expert's] experience and common engineering principles to evaluate the safety of this conveyer and envision alternative designs."[54]

The Fifth Circuit found the trial court properly applied *Daubert*'s principals and did not commit manifest error in excluding the subject testimony for lack of a sufficiently reliable scientific or technical basis and affirmed.[55] The Fifth Circuit reviewed other circuit opinions as to whether *Daubert* is limited to novel scientific techniques or methodologies, and agreeing with the rationale employed by the Seventh and Eighth Circuits, found that *Daubert*'s "focus on a standard of evidentiary reliability and the requirement that proposed expert testimony must be appropriately validated are criteria equally applicable to 'technical, or other specialized knowledge.'"[56] In addition, "the nonexclusive list of factors relevant under *Daubert* to assessing scientific methodology – testing, peer review, and 'general acceptance' – are also relevant to assessing other types of expert evidence."[57]

The Court noted, however, that "[n]ot every guidepost outlined in *Daubert* will necessarily apply to expert testimony based on engineering principles and practical experience, but the district court's 'preliminary assessment of whether the reasoning or methodology underlying the testimony is scientifically valid and of whether that reasoning or methodology properly can be applied to the facts in issue' is no less important."[58] Finally, the Court concluded as follows:

> [W]hether an expert's testimony is based on "scientific, technical or other specialized knowledge," Daubert and Rule 702 demand that the district court evaluate the methods, analysis, and principles relied upon in reaching the opinion. The court should ensure that the opinion comports with applicable professional standards outside the courtroom and that it "will have a reliable basis in the knowledge and experience of [the] discipline." [59]

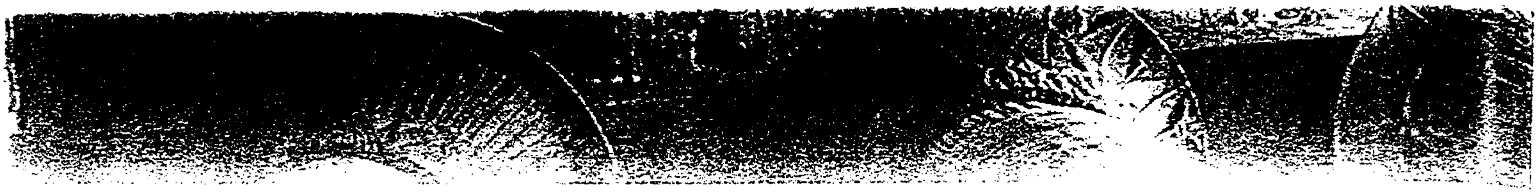
### *Moore*

In *Moore v. Ashland Chemical, Inc.*, [60] the Fifth Circuit cited *Watkins* for the proposition that *Daubert* is not limited to "scientific knowledge" or "novel" scientific evidence. However, the Court stated that the "*Daubert* factors" – empirical testing, peer review and publication, known or potential rate of error, the existence and maintenance of operational standards, and acceptance within a relevant scientific community – which are applicable to "hard science" "generally are not appropriate for assessing the evidentiary reliability of a proffer of expert clinical medical testimony."[61] "Instead, the trial court as gatekeeper should determine whether the doctor's proposed testimony as a clinical physician is soundly grounded in the principles and methodology of his field of clinical medicine."[62] The Court noted, however, that the "*Daubert* factors" may be "relevant and appropriate" in assessing "other types of expert evidence outside the realm of hard science."[63]

However, on November 12, 1997, the Court issued an order granting a rehearing, en banc, on the Court's own motion.[64] The supplemental briefing schedule closed on January 2, 1998. To the author's knowledge, no date has been set yet for oral hearing as of the date this paper was submitted for publication. It remains to be seen what the Fifth Circuit will opine with respect to the application of *Daubert* outside the realm of "scientific knowledge" given the grant of rehearing.

Despite the order for rehearing, the Court issued an amended opinion on November 24, 1997, reported at 1997 U.S. App. LEXIS 33501. The following is a discussion of *Moore* as amended.

In *Moore*, a delivery truck driver brought a negligence action against Ashland Chemical, Inc., *et al.*, alleging that he contracted reactive airways disease as a result of exposure to a mixture of chemical gases on the defendants' premises. The trial court excluded the causation testimony of one of the plaintiff's two clinical physicians, and thereafter entered a take nothing judgment for the plaintiff based upon the jury verdict finding no proximate cause. On appeal, the Fifth Circuit reversed and remanded, finding, *inter alia*, that the trial court's ruling was based on numerous "clearly and manifestly erroneous findings of fact" and errors in applying the law to the facts.[65] Finding that the district court's







erroneous exclusion of the expert's testimony was reversible error, the Court cited to a commentator for the proposition that "simply because a non-scientific expert's testimony touches on evidence that theoretically could be tested by Newtonian science methodology, *Daubert* should not be interpreted so as to permit an advocate to put his or her opponent to the burden of establishing hard scientific reliability-validity upon demand."[66]

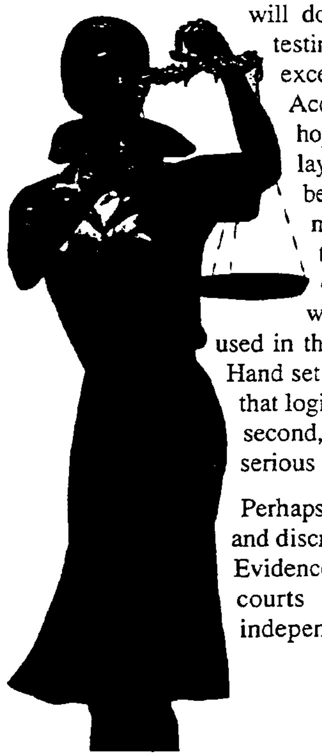
The Court further stated as follows: "*Watkins* . . . explicitly makes clear that Rule 702, as elucidated by *Daubert*, authorizes a qualified expert in a realm outside of hard science to testify to an opinion or inference based on his knowledge, skill, experience, training, or education if it is soundly grounded in the principles and methodology of his discipline and is relevant to a fact in issue or to an understanding of the evidence."[67] According to the Fifth Circuit, "[e]ven prior to *Watkins*, however, this circuit and others had at least implicitly understood this to be part of *Daubert* lore."[68]

Circuit Judge Davis dissented.[69] He commenced his opinion stating that he "thoroughly disagree[d] with the majority's conclusion" and ended with the following cautionary note:

> The Supreme Court has directed the district courts to control with a firm hand expert testimony to prevent litigation abuse so familiar to all of us. The district court took a careful look at Dr. Jenkins' testimony, applied the correct standard, and excluded the testimony. After reviewing the record, I fail to see how the district court could have reached any other conclusion. The majority's "let it all in" view sends exactly the wrong message to conscientious district courts.[70]

## RULE 706 EXPERTS—A PALATABLE SOLUTION

Recognition of the shortcomings of partisan expert testimony is not new. In 1905, Judge Hand wrote of the confusion caused a jury by conflicting expert opinions, concluding that "[the jury] will do no better with the so-called testimony of experts than without, except where it is unanimous."[71] According to Judge Hand, "What hope have the jury, or any other layman, of a rational decision between . . . conflicting statements each based upon [a lifetime of technical] experience."[72] In his consideration of whether expert witnesses were used in the best possible manner, Judge Hand set out to prove two things, "first, that logically the expert is an anomaly; second, that from the legal anomaly serious practical difficulties arise."[73]



Perhaps one answer lies in the sparing and discriminate use of Federal Rule of Evidence 706. Rule 706 grants trial courts the authority to appoint independent experts. In *Daubert*, Justice Blackman specifically ruled that trial courts could assess scientific testimony by appointing experts under Rule 706.[74] Only months after *Daubert*, the Honorable Jack B. Weinstein cited *Daubert* in an opinion concerning the proposed depositions of Rule 706 court-appointed experts and noted that, "[g]iven the trial court's expanded function in evaluating the reliability of expert evidence, it is now more important than ever for the trial court to take an active role in the presentation of expert evidence."[75] However, Rule 706 should not be used as a short cut for trial courts to avoid addressing the complex issues or impairing a party's right to a trial by jury. Another potential answer may be that posited by the Honorable Charles R. Richey – the deletion of references to the word "expert" witness in the Federal Rules of Evidence and the substitution of "opinion" witness.

Nevertheless, until the United States Supreme Court provides further guidance, trial judges will have to function as "amateur scientists" or gatekeepers, hold *in limine* hearings to determine admissibility, or utilize court appointed experts pursuant to Federal Rule of Evidence 706.

## ENDNOTES

1. *Winans v. New York & Erie R.R. Co.*, 62 U.S. (21 How.) 88, 101, 16 L.Ed. 68 (1858).
2. *In re Air Crash Disaster at New Orleans, La.*, 795 F.2d 1230 (5th Cir. 1986).
3. *Id.* at 1234.
4. Jack B. Weinstein, *Improving Expert Testimony*, 20 U. Rich. L. Rev. 473, 478 (1986).
5. *Daubert v. Merrell Dow Pharmaceuticals, Inc.*, 509 U.S. 579, 113 S. Ct. 2786 (1993).
6. 509 U.S. at 585 (citation omitted).
7. *Daubert v. Merrell Dow Pharmaceuticals, Inc.* (*Daubert* II), 43 F.3d 1311 (9th Cir.), *cert. denied*, 116 S. Ct. 189 (1995).
8. *Id.* at 1315.
9. *Daubert, supra* note 5, at 601 (Rehnquist, J., concurring in part and dissenting in part). *See also E.I. du Pont de Nemours & Co. v. Robinson*, 923 S.W.2d 549, 560 (Tex. 1995) (Cornyn, J., dissenting) (accusing the Court of thrusting trial judges into the role of "amateur scientists" and threatening to "invade the jury's province.").
10. *Daubert, supra* note 5, at 590 n.8.
11. *Watkins v. Telsmith, Inc.*, 121 F.3d 984 (5th Cir. 1997).
12. *Id.* at 991.
13. *Id.* (footnotes omitted).
14. *Daubert, supra* note 5, at 597 n.13 (citing Cardozo).
15. Consider the following examples:

    "[A] rigid general acceptance requirement would be at odds with the liberal thrust of the Federal Rules and their general approach of relaxing the traditional barriers to opinion testimony." *Daubert*, 509 U.S. at 588 (quoting *Beech Aircraft Corp. v. Rainey, 488 U.S. 153*, 169 (1988)).

    "That the *Frye* test was displaced by the Rules of Evidence does not mean, however, that the Rules themselves place no limits on the admissibility of purportedly scientific evidence. Nor is the trial judge disabled from screening such evidence. To the contrary, under the Rules the trial judge must ensure that any and all scientific testimony or evidence admitted is not only relevant, but reliable." *Id.* at 589 (footnote omitted).

    "Of course, it would be unreasonable to conclude that the subject of scientific testimony must be known to a certainty; arguably, there are no certainties in science." *Id.* at 590.

    "Proposed testimony must be supported by appropriate validation – i.e., good grounds, based on what is known." *Id.*

    The trial judge must make "a preliminary assessment of whether the reasoning or methodology underlying the testimony is scientifically valid . . . ." *Id.* at 592-93.

    "The inquiry envisioned by Rule 702 is, we emphasize, a flexible one. . . . The focus, of course, must be solely on principles and methodology, not on the conclusions that they generate." *Id.* at 594-95 (footnote omitted).

    "[R]espondent seems to us to be overly pessimistic about the capabilities of the jury and of the adversary system generally. Vigorous cross-examination, presentation of contrary evidence, and careful instruction on the burden of proof are the traditional and appropriate means of attacking shaky but admissible evidence." *Id.* at 596.

    [Note, the author commends Terrell W. Oxford and John M. Helms of Susman Godfrey L.L.P. for the compilation of these quotations.]
16. 509 U.S. 579, 587 (1993).
17. *Id.* at 589-95.
18. *Id.* at 593-95.
19. *Id.* at 591-95.
20. *Id.* at 589-95.
21. *Id.* at 593.
22. *Id.*
23. *Id.* at 594.
24. *Id.* (citations omitted).
25. *Id.* (citations omitted).
26. *Daubert II, supra* note 7, at 1316-317.
27. *E.I. du Pont de Nemours & Co. v. Robinson*, 923 S.W.2d 549 (Tex. 1995).
28. *Id.* at 557 and n. 2 (citing *Daubert II*).
29. *Daubert II, supra* note 7, at 1317.
30. *Id.* at 1319.
31. *E.I. du Pont De Nemours & Co., supra* note 27.
32. *Id.* at 553.
33. *Id.*
34. *Id.* at 553, 557-58.
35. *Id.* at 557.
36. *Id.*
37. *Id.* at 557-58.
38. *Id.* at 557.
39. *Merrell Dow Pharmaceuticals, Inc. v. Havner*, 953 S.W.2d 706 (Tex. 1997).
40. *Id.* at 714.
41. *Id.*
42. *Id.* at 709.
43. *Id.* at 711 (citations omitted).
44. *Id.* at 712.
45. *Id.* at 713.
46. *Id.*
47. *Id.* at 712.
48. Id. at 714.
49. Thomas J. Mack, *Scientific Testimony after Daubert: Some Early Returns from Lower Courts*, Trial 23 (August 1994).
50. *See, e.g., Satcher v. Honda Motor Co.*, 52 F.3d 1311, 1318 (5th Cir. 1995) (rejecting argument that motorcycle experts' testimony was "speculative or lacking in scientific certainty"), *cert. denied*, 116 S. Ct. 705 (1996); *Wheat v. Pfizer, Inc.*, 31 F.3d 340, 343 (5th Cir. 1994) (holding that district court's exclusion of plaintiff's causation expert's testimony was harmless error, but that it would not have met the *Daubert* standard because the effect of the combination of drugs in issue has never been studied or subjected to peer review and publication, "which *Daubert* also identifies as key"); *Carroll v. Morgan*, 17 F.3d 787, 789-90 (5th Cir. 1994) (upholding district court's decision to permit a cardiologist to testify on the cause of death because his method – review of medical and coroner's records and published materials – was grounded in the methods of science); *Marcel v. Placid Oil Co.*, 11 F.3d 563, 567-68 (5th Cir. 1994) (holding that the record was insufficient to support a conclusion that the district court erred by excluding defendants' expert on work life expectations); but see *United States v. Posado*, 57 F.3d 428, 436 (5th Cir. 1995) (vacating and remanding exclusion of polygraph evidence because *Daubert* overruled the Fifth Circuit's per se rule that polygraph tests are inadmissible in criminal cases).
51. *U.S. v. 14.38 Acres of Land*, 80 F.3d 1074, 1077-779 (5th Cir. 1996).
52. *Watkins, supra* note 11.
53. *Id.* at 988.
54. *Id.*
55. *Id.* at 993.
56. *Id.* at 991 (citing Rule 702).
57. *Id.*
58. *Id.* at 990-91 (quoting *Daubert, supra* note 5, at 592-93).

59. *Id.*

60. *Moore v. Ashland Chemical, Inc.*, No. 95-20492, 1997 U.S. App. LEXIS 33501 (5th Cir. Nov. 24, 1997), amending the original opinion dated October 20, 1997, reported at 126 F.3d 679, 1997 U.S. App. LEXIS 28816, *reh'g en banc*, granted, 1997 U.S. App. LEXIS 32349 (5th Cir. Nov. 12, 1997).

61. *Id.* at *23.

62. *Id.* at *28 (citation omitted).

63. *Id.* at *28 n.2.

64. *Moore, supra* note 59.

65. *Id.* at *66-75.

66. *Id.* at *76 (citation omitted).

67. *Id.* at *74-75.

68. *Id.* at *75.

69. *Id.* at *100-20.

70. *Id.* at *100, *120.

71. Learned Hand, *Historical and Practical Considerations Regarding Expert Testimony*, 15 Harv. L. Rev. 40, 56 (1901).

72. *Id.* at 55.

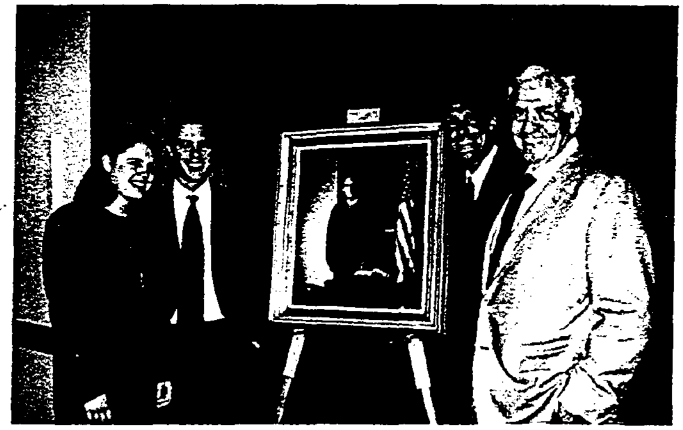
73. *Id.* at 50.

74. *Daubert, supra* note 5, at 595.

75. *In re Joint Eastern & Southern Dists. Asbestos Litig.*, 151 F.R.D. 540, 545 (E.&S.D.N.Y. 1993).

*Sofia Adrogué is of counsel with Susman Godfrey L.L.P. She received her undergraduate degree from Rice University,* magna cum laude, *where she was a member of* Phi Beta Kappa. *She received her law degree from the University of Houston Law Center,* magna cum laude, *and was chief articles editor of the* University of Houston Law Review. *She has served as an adjunct professor, teaching mass tort litigation at the University of Houston Law Center.*

## PORTRAIT UNVEILING



A portrait of the Hon. Judge Eric Andell, former Judge of the 315th Juvenile District Court, was unveiled in a special ceremony on January 23 in the Ceremonial Courtroom. Judge Andell, who now serves on the First Court of Appeals, is joined by his daughter, Bethany, son, John, and father, Phil Andell.

**Patrick CARMICHAEL, Sr., an individual, father and next of kin to Patrick Carmichael, Jr., a minor; Luziminda Carmichael, an individual, mother and next friend of Carina Horn, a minor and administratrix of estates of Janice Horn; Carina Horn, a minor; Leona Carmichael, Shameela Carmichael, Natimah Carmichael, Plaintiffs–Appellants,**

v.

SAMYANG TIRE, INC.; **Hercules Tire Company; Kuhmo, U.S.A.; Kumho & Company, Inc., Defendants–Appellees,**

**Cooper Rubber and Tire Company, Ford Motor Company, Defendants.**

No. 96–6650.

United States Court of Appeals,
Eleventh Circuit.

Dec. 23, 1997.

Plaintiffs brought products liability action against tire manufacturer and tire distributor for injuries sustained when right rear tire on vehicle failed. The United States District Court for the Southern District of Alabama, No. 93–0860–CB–S, 923 F. Supp. 1514, Charles R. Butler, J., granted summary judgment for defendants, and plaintiffs appealed. The Court of Appeals, Birch, Circuit Judge, held that testimony of purported expert on tire failure was not "scientific" and thus was not subject to *Daubert* inquiry for determining admissibility of scientific expert testimony.

Reversed and remanded.

1. Federal Courts 🔑776, 823

Court of Appeals reviews district court's legal decision to apply *Daubert*, which governs admission of scientific expert testimony, de novo, and court's decision to exclude particular evidence under *Daubert* for abuse of discretion. Fed.Rules Evid.Rule 702, 28 U.S.C.A.

2. Evidence 🔑555.2

*Daubert* does not create special analysis for answering questions about admissibility of all expert testimony; instead, *Daubert* provides method for evaluating reliability of witnesses who claim scientific expertise. Fed. Rules Evid.Rule 702, 28 U.S.C.A.

3. Evidence 🔑555.2

For purpose of *Daubert* standard, which governs admission of scientific expert testimony, "scientific" expert is expert who relies on application of scientific principles, rather than on skill- or experience-based observation, for basis of his opinion. Fed.Rules Evid.Rule 702, 28 U.S.C.A.

See publication Words and Phrases for other judicial constructions and definitions.

4. Evidence 🔑555.5

Testimony of purported expert on tire failure was not "scientific" and thus was not subject to *Daubert* inquiry for determining admissibility of scientific expert testimony, in products liability action asserting that defect in tire caused tire to fail and cause injuries to passengers in vehicle, because expert's opinion was not based on any scientific theory of physics or chemistry, but on expert's experience in analyzing failed tires. Fed.Rules Evid.Rule 702, 28 U.S.C.A.

5. Evidence 🔑508, 555.2

It is district court's duty to determine if nonscientific expert's testimony is sufficiently reliable and relevant to assist jury. Fed. Rules Evid.Rule 702, 28 U.S.C.A.

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Steven A. Martino, Robert J. Hedge, Scott E. Denson, Sid Jackson, Jackson, Taylor and Mantino, Mobile, AL, for Plaintiffs–Appellants.

Warren C. Herlong, Jr., Joseph Pr. H. Babington, John T. Dukes, Helmsing, Lyons, Sims & Leach, Mobile, AL, for Defendants–Appellees.

Appeal from the United States District Court for the Southern District of Alabama.

Before BIRCH and CARNES, Circuit Judges, and PROPST *, Senior District Judge.

BIRCH, Circuit Judge:

In this appeal, we determine whether the Supreme Court's *Daubert*[1] criteria for admission of scientific evidence should apply to testimony from a tire failure expert. In granting summary judgment against plaintiff-appellants, the district court relied on *Daubert* to exclude testimony from plaintiff-appellants' expert. Plaintiff-appellants, however, argue that the district court should not have applied *Daubert* because their expert's proffered testimony is not "scientific." We REVERSE.

## I. BACKGROUND

On July 6, 1993, plaintiff-appellants, eight members of the Carmichael family (collectively "the Carmichaels"), were involved in a serious automobile mishap when the right rear tire on their minivan failed. This occurrence resulted in significant trauma to each of the Carmichaels; one member of the family ultimately died from her injuries. For the purposes of this appeal, the parties agree that the failure of a tire manufactured and sold by defendant-appellees (collectively "Samyang") directly caused the mishap.

Following the incident, the Carmichaels submitted the carcass of the failed tire to George Edwards, a purported expert on tire failure. After examining the tire, Edwards determined that its failure was not the result of any abuse by the Carmichaels. Therefore, Edwards concluded that a defect in either the tire's design or its manufacture caused the blowout. Before Edwards could be deposed by Samyang, however, he became too ill to testify and transferred the case to his employee, Dennis Carlson.[2] After reviewing Edwards's file on the tire and discussing the case with Edwards, Carlson confirmed Edwards's conclusion that a design or manufacturing defect caused the blowout. Carlson, though, did not personally examine the tire until approximately one hour before his deposition by Samyang, long after he had rendered his opinion on the cause of the blowout. In his deposition, Carlson then set-forth both his analytical process and his conclusion that the Carmichaels' tire was defective.

Before the district court, Samyang moved for the exclusion of Carlson's testimony on the ground that it could not satisfy *Daubert*'s standards for reliability of scientific evidence. After reviewing Carlson's deposition, the district court agreed and excluded Carlson, writing that "none of the four admissibility criteria outlined by the *Daubert* court are satisfied in this case." *Carmichael,* 923 F.Supp. at 1521. Because the Carmichaels' only proffered evidence of a tire defect was Carlson's testimony, the district court then granted summary judgment for Samyang. *See id.* at 1524. The Carmichaels now appeal the exclusion of their tire expert.

## II. DISCUSSION

[1] In *Daubert,* the Supreme Court established several general criteria for the admission of scientific expert testimony under Federal Rule of Evidence 702.[3] *See Daubert,* 509 U.S. at 593–95, 113 S.Ct. at 2796–

* Honorable Robert B. Propst, Senior U.S. District Judge for the Northern District of Alabama, sitting by designation.

1. *Daubert v. Merrell Dow Pharm., Inc.,* 509 U.S. 579, 113 S.Ct. 2786, 125 L.Ed.2d 469 (1993).

2. Carlson holds a bachelor's and a master's degree in mechanical engineering from the Georgia Institute of Technology. Carlson worked from 1977 to 1987 as a research engineer for Michelin Americas Research & Development, where he was involved for the majority of his tenure in tire testing. Following that experience, Carlson became a senior project engineer at S.E.A., Inc., where he served from 1987 to 1994 as a tire failure consultant before becoming an employee of George R. Edwards, Inc. The District Court assumed for the purpose of its *Daubert* analysis that Carlson is qualified to testify as an expert in tire failure analysis. *See Carmichael v. Samyang Tires, Inc.,* 923 F.Supp. 1514, 1518–19 (S.D.Ala. 1996). We, like the district court, assume that Carlson is an expert for the purposes of this appeal.

3. Rule 702 provides that "If scientific, technical, or other specialized knowledge will assist the trier of fact to understand the evidence or to determine a fact in issue, a witness qualified as an expert by knowledge, skill, experience, training, or education, may testify thereto in the form of an opinion or otherwise."

98.[4] Appealing the district court's exclusion of Carlson's testimony, the Carmichaels argue that the district court should not have applied *Daubert*'s reliability framework because Carlson is not a "scientific" expert. In response, Samyang contends that Carlson's testimony is based on an unreliable scientific analysis. We review the district court's legal decision to apply *Daubert de novo, see Compton v. Subaru of Am., Inc.*, 82 F.3d 1513, 1517 (10th Cir.), *cert. denied*, — U.S. —, 117 S.Ct. 611, 136 L.Ed.2d 536 (1996), and its decision to exclude particular evidence under *Daubert* for abuse of discretion, *see General Elec. Co. v. Joiner*, — U.S. —, 118 S.Ct. 512, — L.Ed.2d — (1997).

[2] Despite Samyang's protestations, "Daubert does not create a special analysis for answering questions about the admissibility of all expert testimony. Instead, it provides a method for evaluating the reliability of witnesses who claim scientific expertise." *United States v. Sinclair*, 74 F.3d 753, 757 (7th Cir.1996). In fact, the Supreme Court in *Daubert* explicitly limited its holding to cover only the "scientific context." *Daubert*, 509 U.S. at 590 n. 8, 113 S.Ct. at 2795 n. 8; *see also United States v. Cordoba*, 104 F.3d 225, 230 (9th Cir.1997) ("*Daubert* applies only to the admission of scientific testimony."); *Compton*, 82 F.3d at 1518 (same); *Iacobelli Constr., Inc. v. County of Monroe*, 32 F.3d 19, 25 (2d Cir.1994) (same).[5] Although the Court's analysis in *Daubert* may suggest reliability issues for district courts to consider as they determine whether proffered evidence is sufficiently reliable for admission under Rule 702, "the trial court's role as gatekeeper is not intended to serve as a replacement for the adversary system: 'Vigorous cross-examination, presentation of contrary evidence, and careful instruction on the burden of proof are the traditional and appropriate means of attacking shaky but admissible evidence.'" *United States v. 14.38 Acres of Land*, 80 F.3d 1074, 1078 (5th Cir.1996) (quoting *Daubert*, 509 U.S. at 596, 113 S.Ct. at 2798).

[3] What, then, is the difference between scientific and non-scientific expert testimony? In short, a scientific expert is an expert who relies on the application of scientific principles, rather than on skill- or experience-based observation, for the basis of his opinion. *See Daubert*, 509 U.S. at 590, 113 S.Ct. at 2795. As the Sixth Circuit explained in *Berry v. City of Detroit:*

> The distinction between scientific and non-scientific expert testimony is a critical one. By way of illustration, if one wanted to explain to a jury how a bumblebee is able to fly, an aeronautical engineer might be a helpful witness. Since flight principles have some universality, the expert could apply general principles to the case of the bumblebee. Conceivably, even if he had never seen a bumblebee, he still would be qualified to testify, as long as he was familiar with its component parts.
>
> On the other hand, if one wanted to prove that bumblebees always take off into the wind, a beekeeper with no scientific training at all would be an acceptable witness *if* a proper foundation were laid for his conclusions. The foundation would not relate to his formal training, but to his firsthand observations. In other words, the beekeeper does not know any more about flight principles than the jurors, but

4. The Court suggested four primary inquiries for determining the reliability of a scientific theory or technique: (1) whether it has been tested; (2) whether it has been subject to peer review and publication; (3) its known or potential rate of error; and (4) whether it is generally accepted by the relevant scientific community. However, the Court emphasized that "[t]he inquiry envisioned by Rule 702 is ... a flexible one. Its overarching subject is the scientific validity—and thus the evidentiary relevance and reliability—of the principles that underlie a proposed submission." *Daubert*, 509 U.S. at 594–95, 113 S.Ct. at 2797.

5. Samyang's citations to *United States v. Lee*, 25 F.3d 997 (11th Cir.1994) (per curiam), for the contrary position are inapposite. In *Lee*, we examined whether a district court should apply *Daubert*'s reliability factors to evidence produced by machines. *Id.* at 998. Because the results produced by the machines were "only admissible through the testimony of an expert witness," and because "courts do not distinguish between the standards controlling admission of evidence from experts and evidence from machines," we remanded for reconsideration in light of *Daubert*. *Id.* at 998–99. Nowhere in *Lee* did we imply that *Daubert* applied to non-scientific expert testimony.

he has seen a lot more bumblebees than they have.

25 F.3d 1342, 1349–50 (6th Cir.1994); *see also Sorenson v. Robert B. Miller & Assoc., Inc.*, Nos. 95–5085, 95–5086, 1996 WL 515351, (applying *Berry*).[6] Thus, the question in this case is whether Carlson's testimony is based on his application of scientific principles or theories (which we should submit to a *Daubert* analysis) or on his utilization of personal experience and skill with failed tires (which we would usually expect a district court to allow a jury to evaluate). In other words, is the testimony at issue in this case more like that of a beekeeper applying his experience with bees or that of an aeronautical engineer applying his more generalized knowledge of the scientific principles of flight?

[4] Having clarified the question posed by this case, it seems apparent to us that Carlson's testimony is non-scientific. Although Samyang is no doubt correct that the laws of physics and chemistry are implicated in the failure of the Carmichaels' tire, Carlson makes no pretense of basing his opinion on any scientific theory of physics or chemistry.[7] Instead, Carlson rests his opinion on his experience in analyzing failed tires. After years of looking at the mangled carcasses of blown-out tires, Carlson claims that he can identify telltale markings revealing whether a tire failed because of abuse or defect.[8] Like a beekeeper who claims to have learned through years of observation that his charges always take flight into the wind, Carlson maintains that his experiences in analyzing tires have taught him what "bead grooves" and "sidewall deterioration" indicate as to the cause of a tire's failure. Indeed, Carlson asserts no knowledge of the physics or chemistry that might explain why the Carmichaels' tire failed. Thus, we conclude that Carlson's testimony falls outside the scope of *Daubert* and that the district court erred as a matter of law by applying *Daubert* in this case.

[5] Still, the inapplicability of *Daubert* should not end the day regarding Carlson's reliability. Under Rule 702, it is the district court's duty to determine if Carlson's testimony is sufficiently reliable and relevant to assist a jury. *See 14.38 Acres*, 80 F.3d at 1078. Moreover, Carlson's testimony is subject to exclusion under Federal Rule of Evidence 403 if its probative value is substantially outweighed by its likely prejudicial effect.[9] Aside from its *Daubert* related arguments, Samyang has presented this court with a number of potentially troubling criticisms of Carlson's alleged expertise and methodology, including his rendering of an opinion regarding the Carmichaels' tire before he had per-

6. An analogy closer to the facts of this case would be the example of an auto mechanic and a burned-out spark plug discussed at oral argument. Given a proper foundation, a mechanic with years of experience with spark plugs might be able to identify for a jury burns or other marks on a spark plug that he believes disclose whether the plug burned out because of normal wear or some defect; an experienced mechanic may recognize patterns of normal and abnormal wear on an auto part even though he has no knowledge of the general principles of physics or chemistry that might explain why or how a spark plug works. Such a mechanic's testimony would be non-scientific, while the testimony of another expert on the nature and effects of combustion (applied to spark plugs) would be scientific.

7. If Carlson or the Carmichaels' counsel were to assert or imply a "scientific" basis for Carlson's testimony at trial, after representing to the district court and to this court that Carlson's opinions are "non-scientific," then we are confident that the district court will be able to take appropriate remedial measures.

8. We note that both Carlson's and Samyang's experts rely on the same markings on the Carmichaels' tire for their analyses; the existence and relevance of these signs has not been questioned by either party before this court.

9. After analyzing Carlson's proffered testimony under *Daubert*, the district court concluded that "Carlson's testimony is simply too unreliable, too speculative, and too attenuated to the scientific knowledge on which it is based to be of material assistance to the trier of fact...." *See Carmichael*, 923 F.Supp. at 1522. Even without requiring Carlson's testimony to satisfy the *Daubert* criteria on remand, the district court still may find that, under all the circumstances, Carlson's testimony is so unreliable as to be unhelpful to a jury. We do not intend our comments regarding Carlson's testimony or qualifications to constrain the district court's discretion to admit or exclude his testimony under the proper Rule 702 or Rule 403 standards.

sonally inspected its carcass.[10] We leave judgments about such matters to the discretion of the district court on remand.

## III. CONCLUSION

The district court erred as a matter of law in applying the *Daubert* criteria to the Carmichaels' proffered expert testimony. Therefore, we REVERSE and REMAND the case to the district court for further proceedings consistent with this opinion.

10. We note that many of Samyang's criticisms of Carlson may also apply to the qualification of Samyang's own tire failure expert. However, we leave such issues for the district court to consider on remand.

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**UNITED STATES of America, Plaintiff–Appellee,**

**v.**

**Jeffrey YOUNG, Defendant–Appellant.**

**No. 96–6699.**

United States Court of Appeals, Eleventh Circuit.

Dec. 23, 1997.

Defendant pleaded guilty in the United States District Court for the Middle District of Alabama, No. CR–95–233–N, Ira De Ment, J., to possessing, with intent to distribute, methamphetamine and, conditionally, to using and carrying three firearms during and in relation to methamphetamine charge. He appealed. The Court of Appeals held that evidence was sufficient to support firearms convictions.

Affirmed in part, sentence vacated, and remanded for resentencing.

**1. Criminal Law ⚷1139**

Whether there was sufficient evidence to sustain conviction is reviewed de novo.

**2. Criminal Law ⚷1139**

Construction of plea agreements is subject to de novo review.

**3. Weapons ⚷17(4)**

Evidence that loaded guns and drugs were present in car within defendant's reach, defendant's admission that drugs were intended for distribution, and defendant's implicit use of car to deliver drugs sufficiently established that defendant's carrying of guns was during and in relation to his distribution of drugs to support conviction of carrying firearms during and in relation to drug trafficking offense, even without any evidence that any sale was in progress or imminent. 18 U.S.C.A. § 924(c)(1).

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William B. Matthews, Jr., Ozark, AL, Stephen Glassroth, Joseph P. Van Heest, Glassroth & Associates, Montgomery, AL, for Defendant–Appellant.

Redding Pitt, U.S. Atty., Artur G. Davis, Asst. U.S. Atty., Montgomery, AL, for Plaintiff–Appellee.

Appeal from the United States District Court for the Middle District of Alabama.

Before EDMONDSON and DUBINA, Circuit Judges, and LIMBAUGH *, Senior District Judge.

PER CURIAM:

Defendant–Appellant Jeffrey Young entered a plea of guilty to possessing, with intent to distribute, methamphetamine and a conditional plea of guilty to using and carrying three firearms in relation to the methamphetamine charge. He appeals the district court's finding that there was sufficient evidence to conclude that Young carried the firearms in relation to the drug charge, and

* Honorable Stephen N. Limbaugh, Senior U.S. District Judge for the Eastern District of Missouri, sitting by designation.

Texas Department of Insurance
333 Guadalupe Street P.O.Box 149104 Austin,Texas 78714-9104
512/463-6169

August 22, 1997

**COMMISSIONER'S BULLETIN NO. B-0032-97**

**TO: ALL PROPERTY AND CASUALTY INSURANCE COMPANIES**

**RE: Coverage A (Dwelling) Coverage Under the Texas Standard Homeowner's Policy HO-B for Losses Resulting from Accidental Discharge Which Causes Damage to Dwelling Foundation**

The United States Fifth Circuit Court of Appeals recently issued the *Sharp* opinion[1] in which the court held that the Texas Standard Homeowner's Policy Form HO-B does not cover structural and cosmetic damage to a dwelling that results from a foundation shift which itself was caused by a plumbing leak beneath the house. Since the issuance of this opinion, the Department has received numerous inquiries from consumers, attorneys, and insurers on the Department's position on this matter. The Department does not agree with the *Sharp* holding. The purpose of this bulletin is to state the Department's position that there is coverage under Coverage A (Dwelling) in the HO-B policy form for such damage and to explain the reasons for this position.

Since the Texas Standard Homeowners Policy Form HO-B (HO-B) was first promulgated in Texas, the policy has provided coverage for damage to the dwelling, including the foundation, resulting from the peril of accidental discharge, leakage, or overflow of water from within a plumbing, heating, or air conditioning system or household appliance. (This peril is referred to as simply "accidental discharge" in the remainder of this bulletin.) In 1978, because some companies were paying for these losses and some were not, the Board amended the HO-B policy to clarify that all losses to the dwelling, including the foundation, caused by accidental discharge were covered. The language and formatting for Coverage A (Dwelling), Coverage B (Personal Property), and Section I Exclusions in the current HO-B policy form was first adopted in 1990 by the former State Board of Insurance (Board) upon the recommendation of the Board-appointed Advisory Committee for a Readable Homeowners Policy. Since the 1990 adoption, the

[1] *Sharp v. State Farm Fire and Casualty Insurance Companies*, 115 F.3d 1258 (5th Cir. June 30, 1997).

Department has consistently interpreted the HO-B policy to provide coverage for losses to the dwelling, including the foundation, caused by accidental discharge. It is the Department's position that under all risk coverage, as provided under Coverage A (Dwelling) in the HO-B policy, there is coverage under the policy when one of the exclusions in the Section I Exclusions is qualified by the terms of the policy (i.e., all-risk coverage) to provide coverage where the otherwise excluded peril is itself caused by a covered peril. For example, if accidental discharge, which is a covered peril under Coverage A (Dwelling) all risk coverage, causes settling, cracking, bulging, shrinkage, or expansion of the foundation (settling, cracking, etc. is an excluded peril under Section I Exclusions) the damage to the foundation is a covered loss. The Department believes that any analysis of whether there is coverage for losses to the dwelling, including the foundation, under the HO-B policy form must include consideration of the nature and intent of "all risk" coverage, as provided under the Coverage A (Dwelling) provision of the policy.

The Department's position that there is coverage in the HO-B policy for damage to foundations resulting from accidental discharge is supported by the following facts:

—The Advisory Committee for a Readable Homeowners Policy, which was appointed in 1989 to review existing residential property policies and draft "easy to read" policies, was directed by the former Board to not in any manner restrict coverage currently available to an insured under the then existing residential property policies. The existing HO-B policy provided coverage for damage to foundations, including cracking and settling resulting from accidental discharge. The advisory committee in presenting the revised readable policies to the Board for adoption in February 1990 indicated that the committee had fulfilled its mandate that no major restrictions in coverage had been made. This advisory committee was composed of representatives of the insurance industry, agents, and consumers and was assisted by Department staff.

—The premiums paid by HO-B policyholders policies are based on benchmark rates that include these types of losses. Had these types of losses been removed from the policy coverage in 1990, there would have been a reduction in rates to reflect this change, and there has been no such reduction. To the contrary, the Department believes that a substantial portion of the increase in homeowners rates in certain areas of the state over the past two years is due to foundation damage caused by accidental discharge of water. For example, according to data collected by the Department, approximately 75% to 80% of the homeowners losses in Nueces County are due to water damage. This compares to the statewide average of 15% to 20%. The Department estimates that approximately 85% of these water losses in Nueces County are due to damage to foundations as a result of accidental discharge. Concomitantly, homeowners rates in Nueces County have increased by 36.4% over the past two years, compared to a statewide decrease of 4.1%.

--Insurers have interpreted the HO-B policy (both before 1990 and since 1990) to provide coverage for damage to foundations resulting from accidental discharge because they have paid such claims. Some of the larger insurers have recently indicated to the Department that they have always paid claims for damage resulting from accidental discharge of water, even when settling and cracking of the foundation was involved, and that they continue to pay these types of claims.

--In 1993, the Texas Legislature adopted Article 5.35-2 of the Texas Insurance Code to require the Commissioner to adopt an endorsement form, to be attached to homeowners, farm and ranch owners, and fire insurance policies promulgated under Article 5.35, that excluded coverage for damage to foundations or slabs of insured dwellings when such damage was caused by accidental discharge. The enactment of this statute and the adoption of the endorsement in 1994 were fully supported by insurers. The Legislature would not have enacted this statute if the HO-B policy was not being interpreted to provide coverage for damage to foundations or slabs caused by accidental discharge. Clearly, according to the Legislature, without the exclusionary endorsement, the coverage is provided in the policy. Article 5.35-2 was repealed in 1995 to enable the Commissioner to determine the best means of addressing the issue of foundation losses from accidental discharge.

--In 1995, a working group was formed at the initiative of the Legislature, and in cooperation with the Department, to address the issue of foundation losses from accidental discharge occurring in various areas of the state. The group was composed of representatives of insurers, agents, the Office of Public Insurance Counsel, and the Department. While the group did discuss the coverage in the HO-B policy for damage to foundations and slabs caused by accidental discharge, the existence of that coverage was not an issue to the group. The group concluded that coverage should be limited for tear-out and replacement of building and land for accessing plumbing leaks and recommended the adoption of endorsements to provide for this limitation in coverage. The Commissioner adopted the recommended endorsements which cannot be used until premium credits for the endorsements are promulgated. The endorsement to the HO-B policy, as adopted by the Commissioner, specifically states that the tear-out limitation endorsement does not affect coverage otherwise provided in the policy for damage or loss to slabs or foundations.

Because decisions of federal circuit courts of appeals and federal district courts with respect to issues of state law are not binding on Texas state courts, the Department expects insurers to pay claims in accordance with the Department's position as stated in this bulletin, and the Department will monitor insurers for compliance. The Department recognizes that there may be legitimate disputes about the cause of a loss, and this bulletin is not intended to address these disputes or render such disputes invalid.

An insurer's refusal to pay claims under the Texas Standard Homeowners Policy Form HO-B for damage to the insured dwelling, including damage to the foundation caused by settling, cracking, bulging, shrinkage, or expansion, caused by the peril of accidental discharge, leakage, or overflow of water from within a plumbing, heating, or air conditioning system or household appliance may subject the insurer to disciplinary action for violations of the Texas Insurance Code, including unfair claim settlement practices pursuant to Article 21.21 §4(10)(a) and Article 21.21-2. Under both statutory and common law, insurers have a duty to deal fairly and in good faith with their insureds. The Texas Supreme Court recently unified the common law and statutory standard for bad faith. Adopting the statutory bad-faith standard in Article 21.21 §4(10)(a)(ii) of the Texas Insurance Code, the court held that an insurer breaches its duty of good faith and fair dealing when the insurer fails to settle a claim if the insurer knew or should have known that it was reasonably clear that the claim was covered.[2]

ELTON BOMER
COMMISSIONER OF INSURANCE

[2] *Universe Life Insurance Co. v. Giles*, 40 Tex. Sup. Ct. J. 810, 815 (July 9, 1997).

# IN THE SUPREME COURT OF TEXAS

No. 97-1093

Joe Balandran and Dolores Balandran, Appellants

v.

Safeco Insurance Company of America, Appellee

On Certified Question from the United States

Court of Appeals for the Fifth Circuit

Argued February 4, 1998

Chief Justice Phillips delivered the opinion of the Court, in which Justice Gonzalez, Justice Enoch, Justice Spector, Justice Baker, Justice Abbott and Justice Hankinson join.

Justice Owen filed a dissenting opinion, in which Justice Hecht joins.

This case comes to us on a certified question from the United States Court of Appeals for the Fifth Circuit. The issue certified is whether the 1991 Texas Standard Homeowner's Policy--Form B covers damage to the insured's dwelling from foundation movement caused by an underground plumbing leak. We hold that the policy provides this coverage.

## I

Safeco Insurance Company of America insured the home of Joe and Dolores Balandran. The form of the policy was the 1991 Texas Standard Homeowner's Policy--Form B. In September 1993, the Balandrans filed a claim against Safeco for damage to their home caused by an underground plumbing leak. The leak caused the soil to expand, damaging the home's foundation as well as its interior and exterior finishes. When Safeco denied the claim, the Balandrans sued the company in state district court. Safeco removed the case to federal court on diversity jurisdiction.

At trial, the jury found that the structural damage was caused by the plumbing leak and awarded the Balandrans $66,500. Safeco, however, moved for judgment as a matter of law, contending that the Balandrans' policy excluded this structural damage regardless of the underlying cause. The trial court granted this motion, rendering a take-nothing judgment for Safeco.

The Balandrans appealed to the Fifth Circuit Court of Appeals. While their appeal was pending, a separate Fifth Circuit panel considered this issue, holding that an identical policy did not provide coverage for foundation damage from a plumbing leak. *See Sharp v. State Farm Fire & Cas. Ins. Co.*, 115 F.3d 1258 (5th Cir. 1997). Subsequently, however, the Texas Commissioner of Insurance issued a bulletin vigorously disagreeing with the *Sharp* decision. *See* Tex. Dep't of Ins. Bulletin B-0032-97 (Aug. 22, 1997). In light of these developments, the panel hearing the Balandrans' appeal certified to us the controlling question regarding policy coverage.

## II

The Balandrans' policy provides two types of coverage. "Coverage A" insures the dwelling itself, while "Coverage B" insures personal property. Coverage A provides the following protection:

We insure against all risks of physical loss to the [dwelling] unless the loss is excluded in Section I Exclusions.

The exclusion relevant to this case is 1(h), which provides:

We do not cover loss under Coverage A (Dwelling) caused by settling, cracking, bulging, shrinkage, or expansion of foundations, walls, floors, ceilings, roof structures, walks, drives, curbs, fences, retaining walls or swimming pools.

We do cover ensuing loss caused by collapse of building or any part of the building, water damage or breakage of glass which is part of the building if the loss would otherwise be covered under this policy.

Safeco argues that the damage to the Balandrans' home clearly falls under this exclusion. The Balandrans apparently concede that, if the exclusion applies, it excludes their claim. However, they present three arguments about why the exclusion does not apply. First, they contend that language in Coverage B (the personal property section of the policy) creates an exception to exclusion 1(h) when the structural damage results from a plumbing leak. Second, they argue that exclusion 1(h) does not apply to structural damage resulting from an underlying cause -- in this case a plumbing leak -- which itself is not an excluded peril under the policy. Finally, the Balandrans argue that the last sentence of exclusion 1(h) (the "ensuing loss" provision) creates an exception to exclusion 1(h) under the present circumstances. Because we conclude that the Balandrans are entitled to prevail on their first argument, we do not reach the other two.

## III

### A

Unlike Coverage A, which insures the dwelling against "all risks," Coverage B insures personal property only against twelve enumerated perils. The ninth of these twelve perils is:

**Accidental Discharge, Leakage or Overflow of Water or Steam** from within a plumbing, heating or air conditioning system or household appliance.

A loss resulting from this peril includes the cost of tearing out and replacing any part of the building necessary to repair or replace the system or appliance. But this does not include loss to the system or appliance from which the water or steam escaped.

*Exclusions 1.a through 1.h under Section I Exclusions do not apply to loss caused by this peril.*

(bold in original, italics added). Even though Coverage B deals with personal property loss, which the Balandrans did not suffer, the Balandrans rely heavily on the last sentence quoted above. They argue that this provision (the "exclusion repeal provision") means exactly what it says: Exclusions 1(a) through 1(h) do not apply to a loss caused by a plumbing leak. Because exclusion 1(h) does not apply to the Balandrans' loss, it is covered under Coverage A, which insures against any risk to the dwelling. In other words, the exclusion repeal provision, on its face, applies to any "loss," not just personal property losses.

Safeco, relying on the structure of the policy, argues that the exclusion repeal provision applies only to personal property losses resulting from a plumbing leak. Because Coverage B deals with personal property coverage, Safeco contends that the exclusion repeal provision should be similarly limited. Safeco argues that we may not construe this sentence without considering its context within the policy. *See State Farm Life Ins. Co. v. Beaston*, 907 S.W.2d 430, 433 (Tex. 1995) ("[C]ourts must be particularly wary of isolating from its surroundings or considering apart from other provisions a single phrase, sentence, or section of a contract.").

As we have already noted, one Fifth Circuit panel has adopted Safeco's approach. *See Sharp v. State Farm Fire & Cas. Ins. Co.*, 115 F.3d 1258 (5th Cir. 1997). Under identical facts, the court held that the damage to the dwelling was excluded under exclusion 1(h), and that the exclusion repeal provision applied only to personal property losses:

We are sympathetic to the Sharps' situation, but we cannot agree that text specifically included in Coverage B, which applies only to personal property, may be imported into Coverage A, which applies to the dwelling or house, in order to create coverage for a loss that does not involve personal property damage. The Sharps' policy clearly and unambiguously divides dwelling losses and personal property losses into two separate "coverages." It therefore would appear to be nonsensical, and a rejection of the obvious structure of the policy, to reach into text that applies solely to Coverage B (Personal Property) to determine the extent of coverage provided under Coverage A (Dwelling).

115 F.3d at 1262.

## B

Several rules of construction guide our consideration of this issue. First, insurance contracts are subject to the same rules of construction as other contracts. *See Beaston*, 907 S.W.2d at 433; *National Union Fire Ins. Co. v. CBI Indus.*, 907 S.W.2d 517, 520 (Tex. 1995); *Forbau v. Aetna Life Ins. Co.*, 876 S.W.2d 132, 133 (Tex. 1994). Our primary goal, therefore, is to give effect to the written expression of the parties' intent. *See Beaston*, 907 S.W.2d at 433; *Forbau*, 876 S.W.2d at 133. We must read all parts of the contract together, *see Beaston*, 907 S.W.2d at 433, striving to give meaning to every sentence, clause, and word to avoid rendering any portion inoperative. *See United Serv. Auto. Ass'n v. Miles*, 161 S.W.2d 1048, 1050 (Tex. 1942). While parol evidence of the parties' intent is not admissible to create an ambiguity, *see National Union*, 907 S.W.2d at 520, the contract may be read in light of the surrounding circumstances to determine whether an ambiguity exists. *See Columbia Gas Transmission Corp. v. New Ulm Gas, Ltd.*, 940 S.W.2d 587, 589 (Tex. 1996); *National Union*, 907 S.W.2d at 520.

If, after applying these rules, a contract is subject to two or more reasonable interpretations, it is ambiguous. *See National Union*, 907 S.W.2d at 520. Where an ambiguity involves an exclusionary provision of an insurance policy, we "must adopt the construction . . . urged by the insured as long as that construction is not unreasonable, even if the construction urged by the insurer appears to be more reasonable or a more accurate reflection of the parties' intent." *National Union Fire Ins. Co. v. Hudson Energy Co.*, 811 S.W.2d 552, 555 (Tex. 1991); *see also Glover v. National Ins. Underwriters*, 545 S.W.2d 755, 761 (Tex. 1977).[(1)]

Applying these rules, we conclude that the exclusion repeal provision is subject to two reasonable interpretations, and is therefore ambiguous. We are mindful of the Fifth Circuit's reasoning in *Sharp*, and we agree that it reflects one reasonable interpretation of the policy language. However, the Balandrans' interpretation is also reasonable. First, the policy on its face states that exclusion 1(h) does not apply to "loss" caused by a plumbing leak; this repeal of exclusion 1(h) is not expressly limited to "personal property loss." That the exclusion repeal provision is contained in Coverage B does not necessarily dictate Safeco's narrow reading. Instead, the exclusion repeal provision could be located under Coverage B simply because that is the only place in the policy that the "accidental discharge" risk is specifically described. Because the exclusion repeal provision applies solely to that risk, it is logical for it to be adjacent to the policy's description of the risk.

Further, Safeco's construction of the policy renders a part of the policy language meaningless. The exclusion repeal provision applies to "[e]xclusions 1.a. through 1.h." Under Safeco's reading, of course, exclusions 1(a) through 1(h) are repealed only for personal property losses caused by a plumbing leak. However, exclusion 1(h) on its face applies only to damage to the dwelling. Thus, if Safeco's reading is correct, it would have been unnecessary to extend the exclusion repeal provision to exclusion 1(h), because that exclusion can never affect personal property losses. Under Safeco's approach, therefore, the part of the exclusion repeal provision referring to exclusion 1(h) is without any effect.

The Balandrans' interpretation becomes even more reasonable when we consider the circumstances surrounding the promulgation of this policy form. Article 5.35 of the Texas Insurance Code, subject to certain exceptions not relevant here, requires insurers to use policy forms adopted or approved by the Commissioner of Insurance. The policy at issue here was promulgated in 1990 by an advisory committee appointed by the Board of Insurance, the Commissioner's statutory predecessor. The Board directed this committee, which consisted of insurance industry representatives and consumer representatives, "to assist the Board with conversion of the Texas Standard Homeowners Policies into a simplified, easy-to-read form for use in the State of Texas." *See* Record of Official Action of the State Bd. of Ins. no. 54929 (July 18, 1989). The Board expressly instructed the committee "that such conversion process shall not in any manner restrict coverages currently available to the insured under a homeowners policy." *Id.*

The policy in effect when the committee started its work unambiguously covered foundation damage resulting from a plumbing leak. Effective since 1978, that policy contained exclusion repeal language similar to that at issue here, but it was located in the *exclusions* section. Thus, one could not argue that the exclusion repeal provision applied only to personal property loss. *See State Farm Lloyds v. Nicolau*, 951 S.W.2d 444, 446 (Tex. 1997) ("Under an express exception, however, these exclusions [referring to, among others, the foundation-damage exclusion] do not apply to losses caused by an `accidental discharge, leakage or overflow of water' from within a plumbing system."). (2) The 1978 policy, like the present one, also recited the "accidental discharge" language in the Coverage B (personal property) section. The committee, in promulgating its "easy-to-read" policy, moved the exclusion repeal language to section B, adjacent to the "accidental discharge" language there, thus eliminating the need to restate this language. The Board subsequently adopted the committee's form after being assured by the committee's chairman, Don Olsen (a representative of State Farm Fire & Casualty Insurance Company), that the revisions were "accomplished in line with [the Board's] charge of making sure that there is no restriction in coverage available to any insured under an existing homeowner policy in Texas." *See* February 14, 1990, Hearing on Property Ins. Rules Concerning Texas Homeowners Policy and Related Matters at 5. The circumstances surrounding the drafting of this policy thus support the Balandrans' theory that the exclusion repeal provision is located within Coverage B merely to simplify the policy, not to restrict the scope of the exclusion repeal.(3)

Safeco cites several cases for the proposition that exclusion 1(h) excludes damage to foundations, regardless of the underlying cause. *See, e.g., General Ins. Co. of America v. Hallmark*, 575 S.W.2d 134, 136 (Tex. Civ. App.--Eastland 1978, writ ref'd n.r.e.); *Lambros v. Standard Fire Ins. Co.*, 530 S.W.2d 138, 140 (Tex. Civ. App.--San Antonio 1975, writ ref'd); *Bentley v. National Standard Ins. Co.*, 507 S.W.2d 652, 654 (Tex. Civ. App.--Waco 1974, writ ref'd n.r.e.). These cases construe the standard homeowner's policy in effect before 1978, which contained no exclusion repeal provision for accidental discharge of water from a plumbing unit. These cases do not apply to the issue before us.

In sum, we conclude that the Balandrans' interpretation of the exclusion repeal provision is not unreasonable. Because the Balandrans are the insureds, we adopt their interpretation as the proper construction of the policy.

* * *

Accordingly, we hold that exclusion 1(h) in the 1991 Texas Standard Homeowner's Policy--Form B does not apply to loss caused by the accidental discharge, leakage or overflow of water or steam from within a plumbing, heating or air conditioning system or household appliance.

---

Thomas R. Phillips

Chief Justice

Opinion Delivered: July 3, 1998

1. This widely followed rule is an outgrowth of the general principle that uncertain contractual language is construed against the party selecting that language. *See* Segalla, 2 Couch on Insurance § 22.14 (3d ed. 1997). It is also justified by the special relationship between insurers and insureds arising from the parties' unequal bargaining power. *See Arnold v. National County Mut. Fire Ins. Co.*, 725 S.W.2d 165, 167 (Tex. 1987).

2. The relevant language from the 1978 policy was as follows:

EXCLUSIONS (Applicable to Property Insured under Coverages A and B and Perils Insured Against)--This insurance does not cover:

. . .

k. Loss under Coverage A caused by settling, cracking, bulging, shrinkage, or expansion of foundations, walls, floors, ceilings, roof structures, walks, drives, curbs, fences, retaining walls or swimming pools.

The foregoing Exclusions a, b, c, f, h, i, j and k shall not apply to Accidental discharge, leakage or overflow of water or steam from within a plumbing, heating or air conditioning system or a domestic appliance (including necessary tearing out and replacing any part of the building covered).

3. Contrary to the dissenting justices' contention, we are not considering this evidence for the purpose of creating an ambiguity. Because the Balandrans' interpretation of the contract language is reasonable, an ambiguity exists on the face of the policy. We merely highlight this evidence because it further supports the result we reach.

## LEGAL ISSUES

## Litigation From the Insurance Carriers Point of View

Mr. Bob singleton

# LEGAL ISSUES

## Expert Witness Examination & Testimony

**Richard W. Peverley, PE**

# EXPERT WITNESS EXAMINATION AND TESTIMONY

**Richard W. Peverley, PE**
**Peverley Engineering, Inc.**
**Houston, Texas**

## INTRODUCTION

Being the recipient of a demand letter certainly is not a good way to start anyone's day. Once this has occurred, however, this unfortunate event can be taken in one of the two following ways. The natural reaction is to look on it with anger, resentment, and denial. This approach will certainly evoke sympathy from those near to us; but, it can also be the first step on the road to an expensive defeat. The other approach is to perceive such an event to be an opportunity to correct a wrong which has been done either to your client or is about to be done to you. A positive assumption is that this is a lesson to be learned not only for the purpose of avoiding such an event in the future but also on how to produce a better product as it is perceived by perhaps the most important people in your business - your clients. Having been in the business of inspecting residential foundations for the past twenty-three years, I can certainly remember the first demand letter that I received. My lesson was a personal commitment made that my firm would do whatever was necessary to avoid committing acts which would place us in the position of becoming the defendant in a law suit and to never to settle such a suite by admitting a wrong doing when it had not occurred. We have stuck with this commitment ever since. I also entered into the business of doing expert witness type work so that I could better understand the system.

In conducting expert witness work, I have had the benefit of seeing the results of the work of expert witnesses and the attorneys on both sides of many cases. This has given me an opportunity to be able to understand how mistakes have been made in the design and construction of residential foundations, how such mistakes can be avoided, and how the litigation process can be controlled on the part of every builder, if not avoided altogether. The purpose of this paper, therefore, is to pass on some of this experience to the benefit of those who have an opportunity to read this paper.

## CONDUCTING A FORENSIC EXAMINATION

The role of any expert witness in a litigation proceeding is to assist the attorney in the conduct of the case. An expert witness is obviously very important in a law suit, particularly in cases where the information is very technical in nature; however, it is the responsibility of the attorney to control the way the case is conducted and to use information provided by the expert witness in such a manner to best represent his client (be it the plaintiff or the defendant) in the best manner possible.

*Expert witnesses do not win or lose cases. They work for attorneys who win or lose, cases.*[Ref.1]

It is essential that an expert witness, in any case, provide unbiased information to his client/attorney. The expert witness, therefore, must never act as an advocate for the client of the attorney, be they plaintiff or defendant, regardless of how much sympathy may be deserving. It is certainly tempting, as a defendant, to be angry at an expert who presents a litany of errors and omissions which may have occurred in the design/construction process; but, remember that it is not acceptable to shoot the messenger.

The following is a typical scenario of how the expert examination of a home which may, or may not, have a foundation problem is conducted:

1. <u>The Hiring of an Expert</u>

An attorney may call and hire an expert bases on his own knowledge or based on recommendations made by friends and/or associates.

> *Perhaps the ultimate complement that an expert can receive is to be hired by an attorney who had lost a prior case where the expert was on the other side.*[Ref. 2]

An attorney may also interview several experts then hire the one who the attorney believes can prove him (or her) with the most competent assistance. The attorney has an option of hiring an engineer as a consultant early in the investigation then ultimately declaring him as an expert later on or hiring another individual as the expert.[Ref. 3] The difference is that a consultant can not be compelled to testify. Scientific evidence to be used by the expert was originally established to meet certain requirements in Fry v. United States[Ref. 4] and later in Daubert v. Merrell Dow Pharmaceuticals, Inc.[Ref. 5] We understand that there may be an issue over whether or not these standards apply to engineers which is being decided in the U. S. Supreme Court.[Ref. 6] The ability of a expert to provide evidence which is required to meet such standards is, of course, important to the attorney. An expert is advised to have a written contract with an attorney.[Ref. 7]

2. <u>Obtain an understanding of the issues in the case</u>

The expert must review all pleadings and discuss the case with his attorney the client. It may or may not be prudent to have a meeting with the client at this time. Such a decision may be made only by the attorney.

3. <u>The Examination of the Building and the Building Site</u>

An examination of the property will be conducted in the following manner:

a. The design drawings and the soil test requirements will be examined, if available.

b. A cursory, walk through examination of the home, grounds, and the landscaping will be conducted to obtain familiarity with the site.

c. The yard plan and building plan sketch will be made. This can be made either from the design drawings or measurements made at the site. An example is shown in Figure 1.

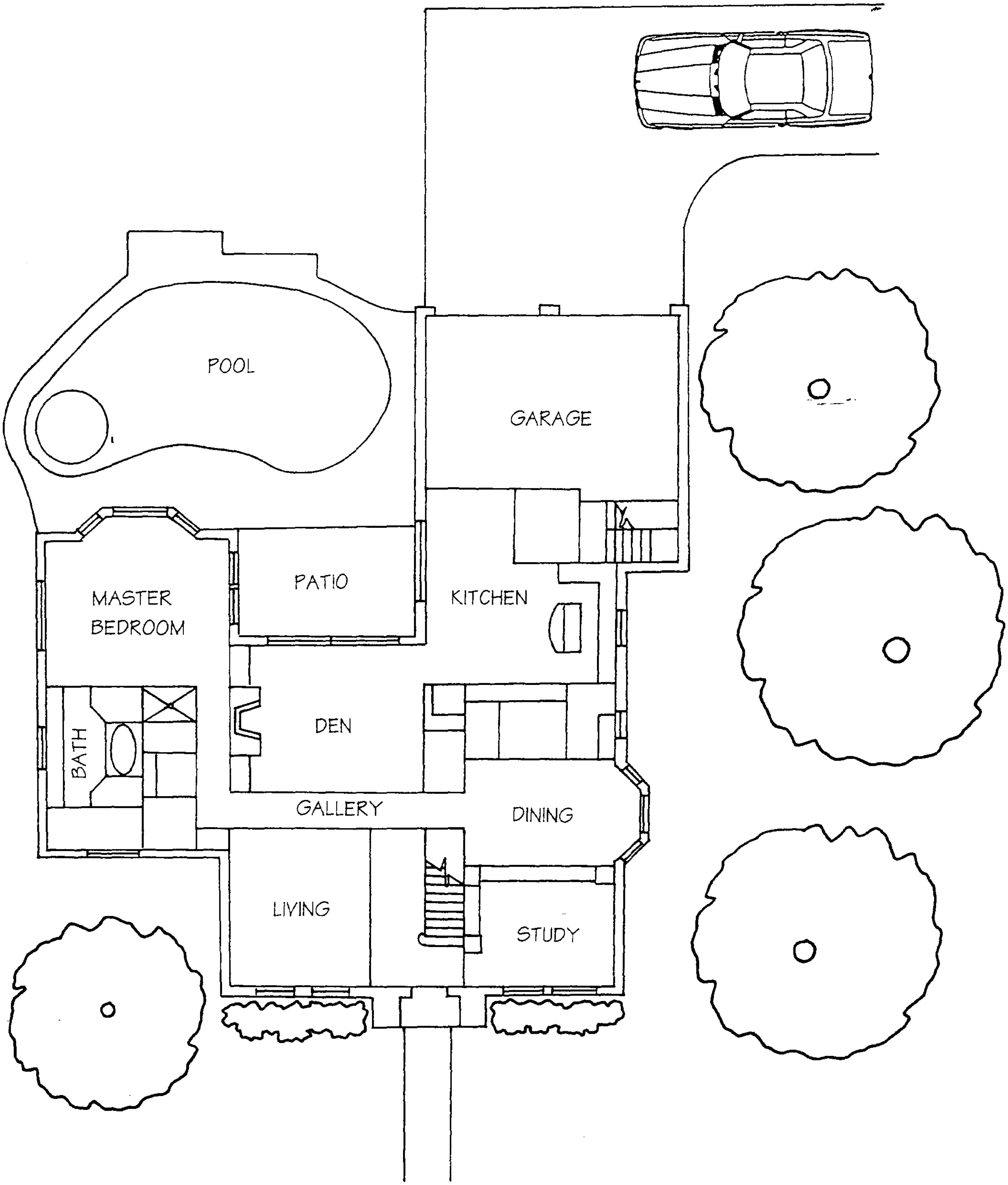



**FIGURE 1. A TYPICAL YARD & BUILDING PLAN SKETCH**

d. Relative floor height measurements shall be made. Such measurements can be made using a water level, a laser level, or a compu level of the type recently manufactured by the Stanley Corporation. An example is shown in Figure 2.

e. The data then can be analyzed by drawing contour intervals. An example is shown in Figure 3. Isometric plots can also be beneficial in the presentation of such data.

f. Evidence of foundation movement such as cracks, separations, distortions, etc. will be noted and listed on the drawing. A typical example, which we call a phenomenon plan, is shown in Figure 4.

3. Test Data Analysis

All of the foregoing data must be analyzed by the expert to see what kind of a message it is sending with regard to what happened to cause the owners to be sufficiently upset to have instituted the litigation proceedings.

4. Report Preparation

At this stage of the forensic examination, a formal report is generally prepared. One cannot over-emphasize the importance of the report preparation. Since the ultimate use of such a report is that it will be reviewed by a jury, the report must be presented in clear, concise terms that can be understood by the average individual. The report must list all assumptions made during the expert's study and identify their inherent limitations. Proper references must be included where knowledge and/or data are used. If it is, in the judgment of the attorney/client, prudent not to prepare a report at this stage or possibly not at all, the expert has no choice but to comply with such a request.

*It is noted that the forgoing procedures will probably satisfy current documented recommendations for such inverstigations.* [Refs. 8 & 9]

5. Review of all Subpoenaed Documents

The attorney/client is responsible for obtaining all of the documentation available from the other side. The expert can be useful in, helping his attorney/client understand what documents should be available and what documents are important. The attorney for the defendant will need to obtain such things as expert witness reports which will need to be reviewed by his expert. The subpoenaed documents may, or may not, be available prior to writing the final report.

6. Test Witnessing

Forensic examination of a foundation of the soil properties is often an essential part of an expert witnesses work. Such an examination is often conducted by a testing organization who specializes in this type of work. Such tests may include, but may not necessarily be limited to, soil borings, concrete core testing, rebar placement examinations, configuration identifications, etc. The manner in which such examinations

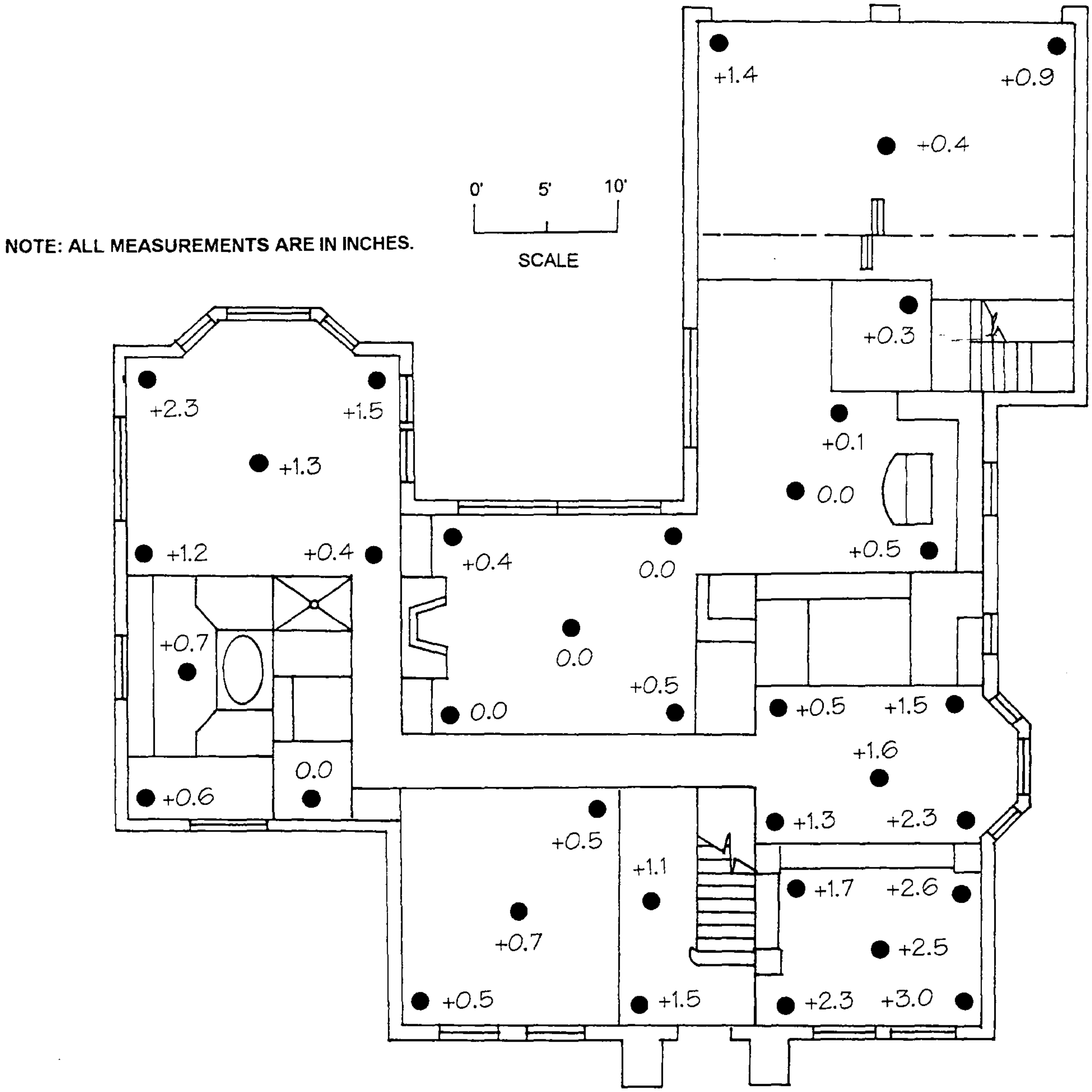



**FIGURE 2. AN EXAMPLE OF RELATIVE HEIGHT MEASUREMENTS**

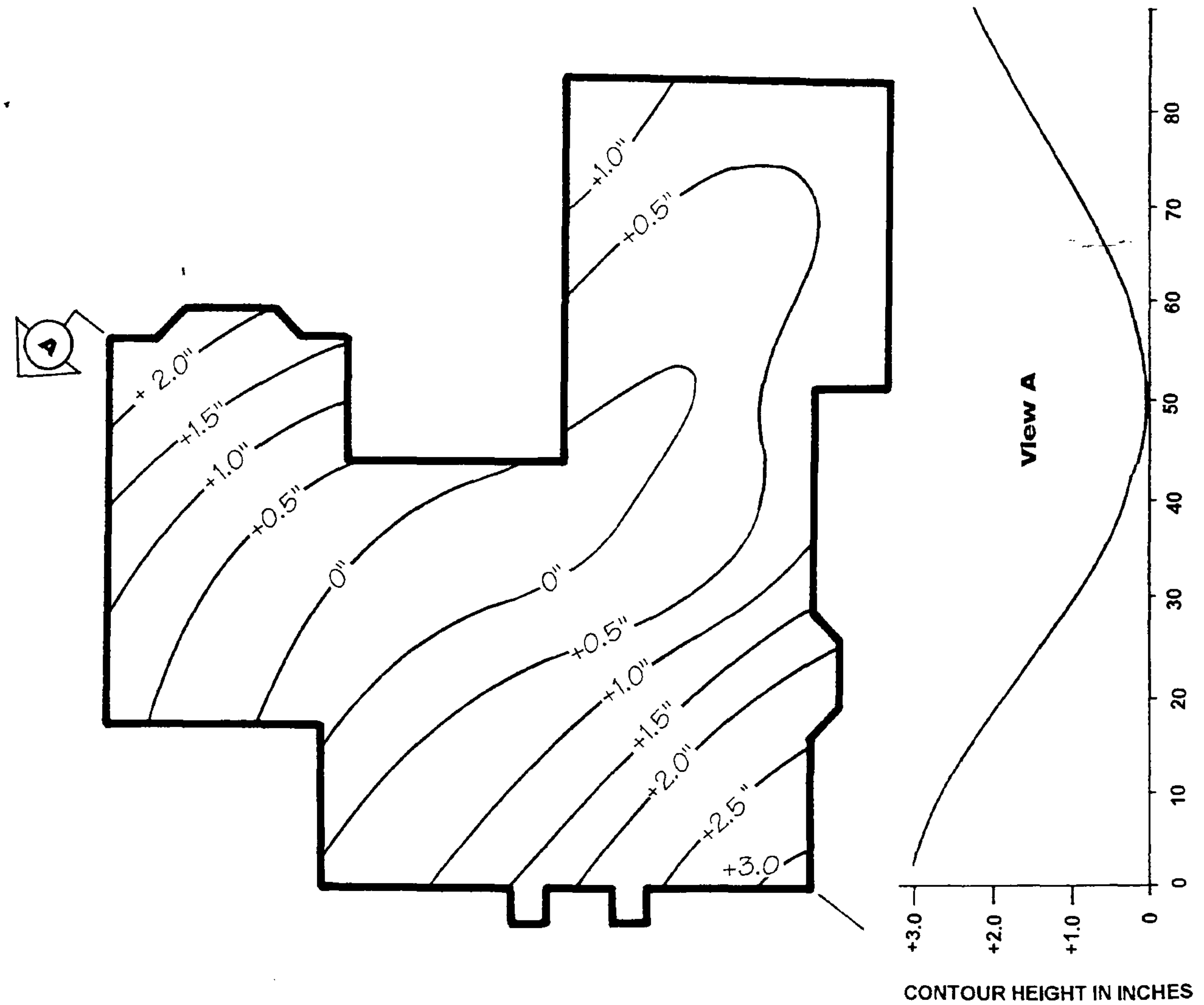



**FIGURE 3. AN EXAMPLE OF CONTOUR INTERVALS WHICH WERE DERIVED FROM FIGURE 3**

| NO. | DAMAGE PHENOMENON DESCRIPTION |
|---|---|
| E1 | BRICK VENEER/WINDOW SEPARATION |
| E2 | VERTICAL CRACK @ BRICK CORNER |
| E3 | BRICK VENEER/WINDOW SEPARATION |
| E4 | DIAGONAL CRACK UNDER WINDOW |
| E5 | DIAGONAL CRACK UNDER WINDOW |
| E6 | BRICK VENEER/WINDOW SEPARATION |
| E7 | VERTICAL CRACK OVER WINDOW |
| E8 | HORIZONTAL CRACK @ WINDOW TOP |
| E9 | VERTICAL CRACK OVER WINDOW |
| E10 | BRICK VENEER/WINDOW SEPARATION |
| E11 | DIAGONAL CRACK UNDER WINDOW |
| I1 | DIAGONAL CRACK @ UPPER CORNER |
| I2 | HORIZONTAL CRACK @ LOWER CORNER |
| I3 | SHEETROCK CORNER SEPARATION |
| I4 | SHEETROCK CORNER CRACK |
| I5 | DOOR FRAME HAS FRAME DISTORTION |
| I6 | DIAGONAL CRACK @ UPPER CORNER |
| I7 | HORIZONTAL CRACK @ LOWER CORNER |
| I8 | CABINET DOORS WILL NOT CLOSE |
| I9 | DOOR FRAME HAS FRAME DISTORTION |
| I10 | DIAGONAL CRACK @ DOOR CORNER |
| I11 | SHEETROCK CORNER SEPARATION |

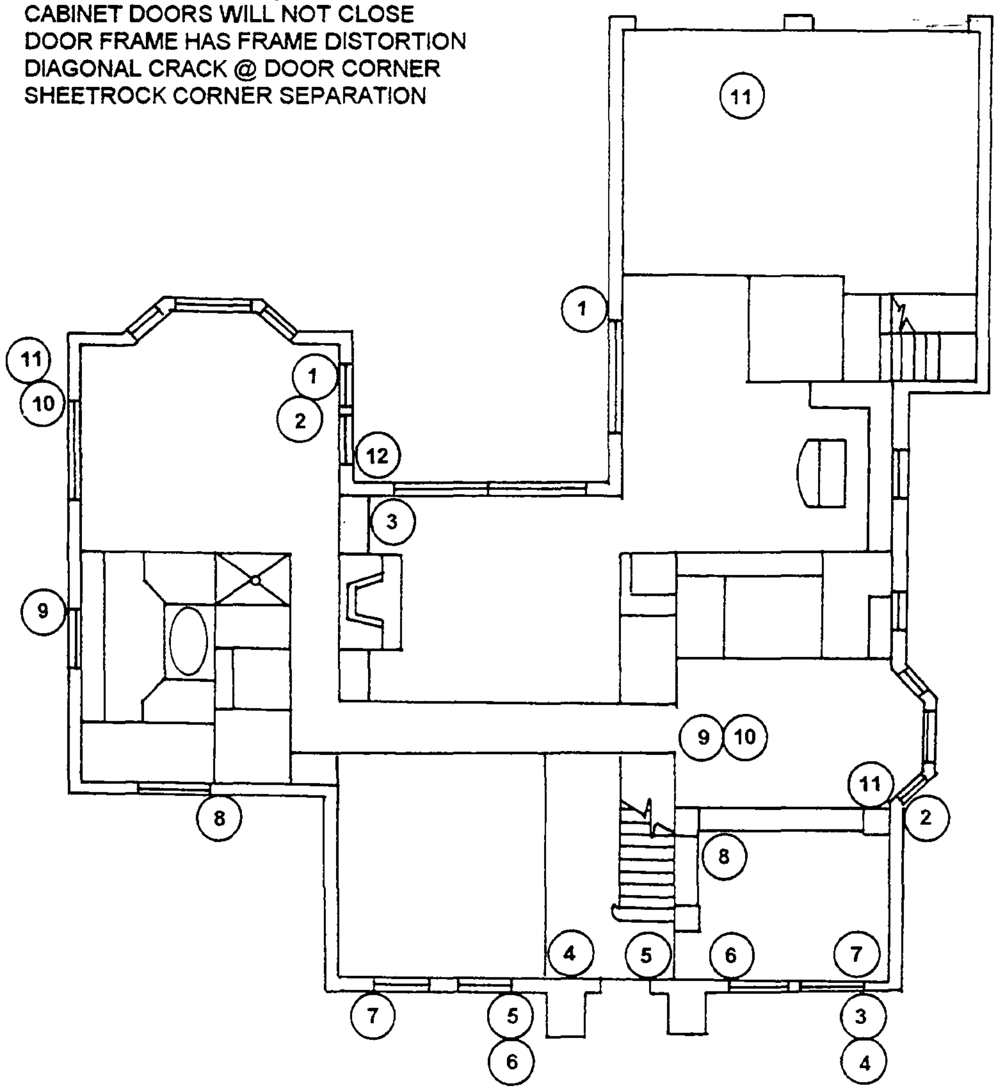



**FIGURE 4. A TYPICAL DAMAGE PHENOMENON PLAN**

are to be conducted will be discussed in the sister paper prepared by Mr. David Eastwood of GeoTech Engineering & Testing.

7. Providing Advice to the Attorney/Client

   At this state in the proceedings, there is often long periods of time where there appears to be little expert witness activity, At this stage, the role of expert witness then is to provide advice to the attorney/client as so requested. On occasion, one expert witness may be requested to accompany the deposition of another or to witness testing conducted by the expert for the other side. Experts are occasionally requested to provide advice during mediation proceedings.

8. Expert Witness Testimony

   Expert witness testimony may be in the form of depositions, arbitration and cross-examination, or appearance at trial. It is extremely important for an expert witness to have conversations with the attorney/client before testimony is ever given. Such meetings are essential for the expert witness to fully understand all of the issues of the case, to fully understand what approach is being taken by the attorney for the other side, to fully appreciate his attorney/client's courtroom tactics.

## LITIGATION CONTROL AND/ OR AVOIDANCE

The ability of those of us in the construction, or construction related, businesses to defend ourselves in a litigation proceeding must not begin in the courtroom but must always begin in action taken during the construction and/or examination process. Any builder, for example, who has an effective and well-documented quality control program will find himself in an excellent position not only to defend himself in litigation proceedings, but possibly to avoid them at all.

*Quality can be defined as adherence to requirements.* [Ref 10]

At first glance, this definition seem to lack something. A very expensive automobile is often said to be of a higher quality than a cheaper one, silk garments made in Italy are of a higher quality than cotton ones made in Formosa, etc. Why is one item judged to be of a higher quality than another? Simply stated, it meets the requirements established by the buying public. Prior to World War 2, the term "Made in Japan" meant a quality standard which rated a little higher than junk. Yet, today the quality standard in the automobile business and the electronics business has been set by the Japanese. Why? They run better, they run longer, and they need to be repaired less. Thus, the requirements of the consumer are satisfied. In the residential foundation business, the consumer requirements are simple - a foundation must support the residence to such a degree that differential movements do not cause cosmetic or structural damage to the building.

In terms of a construction program, quality and quality control must be established to meet the objectives described above. It means assuring that all soil testing is conducted in accordance with well documented standards of this type of work, to assure that the design meets adherence to all accepted industry standards in terms of what needs to be done to limit the type of deflections which cause damage to occur, adherence to the minimum standards provided by building codes which are in effect at that time and perhaps most important, adherence to the requirements contained in the designed documents. In most cases the builder has control over the soil testing and the foundation design. Soil testing should not be done at the lowest of all possible costs. Standards have recently been developed by the Texas Board of Professional Engineers [Ref. 11] for minimum soil test requirements. In the flat-lands of the greater Houston area the adherence to these minimal requirements is generally acceptable. There are, however, some parts of the greater Houston area where additional soil testing is required. The soil testing agencies know where such areas exists and the builder is well advised to listen and provide any advice given with regard to the need for additional tests. The engineering firms who design the foundation need to understand the builders objective in the design of the foundation. It is, in the authors opinion, entirely possible to design a foundation which will not fail. There is, of course, a limitation on how much money is to be spent in the design and construction of a residential foundation; however, experience in the litigation over residential foundations has shown that often the cheapest foundation can ultimately be the most expensive.

Having obtained all the necessary designed documents (i.e., the soil reports and the foundation design documents), the construction quality control assurance program now comes into play. The following is my opinion of what an ultimate quality assurance program should be. The reader may think a program of this depth to be somewhat of a strain. The reader may then wish to choose those elements in the program which seem to be beneficial. The following advice is, therefore, provided:

A SUGGESTED QUALITY CONTROL PROGRAM

1. <u>Written Commitment</u>

   In order for any quality control program to be effective, a commitment must be made to adhere to such a program in writing. Such a commitment can be in the form of a statement made by the chief executive officer of the company, regardless of its size, which should be posted not only in the office but at the construction site as well. It must be absolutely understood by everyone in the company that the commitment to quality is not simply words, but is an obligation that every member of the company adhere to all the requirements that exist and to avoid and never engage in any shortcuts which can compromise the ability of the residential foundation to be able to perform once the home has been totally constructed.

2. <u>Commitment by Subcontractors</u>

   The commitment to quality construction must be included in all contracts for all subcontract work. These contracts should have a specific paragraph in which they agree to comply with all of the quality control requirements submitted by the builder, then in particular, to agree with the inspection program conducted by the builder.

3. Appointment of the Inspector

Every builder must have an inspector designated to represent the interests of the builder. The inspector must report directly to the builder and must be independent of all of the builder's superintendents or subcontractors. The inspector can, of course, be the builder himself; however, many builders have other obligations which may preclude them from effectively carrying out these duties. One builder with which the author is familiar has several superintendents, will appoint one superintendent to be the inspector of the construction activities on another superintendents job. The inspector may be a member of the builder firm or may be an employee of an outside company. In any case, it must be understood by all of the participants in the foundation placement and must be in writing that the inspector has independence over those doing the work and has the authority to stop the work in-process that is in non-compliance with the requirements.

4. Inspection Checklist

A checklist is an essential ingredient for every inspection/inspector and for every inspection process. A checklist must be prepared prior to the time the work is done and must include the following essential activities:

a. The soils report must be reviewed and every activity identified in the soil report which requires some type of inspection must be identified on the list. In some cases, some specific inspection activities must be done by the soil testing agency (e.g., compresion testing).

b. The design drawings must be thoroughly reviewed and provisions made to conduct tests so as to be identified on the drawings. For example, if the drawings limit the slump of the concrete, slump testing should then be conducted. Likewise, concrete strength requirements can only be assured through the digging of concrete cylinder samples which will eventually need to be compression tested. Form placement inspections must be conducted to assure that the configuration is correct and that all design requirements have been satisfied. In particular, the elevation of the top of the forms should be measured using a construction type of transit.

c. A proper equipment inspection is essential to assure that all equipment is in place, that there are sufficient in number of personnel to handle the work, and that all personnel understand their duties and responsibilities, particularly with regards to the placement of the reinforcing steel in/or post-tension cables.

d. The inspector must be present during the entire time of the pour placement, to observe whether or not all requirements are being satisfied. Perhaps one of his most important duties is to ensure that no water is added to any concrete truck unless the need to add such water has been assured through slump testing.

e. After the concrete placement has been completed, the inspector must be sure that the concrete curing process is in accordance with the design requirements.

f. After the concrete has cured to the point where it can be safely walked upon, the relative flatness of the foundation should be measured not only around the perimeter, but at selected interior points as well.

g. The inspectors activities must be documented not only on the checklist, but in notes and drawings as well, and must be securely filed in a safe place in the builders office.

5. Document Management

A file must be maintained which includes all of the documentation associated with the placement of the foundation. Such documentation must include, but is not necessarily limited to, the soil test reports, the design drawings, completed inspection checklists, slab height measurements, etc, Since a builder can be held liable for the performance of a foundation for a period of ten years, the builder must take those measures necessary to protect these documents for at least the same period of time because they may become essential in providing a defense should the performance of the foundation be questioned in the future.

6. Client Follow-up

The importance of the relationship between the builder and his client cannot be overemphasized. This can perhaps be best illustrated by the following example. One of the larger builders in the greater Houston area is absolutely and totally responsive to any complaints or questions provided by persons who have purchased one of the homes that the builder has constructed. An officer of the company will promptly visit the site and listen carefully to the concerns expressed by the homeowner. When such a complaint has been received and there is even a modicum of evidence to support the complaint, the builder then employs an Engineer to inspect the property and provide a written report not only on its condition but with any recommendations the engineer may have with regard to the conditions observed. If the foundation has indeed incurred a performance failure and remedies are necessary, this builder provides such remedies without question and then provides follow-up inspections of the home by that builders engineer to assure that the foundation remedies were effective. To the best of this authors knowledge, this builder has never been sued because of a foundation problem.

*You have your two most important Clients wrapped up in one package - your buyer and your personal integrity.* [Ref. 12]

## CASE HISTORIES

The following actual examples are provided to show just how things can go wrong which can cause litigation problems for the builders involved. Some of these things are very small and some were beyond the control of the builder; yet all created a substantial amount of misery. All of these examples are of cases for which some decision has been made,but to the best of our knowledge are no longer active.

**CASE 1**

THE CONDITIONS

This case involved a residence located in the Clear Lake area which was constructed on a lot which sloped down to a wooded bayou area. The plot plans are shown in Figure 5. The foyer entry led to a large den room with large windows that looked over the terraced landscaped yard to the pool in the bayou area. What had troubled the owner was that the blue floor tile in the den had a propensity to develop cracks; regardless how many times it was replaced by the builder.

THE INVESTIGATION

Our first recommendation was to have a minimum of three concrete cores taken. The measured strengths were 1595 psi, 1738 psi, and 1877 psi; whereas, the specified strength was 2500 psi. The core average of 1737 psi was well below the 2125 psi as required by ACI-318 while the 1595 psi strength was also below the allowable minimum of 1875 psi which is also specified in the ACI Code. The builder went into a state of denial and soon a law suite was filed. The concrete truck tickets were obtained and it was found that the foundation subcontractor had added as much as 100 gallons of water to a 5 yard truck. As the law suit wore on, the foundation began to move causing a substantial amount of damage. Further investigations showed that, in addition to the under strength concrete, there were no bells on the piers and there were sizable gaps between the top of the piers and the bottom of the grade beams.

THE RESULTS

Eventually, the insurance company bought the house back at full market value and paid the owner's attorney a large sum of money.

THE LESSON

Lesson 1 was that the builder totally trusted his subcontractor and failed to have proper inspections done on his behalf. Lesson 2 was that the builder failed to obtain proper engineering advise and, as a result, did not take advantage of the owners good will before the resentment syndrome set in during the litigation process. Lesson 3 was that the defense attorneys misread the owner (who was a retired petro-chemical executive) with regard to his tenacity and ability to sustain the cost of the suit.

**CASE 2**

THE CONDITIONS

This was a residential building which was located in the Southern part of the First Colony Subdivision. The current owner purchased the home when it was approximately 2 years old. He did not have it inspected because it was still under warranty with the builder. When

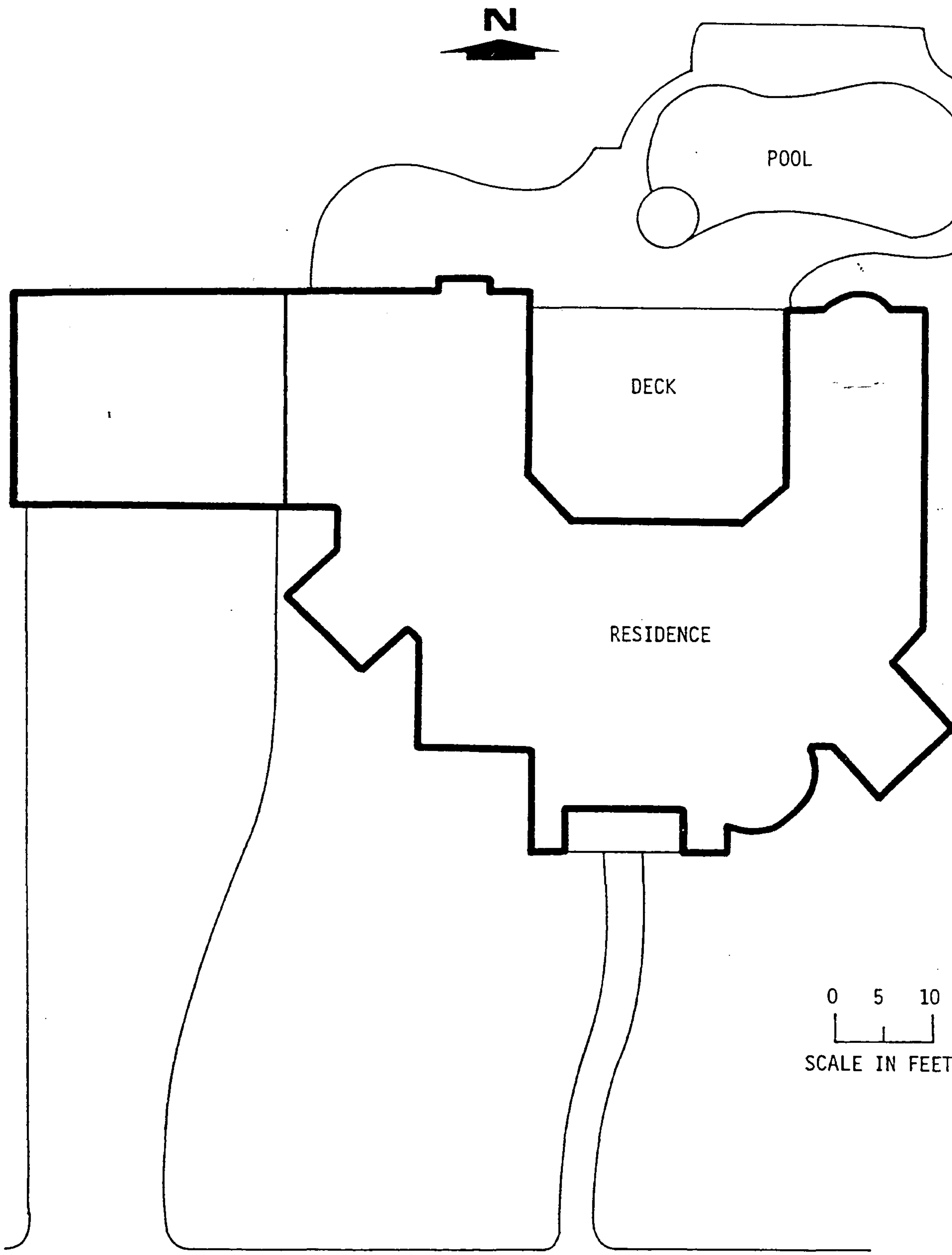



FIGURE 5. A YARD & PLOT PLAN FOR A HOME WHOSE FOUNDATION HAD UNDER-STRENGTH CONCRETE.

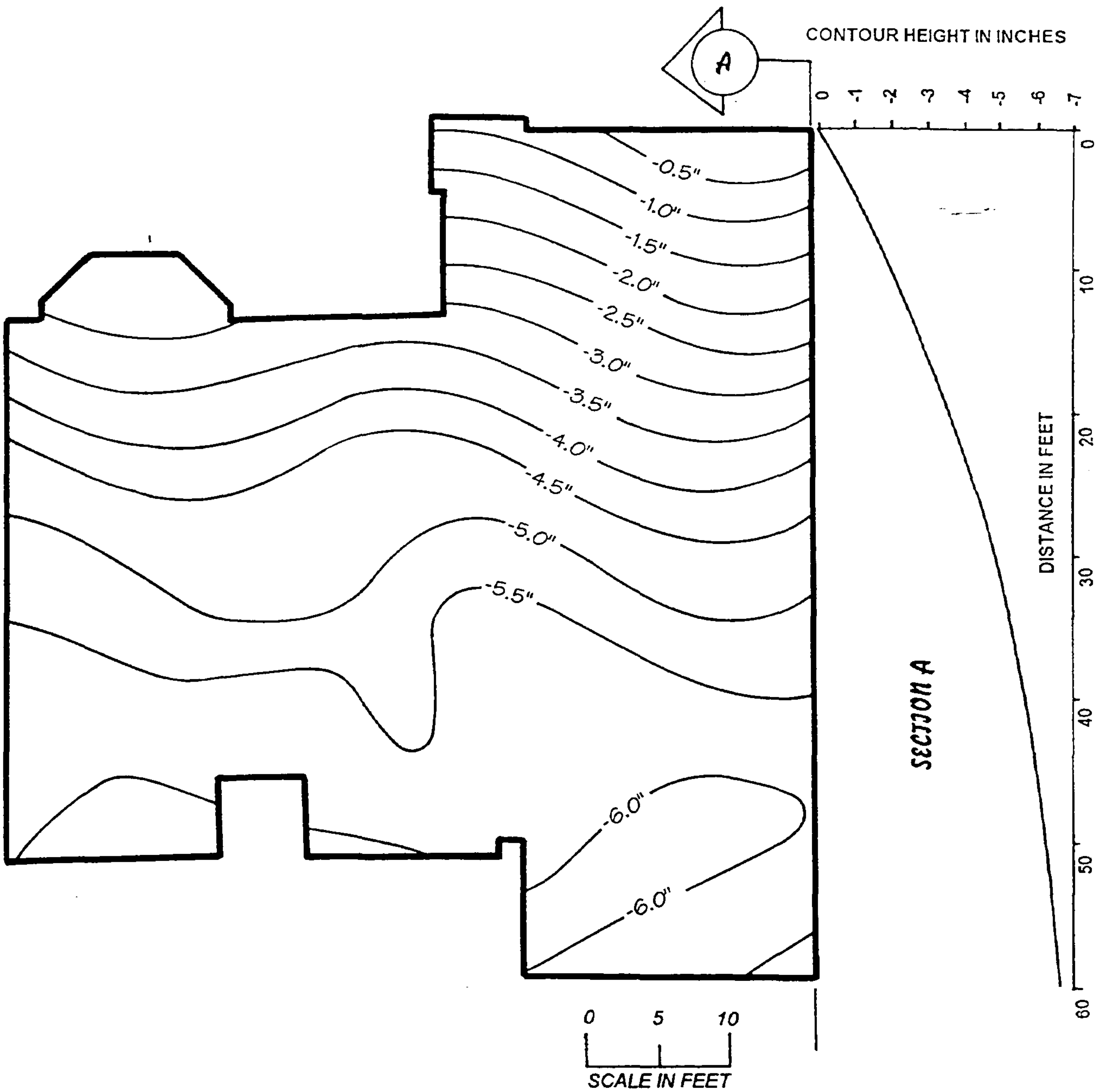



**FIGURE 6. CONTOUR INTERVALS FOR A HOME WITH A 6$^1/_2$ INCH SLOPE ON THE INTERIOR FLOORS**

cracking developed, we were hired and found a $6^1/_2$ inch slope in the floors, as shown in Figure 6.

THE INVESTIGATION

An analysis of the sloping data and an investigation of the damage in the home led us to believe that the sloping of the floors was the result of post-construction deflections. The expert for the insurance carrier opined that the slope was built-in at the time of construction. The builder was not heard from.

THE RESULTS

There were 2 arbitration hearings before arbitrators employed by the insurance carrier. Both found for the insurance company. We understand that the results may be under appeal.

LESSONS LEARNED

The builder never made foundation slab height measurements. The builders insurance carrier did not require that foundation slab height measurements be made. If these were to be the builders first line of defense, such measurements should have been made. In fact, the builder had no idea of what the sloping condition of this foundation was at the time it was constructed.

**CASE 3**

THE CONDITIONS

This was a relatively new home with a pier-and-beam type of foundation. The manifestation of foundation differential movement was that the brick veneer kicked-out at the bottom. Please see Figure 7. The interior damage was minimal as were the slopes on the interior floors.

THE INVESTIGATION

There were 3 unusual conditions in this foundation. First, there were no interior concrete beams between the side walls or between the front and rear walls. Second, the foundation grade beams were tilted outward at the top at the center of the side walls and the center of the front and rear walls. Third, the builder had placed a layer of thick visqueen on top of the expansive soils under the foundation in the crawl-space. At the time of our first visit, this soil was totally saturated. Crawling across the top of the visqueen layer felt like crawling across the top of a water-bed.

THE RESULTS

The visqueen was removed and the outward movements of the brick veneer, along with other forms of damage ceased. It was our belief that the cause of this condition was questionable

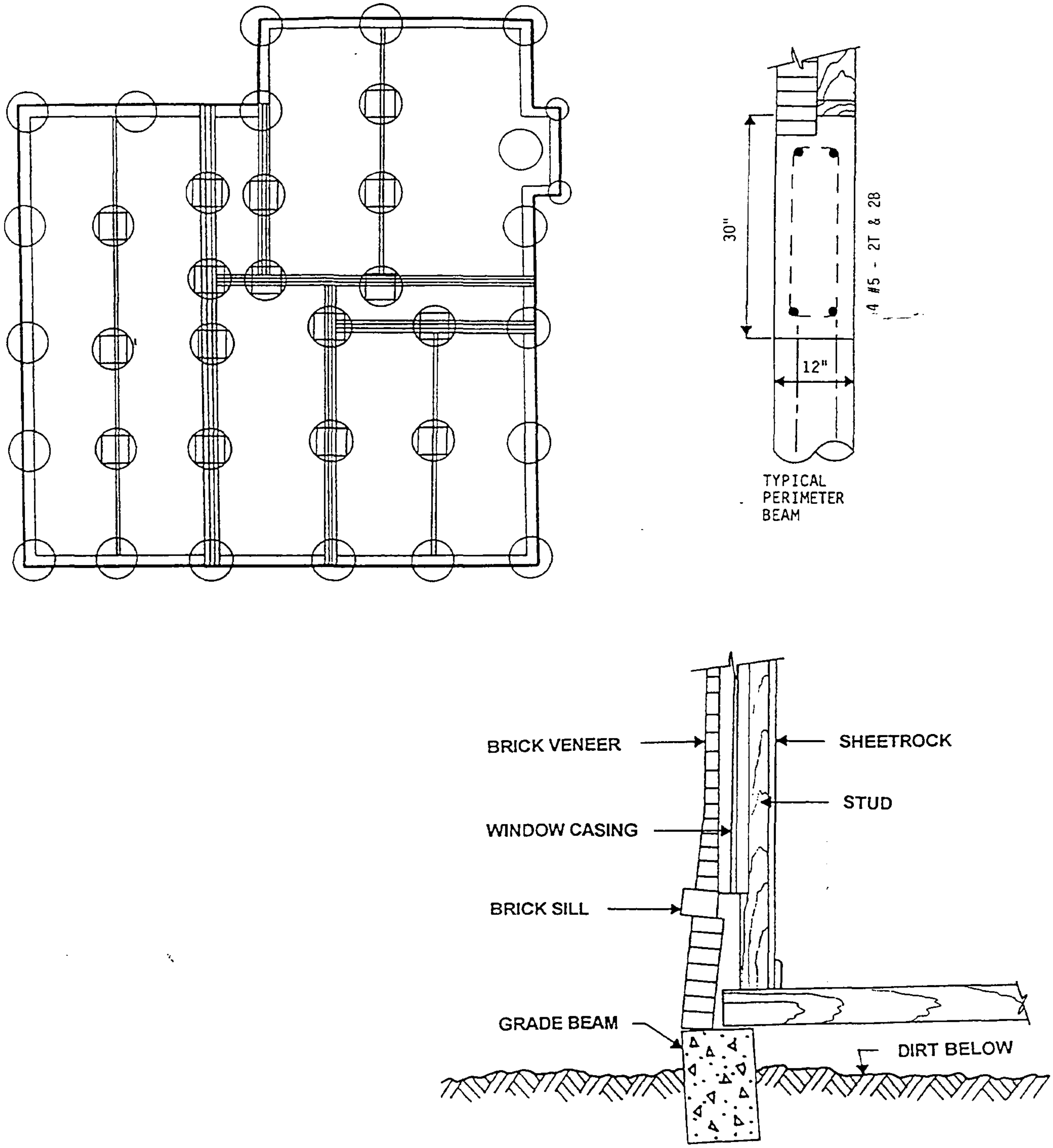



**FIGURE 7. A HOME FOUNDED ON A PIER-&-BEAM FOUNDATION WHOSE WALLS WERE FORCED OUTWARD AT THE BOTTOM APPARENTLY BY SOIL PRESSURES**

foundation design, combined with the placement of the visqueen layer on top of the soil. The builder, or his insurance carrier, paid for the repair of the damage.

### THE LESSON

There was an old TV commercial where mother nature was extolling the virtues of her natural butter. Some obnoxious announcer then said - this is not butter but some brand of margarine ; whereas, mother nature zapped him with a lightening bolt and sweetly said - its not nice to try to fool mother nature. The lesson here then was that the visqueen should not have been placed without finding out the consequences.

## CASE 4

### THE CONDITIONS

This was a relatively new home in Bellaire, Texas which developed a sloping condition, as shown in Figure 8, along with foundation induced damage. The foundation had been placed on top of drilled piers.

### THE INVESTIGATION

There was some negative slope in the back yard which allowed water to pond next to the foundation. Soils testing revealed that bank sand had been used between the top of the soil and the bottom of the slab. When the natural clay soils became saturated, they began to swell. The fill sands also hardened. The upward forced the pier tops and grade beams to separate despite the presence of rebar ties between the two.

### THE RESULTS

The case was settled to the homeowners satisfaction.

### THE LESSON

Bank sand is cheaper than select fill except in cases such as this.

## CASE 5

### THE CONDITIONS

This was a home in Bellaire, Texas. The home was sold while it was under construction by the builder. Later, the home developed symptoms which were typical of differential foundation movements. The owners hired an attorney who hired a construction consultant who, in turn, declared the foundation to be a total failure.

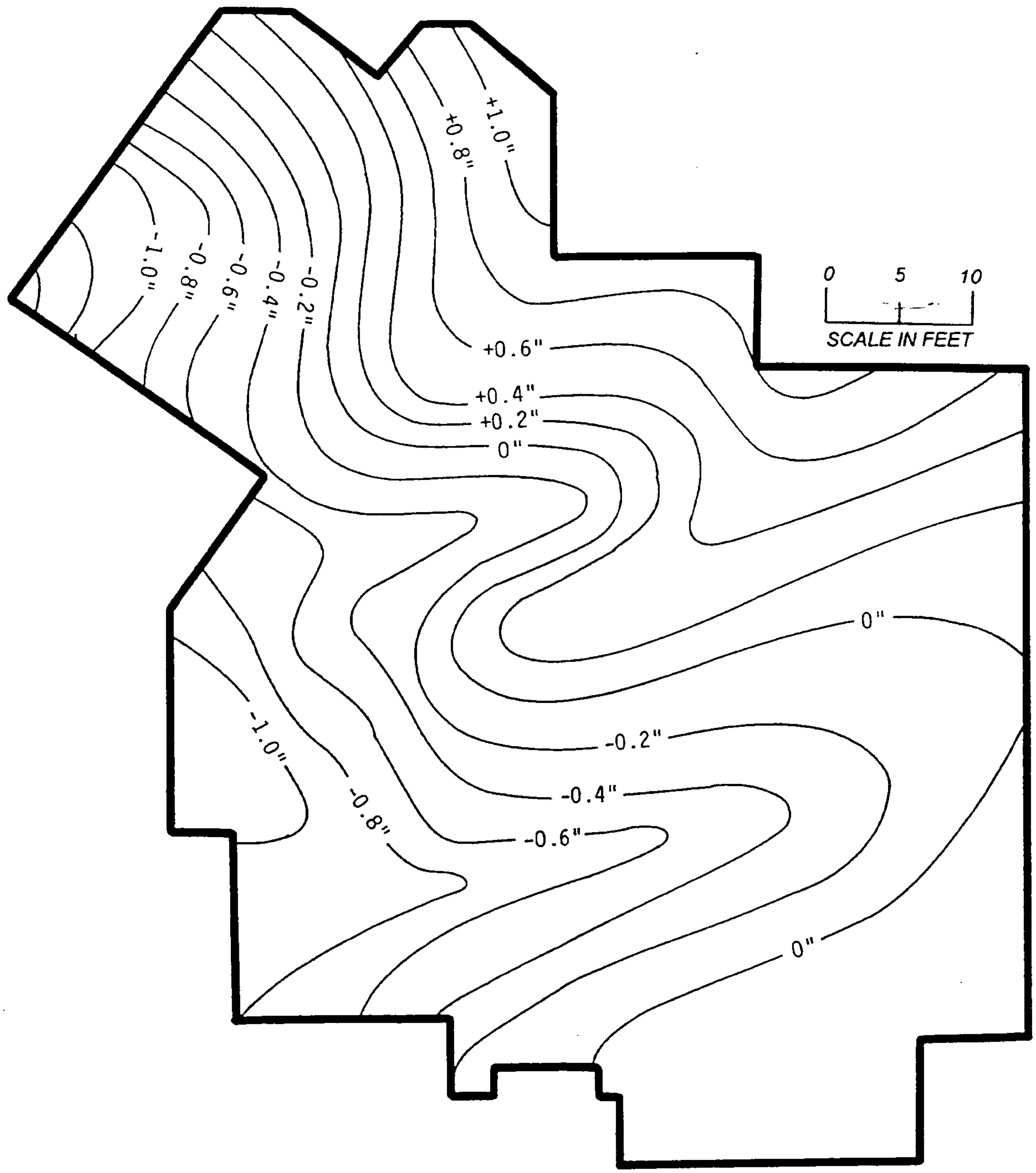



**FIGURE 8. CONTOUR INTERVALS OF A FOUNDATION WHICH HAD BEEN ADVERSELY AFFECTED BY SOIL HEAVING**

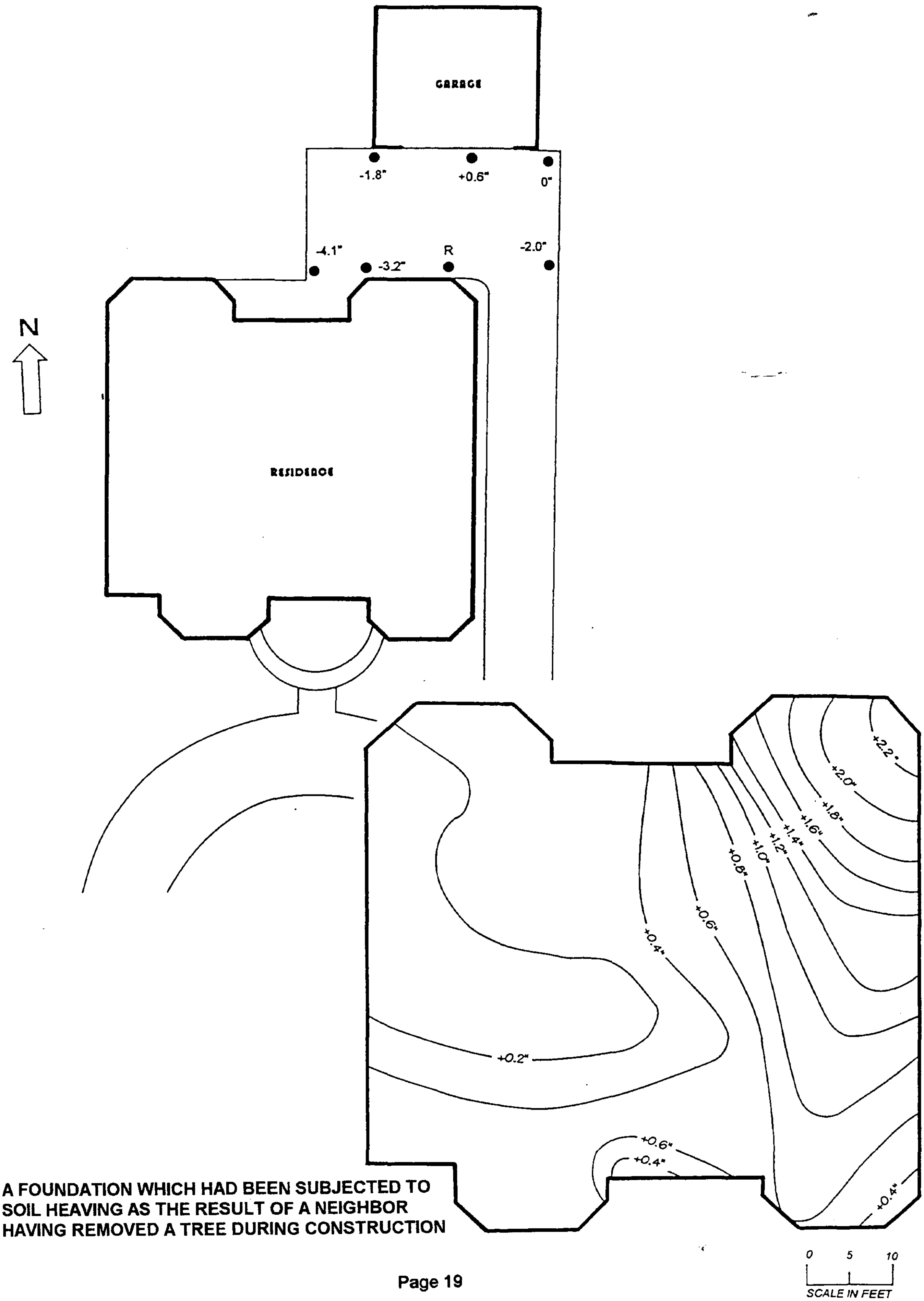



FIGURE 9. A FOUNDATION WHICH HAD BEEN SUBJECTED TO SOIL HEAVING AS THE RESULT OF A NEIGHBOR HAVING REMOVED A TREE DURING CONSTRUCTION

THE INVESTIGATION

The elevation contours are shown in Figure 9. Obviously, the floors show a rise in elevation at the rear of the building. Measurements were made on the flatwork between the garage and the home and these are also shown in Figure 9. All symptoms point towards swelling soils which were likely the result of the removal of a tree. The builder stated that no trees were removed during the construction of the home. The builders attorney, on our advise, questioned the neighbors. The lady who owned the home on the right side informed the attorney that when this nice new house was in the process of being constructed, she worried that her tree might damage the foundation so she had it cut down. The swelling soils resulted.

THE RESULT

The case was settled.

THE LESSON

Be aware of every thing that goes on around you during construction.

**CASE 6**

THE CONDITIONS

This was a home in Southside place which was designed by the owner, who was a graduate architect. The foundation was designed by an Engineer. Shortly after it was constructed, cracking began to appear in the floor of the large den room. Floor measurements showed this portion of the foundation to be heaving. Shortly thereafter, a law suit was filed.

THE INVESTIGATION

After an extensive investigation, some construction defects were observed; none of which could have contributed to the conditions in the den room. During the document production, the City produced a survey of the property as it existed before it was acquired by the owners, who had the lot leveled and the select fill placed. By superimposing the outline of the new home over that of the original home and garage, as shown in Figure 10, it was apparent that a Magnolia tree had been removed before the builder had ever seen the lot. As a result of the tree removal, soil swelling had occurred and was the cause of the problems in the den room.

THE RESULTS

The case was settled. Because of the other defects observed during the investigation, the builders insurance carrier was forced to pay some money to the owners.

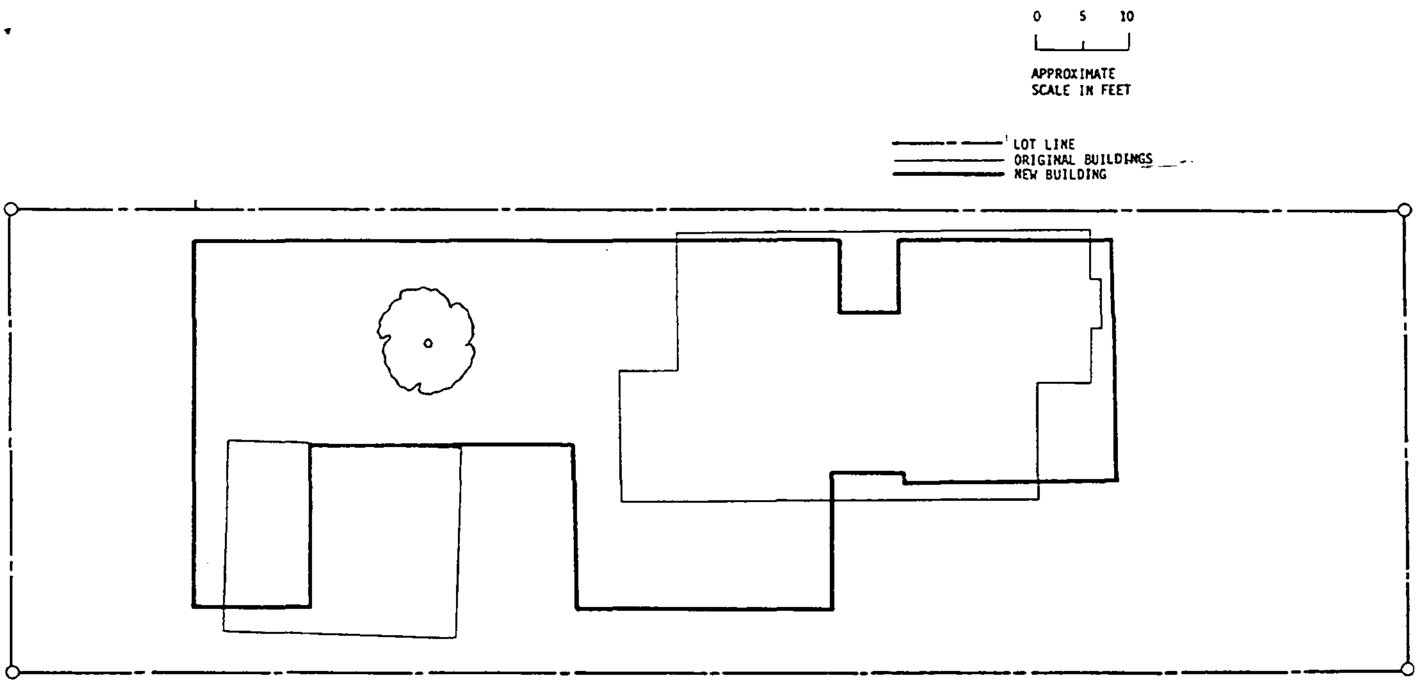



FIGURE 10. THE OUTLINE OF A NEW HOME FOUNDATION SUPERIMPOSED ON THE OUTLINE OF THE ORIGINAL HOUSE & GARAGE. NOTE THAT A MAGNOLIA TREE BETWEEN THE HOUSE & GARAGE ON THE PROPERTY PRIOR TO CONSTRUCTION HAD ALSO BEEN REMOVED. SOIL HEAVING RESULTED.

LESSONS LEARNED

Never assume anything about the building site. Also, remember that a forensic examination will tend to uncover construction defects, which may otherwise not be relevant to the performance of a foundation.

**Case 7**

THE CONDITIONS

The home had been constructed in the general 1960 area. Since it was not within the limits of any city, no permit was required. Soil testing was not done and the foundation plans were not designed by an Engineer. After the foundation had incurred a performance failure, a law suite was filed.

THE INVESTIGATION

The appraiser, who was retained by the owner's attorney, found an old map which showed this building site was once a cattle pond which had been filled in - a fact the builder testified he did not know.

THE RESULTS

The jury in the suite found for the homeowner.

LESSONS LEARNED

Just because there may not be a code requirement for soils testing, they must always be done.

**CASE 8**

THE CONDITIONS

A 10 story concrete condominium building developed cracking and rust bleeding. The original contract called for a wire reinforced structure. Neither the contractor or builder had retained a copy of the construction drawings. The plaintiffs had testing done which showed the balconies to essentially be unreinforced. They also performed a petrographic analysis which showed, in the opinion of the plaintiff's expert, the concrete to be faulty.

THE INVESTIGATION

The design engineer was no longer in business. An employee of the design engineer recalled that the reinforcement was changed to steel bars. A set of reinforcement bar detail drawings showed up in the documentation which was produced. Rebar placement testing was done which not only verified the use of reinforcing steel bars, but showed the balconies to be properly reinforced. A rebar design program was recreated which showed the design to be adequate. A careful examination of the petrographic data showed the plaintiffs expert opinion to be faulty.

THE RESULTS

The case was settled for a modicum of the original demand and the building was repaired and is still occupied.

LESSONS LEARNED

Although this was not, in the strictest sense, a foundation case, the lessons apply to every foundation which was ever constructed. Never, but never, trust anyone else to protect your backsides. Retain for at least 10 years copies of all documents, including even what may appear to be those of minor significance. Also, never assume a so-called expert to be correct.

**CASE 9**

THE CONDITIONS

The home was located in the Southern part of the First Colony Subdivision near the Brazos River. A foundation performance failure occurred which, in turn, caused severe damage. The builder initially denied responsibility and a law suit was filed.

THE INVESTIGATION

The original design was based on a subdivision soils test. Site soil testing then showed the surface to be of a clay constituency which was underlain with bank sand. The builder then hired an Engineer who agreed that foundation repair was required.

THE RESULTS

The foundation was underpinned using Chance helical piers which had to be driven to depths of 24 feet to be able to find sufficient strength.

LESSON LEARNED

Never depend on subdivision soils test result. Never assume that all soils in the greater Houston area are clay down to China.

**CASE 10**

THE CONDITIONS

A home in Southside place was constructed on a pier-and-beam foundation resting on 42 inch diameter bell-bottom piers. The bell diameters were measured by the Engineer of Record. About 2 years into the life of the home, damage began to appear.

THE INVESTIGATION

Floor height measurements were made for 3 years, as shown in Figure 11, which indicated a continuing upward movement of the foundation at its rear wall. It was found that a Pecan tree had been removed by the builder during construction. The piers were found to have moved upward as the result of the soil heaving, which occurred after the removal of the tree, and had not only produced a cessation of the soil desiccation but also prompted its rehydration. The upward movement of one pier, in fact, produced a sufficient force on the foundation grade beam to have caused a fracture at one corner.

THE RESULTS

The case was settled to the dissatisfaction of the owner. Further legale action is anticipated.

LESSONS LEARNED

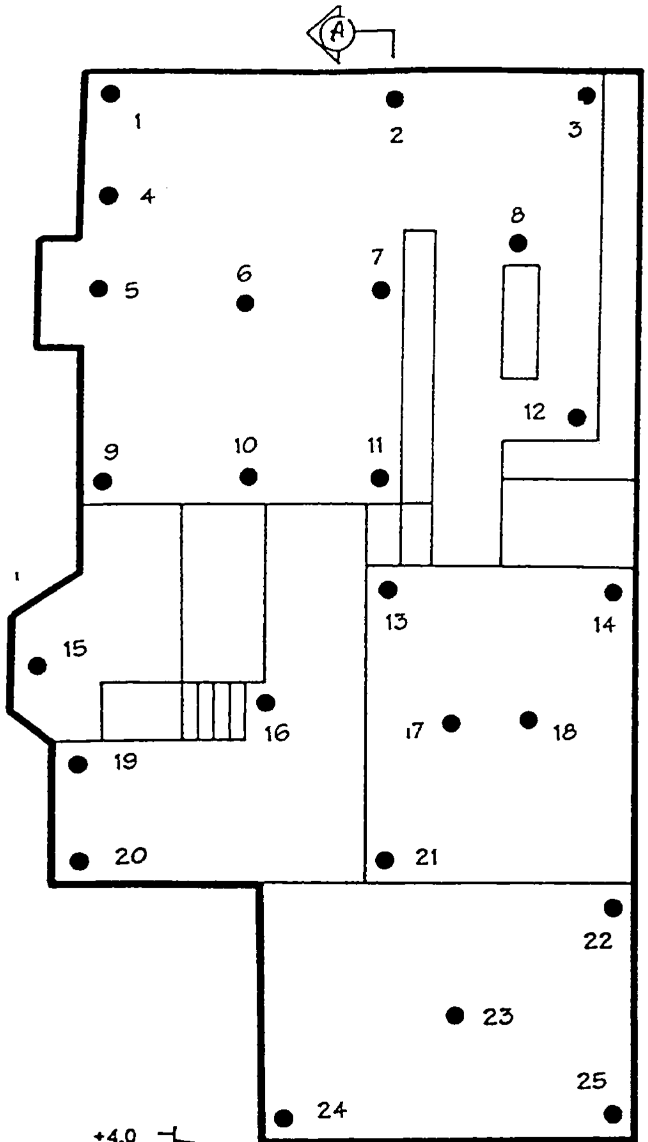
Never assume without verification. The builder assumed that the removal of the tree would enhance the performance of the foundation. The opposite occurred. One issue which was never settled during this law suit involved the extent of the builder's responsibility. The builder pleaded that no one knew at the time the tree was removed, or that it could have a negative impact on foundation performance. Assuming the builder to be correct, who then is responsible for the overall performance of a residential structure. The builder relied on Licensed Engineers to advise him and it was his contention that they did not properly do so. The Exxon Corporation relied on a licensed pilot to successfully guide the tanker Valdez out of the Harbor and he did not. Who paid? Certainly not the pilot.

## CONCLUSIONS

The philosopher George Santayana is credited with the often quoted saying; "Those who cannot remember the past are condemned to repeat it." [Ref. 13] To err once is human, to err twice is damned expensive. The same logic applies to the errors made within the same profession; particularly, when the members of the home building profession pay dues to an organization which could serve them even better by recording and publishing case histories of failures which occurred in the building profession. Perhaps one of the readers of this paper could pass the word on to the Greater Builders Association in this regard.

Structural failures have haunted the structural engineering profession since the first buildings were constructed. There were once seven wonders of the world - now only one survives since the others have fallen down. There has always been an inherent interest in past failures, since they may help to avoid the condemnation of repeating the same mistake.

*An engineer named Mario Salvadori published a book titled Why Buildings Stand Up.* [Ref. 14] *When he presented a copy to his mother-in-law, she said "This is nice, but I would be more interested in reading why they fall down." His next publication was titled, Why Buildings Fall Down.* [Ref. 15]




| MEAS NO. | MEASUREMENT #1 | #2 | #3 | #4 | #5 | #6 |
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| 2 | +1.1 | +1.1 | +1.3 | +1.6 | +2.0 | +3.9 |
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| 20 | +0.1 | +0.1 | 0.0 | +0.1 | +0.2 | +0.2 |
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| 23 | +0.1 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 24 | +0.1 | +0.1 | +0.1 | 0.0 | 0.0 | +0.1 |
| 25 | +0.4 | +0.4 | +0.3 | +0.3 | +0.3 | +0.4 |

ALL MEASUREMENTS ARE IN INCHES.

MEASUREMENT DATES:
NO. 1 NOV 1993 NO. 2 JUL 1994
NO. 3 AUG 1994 NO. 4 JAN 1995
NO. 5 MAY 1995 NO. 6 OCT 1996

PEV-012

*RESIDENCE OUTLINE SHOWING RELATIVE MEASURED HEIGHTS*

*3784 DARCUS*

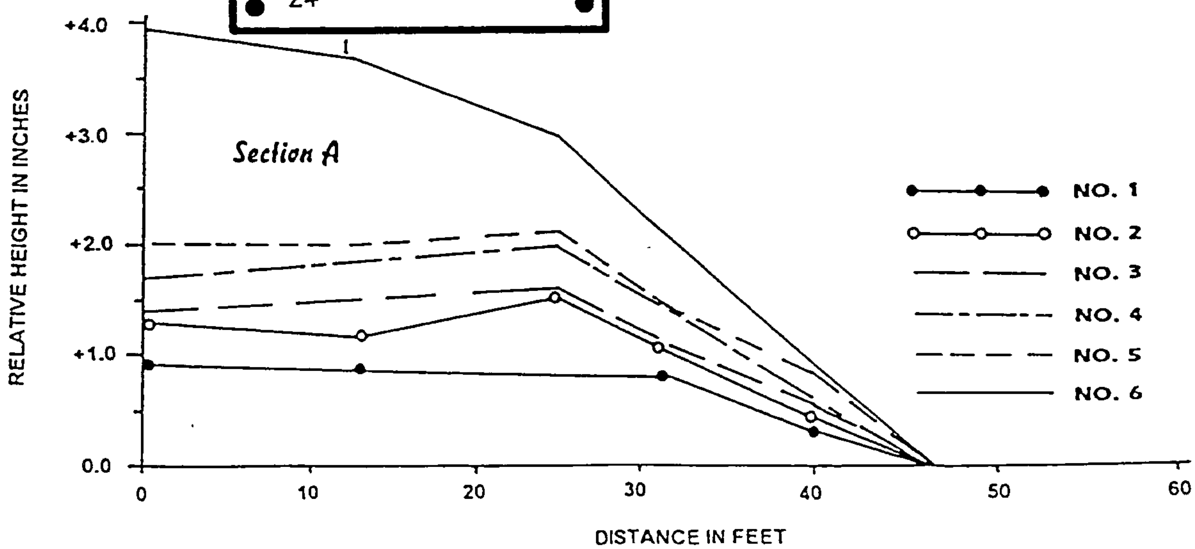


**FIGURE 11. A PIER-&-BEAM FOUNDED ON 42 INCH DIAMETER BELL BOTTOM PIERS WHICH HAD BEEN ADVERSELY AFFECTED BY THE REMOVAL OF A TREE DURING THE CONSTRUCTION PROCESS.**

Unfortunately, these lessons were not always complete. We did learn about soils and we did learn about materials, and we did learn about some construction techniques. The presence of such failures as the Tacoma Narrows Bridge, the Kemper Arena, and the Kansas City Hyatt Regency Balcony have made us realize that there is a whole world of new problems and new environments which will cause future failures to occur. In the work of residential foundations, we do not have to worry about someone dying. We do, however, need to worry about the costs of repairing our mistakes and for simply surviving. We would also like to point out that Santayana also said "We must welcome the future, remembering that soon it will be the past; and we must respect the past, knowing that once it was all that was humanly possible." [Ref.15]

*Why is it we can spend such a large amount of money to repair foundation failures but no money is available for research to keep them from occurring in the first place.* [Ref. 16]

In closing, it is my sincere desire that some of the information contained in this paper will be beneficial in helping the home building business reduce, and even avoid, involvement in the litigation process.

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**LEGAL ISSUES**

**Forensic Materials Testing**

**David Eastwood, PE**

# GEOTECHNICAL GUIDELINES

# FOR

# DESIGN, CONSTRUCTION, MATERIALS AND

# MAINTENANCE OF

# RESIDENTIAL PROJECTS IN THE HOUSTON AREA

**By**


**David A. Eastwood, P.E.**
**Geotech Engineering and Testing, Inc.**


**Presented at the**

**Soils-Structure Interaction Seminar**

**Novmeber 1998**

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## ILLUSTRATIONS

## INTRODUCTION

The variable subsoil conditions in the Gulf Coast area has resulted in very special design requirements for residential and light commercial foundations. The subsurface conditions should be carefully considered when a subdivision or a residence is to be built. Proper planning from the stand point of environmental conditions, subsidence, faulting, soil conditions, design, construction, materials, quality control and maintenance program should be considered prior to any development.

The purpose of this document is to recommend the scope of geotechnical work to develop soils and foundation data for a proper and most economical design and construction of foundations in the Houston area. It is our opinion that portions of these studies should be performed prior to developing the subdivision or buying the lots in order to minimize potential future soils and foundation problems. These problems may arise from the presence of hazardous waste, faulting, poorly compacted fill, soft soil conditions, expansive soils, perched water table, presence of sand and silts, tree roots, etc. This guideline is divided into six segments, including Pre-Development Studies, design, construction, materials, quality control, maintenance program and foundation stabilization. Our recommendations are presented from a geotechnical stand point only and should be complemented by a structural engineer.

## PRE-DEVELOPMENT STUDIES

### Environmental Site Reconnaissance Study

Environmental site assessment studies are recommended on the tracts of land for subdivision or commercial developments. A study like this is generally not required for a single lot in an established subdivision or an in-fill lot in the city. This type of study is used to evaluate the potential risk of environmental contamination that is on or used to be on a project site prior to development. The study is divided into phases, Phases I through III.

The scope of Phase I includes a preliminary site reconnaissance, including: (a) document search, (b) site walk through, (c) review of aerial photographs, (d) historical ownership report, (e) regulatory data review and (f) a report of observations and recommendations.

In the event that the results of the Phase I study indicates the potential for the presence of contaminants, a Phase II study is performed. The scope of Phase II study may include: (a) soil and groundwater sampling, (b) chemical testing and analysis, (c) site reconnaissance, (d) contact with state and federal regulatory personnel, and (e) reporting.

A Phase III study involves implementing the recommendations given in the Phase II study; including remediation and monitoring.

## Subsidence

Potential subsidence problems should be considered when developing subdivisions in the coastal areas, such as Clear Lake, Seabrook, Baytown, etc. Also, other parts of Houston, subject to groundwater removal are also subject to subsidence. This type of study is generally not needed for a single lot in an established subdivision or an in fill lot in the city.

Subsidence is the sinking of the land surface caused by the withdrawal of groundwater. The land elevation lost to subsidence is generally permanent and irreversible. In the Harris-Galveston region of Texas, subsidence poses the greatest threat in the coastal areas susceptible to flooding due to high tides, heavy rainfall and hurricane storm surge. Because of low elevation, any additional subsidence in the coastal areas results in a significant increase in potential tidal flooding or permanent inundation.

The rate of land subsidence in Harris County has been reduced significantly due to changes in water development from the surface water instead of groundwater.

A review of recent subsidence data available from Harris County Subsidence District indicates that the subsidence in areas such as Pasadena, Southwest Houston, etc. have slowed down significantly. However, the subsidence rate in the Addick Area (West, Northwest Houston) is about one-inch per year.

## Geologic Faulting

Many faults have been observed within the Gulf Coast Region of Texas. In general, faults are caused by groundwater and oil removal from the underlying surface. Faults originate several thousand feet below the ground surface and can often cause displacement of the ground surface, causing broken pavement and damage to residential and commercial structures.

Faults are studied in several phases. A Phase I fault study will include the first step in identification of faulting. The scope of a Phase I investigation includes the following elements:

1. Literature Review. This includes a search for, and study of, published data on surface faults in the area of the site.

2. Remote Sensing Study. Aerial photographs, infra-red imagery, where available, should be studied.

3. Field Reconnaissance. This includes a visit to the study area and vicinity by a qualified engineer to examine the area for physical evidence of a possible fault or faults. Physical evidence includes, but is not limited to, (a) natural topographic scarps, (b) soil layer displacements that may be recognized in ditches, creek banks and trenches, (c) breaks in pavements, (d) distress in existing buildings, and (e) vertical offsets in fences.

Once a residence is built on an active fault, the foundation for the residence will be subject to a continual movement and subsequent distress. Foundation stabilization of structures built on active faults can be difficult, but possible. A study of geologic faulting is recommended prior to development of any subdivision in the Gulf-coast area.

## GENERAL SOILS AND GROUNDWATER

### Geology

The Houston area is located on the Gulf of Mexico Coastal Plain, which is underlain largely by overconsolidated clays, clay shales and poorly cemented sands to a depth of several miles. Nearly all soil of the area consists of clay, associated with moderate amounts of sand. Some of the formations in the Houston area consist of Beaumont, Lissie, and Bentley.

The Beaumont formation has significant amounts of expansive clays, resulting in shrink/swell potential. Desiccation of this formation also produces a network of fissures and slickensides in the clay that is potential plains of weakness. The Beaumont formation generally occurs in South, Southwest, East, and Central Houston. The Lissie and Bentley formations generally occur in North and part of West Houston. These formations consist of generally sands and sandy clays. These soils are generally low to moderate in plasticity with low to moderate shrink/swell potential.

### General Soils Conditions

Variable soil conditions occur in the Houston area. These soils are different in texture, plasticity, compressibility, and strength. It is very important that foundations for residential and light commercial structures be designed for subsoil conditions that exists at the specific lot in order to minimize potential foundation and structural distress. Details of general subsoil conditions at various parts of the Houston area are described below. These descriptions are very general. Significant variations from these descriptions can occur. The General soil conditions are as follows:

| Location | Soil Conditions |
|---|---|
| Northwest and Northeast Houston, including Kingwood, The Woodlands, Cypresswood, Copperfield, Atascocita area, Fairfield, Worthom, and Oaks of Devonshire | Generally sandy surficial soils occur in these areas. The sands are generally loose and are underlain by relatively impermeable clays and sandy clays. This condition promotes perched water table formation which results in the loss of bearing capacity of the shallow foundations such as a conventionally-reinforced slab or post-tensioned slabs. This condition also may cause subsequent foundation settlement and distress. |

| | |
|---|---|
| South, Southwest, Southeast, and part of West Houston including, Kirbywoods, parts of the South Shore Harbour, Kelliwood Gardens, Clear Lake area, New territory, Greatwood, First Colony, Brightwater, Vicksburg, Pecan Grove, Woods Edge, Cinco Ranch, and Lake Olympia. | Generally highly plastic clays, and sandy clays are present in these areas. These clays can experience significant shrink and swell movements. The foundations must be designed for this condition. Parts of Cinco Ranch has a surficial layer of sands, underlain by expansive clays. The foundations these soils should be designed, assuming a perched water table condition. |
| Central Houston, including Bellaire, Tanglewood, West University, River Oaks. | Highly expansive clays, drilled footings are the preferred foundations system. Soft soils are observed in some lots. The soils in the River Oaks area are generally moderately expansive. |
| Memorial area, Heights, spring Branch, Hunter's Creek, Bunker Hill, Piney Point, Hedwig Village. | Moderately expansive sandy clays, clays, and sands. Special foundations must be used for structures near ravines. Look for faults. |
| Other Locations: | |
| (a) Weston Lakes, Oyster Creek. | Very sandy soils in some areas, variable soil conditions. Slab-at-Grade is a typical foundation; sometimes piers. Shallow water table at Oyster Creek. Highly expansive soils in parts of Weston Lakes. |
| (b) Sugar Mill, Sugar Creek, Plantation Colony, Quail Valley, Sweetwater. | Highly expansive clays on top of loose silts and sands. Variable soil conditions. A floating slab is a typical foundation. Piers can also be used at some locations. Soft in some lots. Shallow water table. |

Water Level Measurements

The groundwater levels in the Gulf Coast area vary significantly. The groundwater depth in the Houston area generally ranges from 8- to 30-feet. Fluctuations in groundwater level generally occurs as a function of seasonal rainfall variation, temperature, groundwater withdrawal, and construction activities that may alter the surface and drainage characteristics of the site.

The groundwater measurements are usually evaluated by the use of a tape measure and weight at the end of the tape at the completion of drilling and sampling.

An accurate evaluation of the hydrostatic water table in the relatively impermeable clays and low permeability silt/sands requires long term observation of monitoring wells and/or piezometers. It should be noted that it is not possible to accurately predict the pressure and/or level of groundwater, that might occur based upon short-term site exploration. The installation of piezometers/monitor wells is beyond the scope of a typical residential geotechnical reports. We recommend that the groundwater level be verified just before construction if any excavations such as construction of drilled footings/underground utilities, etc. are planned.

The geotechnical engineer must be immediately notified if a noticeable change in groundwater occurs from the one mentioned in the same report. The geotechnical engineer should then evaluate the affect of any groundwater changes on the design and construction of the facilities.

Some of the groundwater problem areas in Houston include Southside Place, parts of Sugar land, etc. One should not confuse the perched water table with the groundwater table. A perched water table occurs when bad drainage exists in areas with a sand or silt layer, about two- to four-foot thick, underlain by impermeable clays and sandy clays. During the wet season, water can pond on the clays and create a perched water table. The surficial sands/silts become extremely soft, wet and may lose their load carrying capacity.

## DESIGN

Foundations and Risks

Many lightly loaded foundations are designed and constructed on the basis of economics, risks, soil type, foundation shape and structural loading. Many times, due to economic considerations, higher risks are accepted in foundation design. Most of the time, the foundation types are selected by the owner/builder, etc. It should be noted that some levels of risk is associated with all types of foundations and there is no such thing as a zero risk foundation. All of these foundations must be stiffened in the areas where expansive soils are present and trees have been removed prior to construction. The following are the foundation types typically used in the area with increasing levels of risk and decreasing levels of cost:

| FOUNDATION TYPE | REMARKS |
|---|---|
| Structural Slab with Piers | This type of foundation (which also includes a pier and beam foundation with a crawl space) is considered to be a minimum risk foundation. A minimum crawl space of six-inches or larger is required. Using this foundation, the floor slabs are not in contact with the subgrade soils. This type of foundation is particularly suited for the areas where expansive soils are present and where trees have been removed prior to construction. The drilled footings must be placed below the potential active zone to minimize potential drilled footing upheaval due to expansive clays. In the areas where non-expansive soils are present, spread footings can be used instead of drilled footings. |
| Slab-On-Fill Foundation Supported on Piers | This foundation system is also suited for the area where expansive soils are present. This system has some risks with respect to foundation distress and movements, where expansive soils are present. However, if positive drainage and vegetation control are provided, this type of foundation should perform satisfactorily. The fill thickness is evaluated such that once it is combined with environmental conditions (positive drainage, vegetation control) the potential vertical rise will be minimum. The structural loads can also be supported on spread footings if expansive soils are not present. |
| Floating (Stiffened) Slab Supported on Piers. The Slab can either be Conventionally-Reinforced or Post-Tensioned | The risk on this type of foundation system can be reduced sizably if it is built and maintained with positive drainage and vegetation control. Due to presence of piers, the slab can move up if expansive soils are present, but not down. In this case, the steel from the drilled piers should not be dowelled into the grade beams. The structural loads can also be supported on spread footings if expansive soils are not present. |
| Floating Slab Foundation (Conventionally-Reinforced or Post-Tensioned Slab) | The risk on this type of foundation can be reduced significantly if it is built and maintained with positive drainage and vegetation control. No piers are used in this type of foundation. Many of the lightly-loaded structures in the state of Texas are built on this type of foundation and are performing satisfactorily. In the areas where trees have been removed prior to construction and where expansive clays exists, these foundations must be significantly stiffened to minimize the potential differential movements as a result of subsoil heave due to tree removal. |

The above recommendations, with respect to the best foundation types and risks, are very general. The best type of foundation may vary as a function of structural loading, house geometry, and soil types. For example, in some cases, a floating slab foundation may perform better than a drilled footing type foundation.

## Foundation Types

Residential structures in the Houston area are supported on drilled footings, post-tensioned slabs, or conventionally reinforced slabs. In general, properly designed post-tensioned slabs or conventionally-reinforced slabs perform satisfactory on most subsoils. Drilled footings may provide a superior foundation system when large slabs, significant offsets or differential loading occurs on the foundations.

The selection of foundation is a function of economics and the level of the risk that the client wants to take. For example, a structural slab foundation is not used for a track home that costs about $100,000. This type of foundation is used for houses that cost usually much more expensive. In general, floating slab type foundations are used with houses with price ranges of less than $200,000 or when subsoil conditions dictates to use this type of foundation.

## Geotechnical Foundation Design Criteria

Foundations for a residential structure should satisfy two independent design criteria. First, the maximum design pressure exerted at the foundation base should not exceed the allowable net bearing pressure based on an adequate factor of safety with respect to soil shear strength. Secondly, the magnitude of total and differential settlements (and shrink and swell) under sustained loads must be such that the structure is not damaged or its intended use impaired.

It should be noted that properly designed and constructed foundation may still experience distress from improperly prepared bearing soils and/or expansive soils which will undergo volume change when correct drainage is not established or an incorrectly controlled water source becomes available.

The design of foundations should be performed by an experienced structural engineer using a soils report from an experienced soils engineer. The structural engineer must use a lot/site specific soils report for the foundation design. The structural engineer should not use general subdivision soils reports written for underground utilities and paving for the slab design. Furthermore, he should not design slabs with disclaimers, requiring future soils reports to verify his design. The designers or architects should not provide clients with foundation design drawings with generic foundations details. All of the foundation drawings should be site and structure specific and sealed by a professional structural engineer.

## Recommended Scope of Geotechnical Studies

Soil testing must be performed on residential lots before a foundation design can be developed. The recommended number of borings should be determined by a geotechnical engineer. The number of borings and the depths are a function of the size of the structure, foundation loading, site features, and soil conditions. As a general rule, a minimum of one boring for every five lots should be performed for subdivision lots. This boring program assumes that a conventionally-reinforced slab or a post-tensioned slab type foundation is going to be used. Furthermore, many lots will be tested at the same time so that a general soils stratigraphy can be developed for the entire subdivision. In the event that a drilled footing foundation is to be used, a minimum of one boring per lot is recommended. In the case of variable subsoil conditions, two or more borings per lot should be performed. A minimum of two borings is recommended for custom homes or a single in-fill lot. A minimum boring depth of 15-feet is recommended for the design of post-tensioned or conventially-reinforced slabs. The boring depths for the design of drilled footing foundations should be at least 15-feet deep. In the event that the lot is wooded and expansive soils are suspected, the boring depth (if drilled footings are to be used) should be increased to 25-ft. On the wooded lots, when the presence of expansive soils are suspected the borings should be drilled near the trees, if possible. Root fibers should be obtained to estimate the active zone depth. The active zone depth is defined as the depth within which seasonal changes in moisture content/soil suction can occur. In general, the depth of active zone is about two-feet below the lowest root fiber depth.

The borings for the residential lots should be performed after the streets are cut and fill soils have been placed and compacted on the lots. This will enable the geotechnical engineer to identify the fill soils that have been placed on the lots. All fill soils should have been tested for compaction during the placement on the lots. A minimum of one density test for every 2500 square feet per lift must be performed once a subdivision is being developed. Fill soils may consist of clays, silty clays, and sandy clays. Sands and silts should not be used as fill materials. Typical structural fill in the Houston area consists of silty clays and sandy clays (not sands) with liquid limits less than 40 and plasticity index between 10 and 20. The fill soils should be placed in lifts not exceeding eight-inches and compacted to 95 percent of the maximum dry density (ASTM D698-91). On-site soils with the exception of sands can also be used as structural fill under floating slab foundations. A floating slab foundation is defined as a conventionally-reinforced slab and a post-tensioned slab.

In the case of a subdivision development, the developer should perform only the borings for the streets and underground utilities. The borings for the lots should wait until all fill soils from street and underground utility excavations are placed and compacted on the lots. In general, the geotechnical testing of the soils for the lots should be the builders responsibility. We recommend that all of the foundations in the subdivision be engineered by a registered professional engineer specializing in residential foundation design.

In the areas where no fill will be placed on the lots prior to site development, the borings on the lots can be performed at the same time as the time as the borings for streets. The soils data from the street and underground borings should never be used for the slab design. This is due to potential in variability in the soil conditions, including soils stratigraphy, compressibility, strength, and swell potential.

Soil borings must be performed prior to foundations underpinning for distressed structures. This is to evaluate the subsoil properties below the bottom of the drilled footings. The depth of drilled footings for foundation underpinning should be determined by a geotechnical engineer. Unfortunately, this is not always followed, and many "so called" foundation repair jobs are performed incorrectly, causing significant financial loss for the client.

In the event of building additions, a minimum of one boring is recommended on residential additions of less than 1,000 square feet. A minimum of two borings is recommended for additions greater than 1,000 square feet.

In general, a scope of typical geotechnical exploration does not include the evaluation of fill compaction. These studies should have been performed at the time of fill placement.

## Foundation Design Considerations

In the areas where highly expansive soils are present, the drilled footings should be founded in a strong soil stratum below the zero movement line. This depth is defined as the depth below which no upward movements occur. It is possible to found a drilled footing below the zero movement line and within the active zone depth. The active zone is defined as the zone within which seasonal changes in subsoil moisture can occur. This is shown on Plate 1. Drilled footings in the area with deep active zones, where trees are present, and subsoils are expansive can be as much as 18-feet deep. The depth of drilled footings should also be determined such that the uplift along the pier shafts be resisted by the presence of bells or shaft skin friction below the zero movement line. The depth of the active zone should be verified by a geotechnical exploration. The evaluation of active zone depths and zero movement line should be performed using the techniques provided in the 1996 Post-Tensioning Institute Slab-on-Grade Design Manual. Drilled footings founded at shallower depths may experience uplift due to expansive soils. In the areas where non-expansive soils are present, the footing depth can be as low as eight-feet.

The grade beams for a floating slab foundation should penetrate the clay soils a minimum of 12-inches. The grade beam penetrations for a floating slab foundation into the surficial sands should be at least 18-inches to develop the required bearing capacity. A minimum grade beam width of 12-inches is recommended in sands and silts.

In the event that a floating slab (post-tensioned slab or a conventionally-reinforced slab) is constructed in sands or silts, the geotechnical engineer must specify bearing capacity, assuming saturated subsoil conditions. This results in bearing capacities in the range of 600- to 900 psf in a typical sand or silt soils in the Houston area. Higher bearing capacity values can be used if the sands/silts do not get saturated during the life of the residence. This assumption is generally unrealistic due to the presence of sprinkler systems, negative drainage, and cyclic rainfall in the Houston area.

Design parameters for a post-tensioned slab on expansive clays must carefully evaluated by a geotechnical engineer. It should be noted that the 1996 post-tensioned slab design manual does not directly model the poor drainage, the effect of the trees, and the depth of the active zone. The geotechnical engineer must modify the design parameter presented in the manual to come up with the proper design parameter. It should be noted that it is currently very difficult (to impossible) to design economical floating slab foundations on expansive soils on wooded lots where trees are to be removed prior to slab construction.

## Floor Slabs

The floor slabs for foundations supported on drilled footings may consist of (a) structural slabs with crawl space, (b) slab-on-fill or (c) slab-on-grade.

A structural slab should be used when a minimum risk foundation is to be used. This type of floor slabs are generally expensive. A slab-on-fill will be less expansive than a structural slab with crawl space. The fill thickness in areas where expansive soils are present should be about 18-to 48-inches. The higher fill thickness should be used in areas such as Bellaire, Tanglewood, New Territory, etc, where highly expansive clays exists (plasticity indices above 50).

In the event that a structural slab foundation is used, the crawl space area should be properly drained so that any water would drain towards the exterior grade beams. Furthermore, the area should be properly vented.

The floor slabs can be supported at grade on drilled footings if the subsoils are non-expansive. All of the subgrade soils should be prepared in accordance to the soils report site preparation section prior to fill placement.

## Void Boxes

Void boxes are historically used under the grade beams to separate the expansive soils from the grade beams. The void boxes collapse once the underlying expansive soils swell up; thereby minimizing uplift loads as a result of expansive soils on the grade beams. This can be an effective feature for reducing potential pressures on grade beams.

In areas of poor drainage, void boxes may act as a pathway for water to travel under a foundation system. This condition may result in an increase in subsoil moisture contents and subsequent swelling of the soils. This may result in uplift loads on the floor slabs, and subsequent distress to the foundation and structural system.

We recommend that the decision on whether or not to use void boxes be made by the owner/builder after both the positive and negative aspects of this issue are evaluated. Based on our and other experts personal experience with void boxes, it is our opinion that they will not provide an effective feature for reducing swell pressure on the grade beams. In general, the validity of void box usage is presently being questioned because of the frequency of observed negative effects which may outweigh its benefits.

## Site Drainage

It is recommended that site drainage be well developed. Surface water should be directed away from the foundation soils (use a slope of about 5% within 10-feet of foundation). No ponding of surface water should be allowed near the structure.

Residential Structures Constructed near the Bayous

Many large residential structures are being build near the bayous. Portions of the slopes on the bayous are very steep with slopes steeper than 3(h):1(v). The foundations for residences near the bayous must be provided by the use of deep drilled footings/piling. The geotechnical boring depths should be at least twice the depth of the bayou.

Any foundation which falls within the hazard zone which extends from the toe of the slope, extending backward on a 4(h):1(v) slope to the existing grade should be supported on deep foundations. Foundations outside the hazard zone may be supported on shallow piers. The floor slabs in the hazard zone should consist of a structural slab. The floor slabs outside the hazard zone may consist of slab-on -fill or slab-on-grade. No skin friction should be used for piers within the hazard zone from the surface to the toe of the slope elevation.

We recommend the stability of bayou slopes be evaluated using a slope-stability analyses, using computer solutions. The house should be placed on top of the slope and the stability of the slope for global stability should be evaluated. The slopes should then be flattened and covered with erosion protection to minimize potential sloughing and erosion problems.

## CONSTRUCTION

Site Preparation

Our recommendations on site preparation are summarized below:

1. In general, remove all vegetation, tree roots, organic topsoil, existing foundations, paved areas and any undesirable materials from the construction area. Tree trunks under the floor slabs should be removed to a root size of less than 0.5-inches. We recommend that the stripping depth be evaluated at the time of construction by a soil technician.

2. Any on-site fill soils, encountered in the structure and pavement areas during construction, must have records of successful compaction tests signed by a registered professional engineer that confirms the use of the fill and record of construction and earthwork testing. These tests must have been performed on all the lifts for the entire thickness of the fill. In the event that no compaction test results are available, the fill soils must be removed, processed and recompacted in accordance with our site preparation recommendations. Excavation should extend at least two-feet beyond the structure and pavement area. Alternatively, the existing fill soils should be tested comprehensively to evaluate the degree of compaction in the fill soils.

3. The subgrade areas should then be proofrolled with a loaded dump truck, scraper, or similar pneumatic-tired equipment. The proofrolling serves to compact surficial soils and to detect any soft or loose zones. Any soils deflecting excessively under moving loads should be undercut to firm soils and recompacted. The proofrolling operations should be observed by an experienced geotechnician.

4. Scarify the subgrade, add moisture, or dry if necessary, and recompact to 95% of the maximum dry density as determined by ASTM D 698-91 (Standard Proctor). The moisture content at the time of compaction of subgrade soils should be within -1 to +3% of the proctor optimum value. We recommend that the degree of compaction and moisture in the subgrade soils be verified by field density tests at the time of construction. We recommend a minimum of four field density tests per lift or one every 2500 square feet of floor slab areas, whichever is greater.

5. Structural fill beneath the building area may consist of off-site inorganic silty clays or sandy clays with a liquid limit of less than 40 and a plasticity index between 10 and 20. In the event that a floating slab foundation system is used, on-site soils (with the exception of sands or silts), free of organics, can be used as structural fill. Other types of structural fill available locally, and acceptable to the geotechnical engineer, can also be used.

   These soils should be placed in loose lifts not exceeding eight-inches in thickness and compacted to 95 percent of the maximum dry density determined by ASTM D 698-91 (Standard Proctor). The moisture content of the fill at the time of compaction should be within $\pm 2\%$ of the optimum value. We recommend that the degree of compaction and moisture in the fill soils be verified by field density tests at the time of construction. We recommend that the frequency of density testing be as stated in Item 4.

6. The backfill soils in the trench/underground utility areas should consist of select structural fill, compacted as described in Item 4. In the event of compaction difficulties, the trenches should be backfilled with cement-stabilized sand or other materials approved by the Geotechnical Engineer. Due to high permeability of sands and potential surface water intrusion, bank sands should not be used as backfill material in the trench/underground utility areas.

7. In cut areas, the soils should be excavated to grade and the surface soils proofrolled and scarified to a minimum depth of six-inches and recompacted to the previously mentioned density and moisture content.

8. The subgrade and fill moisture content and density must be maintained until paving or floor slabs are completed. We recommend that these parameters be verified by field moisture and density tests at the time of construction.

9. In the areas where expansive soils are present, rough grade the site with structural fill soils to insure positive drainage. Due to their high permeability of sands, sands should not be used for site grading where expansive soils are present.

10. We recommend that the site and soil conditions used in the structural design of the foundation be verified by the engineer's site visit after all of the earthwork and site preparation has been completed and prior to the concrete placement.

Other Construction Considerations

1. Grade beam excavations should be free of all loose materials. The bottom of the excavations should be dry and hard.

2. Surficial subgrade soils in the floor slab areas should be compacted to a minimum of 95% of Standard Proctor Density (ASTM D 698-91). This should be confirmed by conducting a minimum of four field density tests per slab, per lift.

3. Minimum concrete strength should be 3,000 psi with a maximum slump of 5-inch. Concrete workability can be improved by adding air to the concrete mix and the use of a concrete vibrator. The concrete slump and strength should be verified by slump tests and concrete cylinders.

4. The Visqueen, placed under the floor slabs, should be properly stretched to maximize soil-slab interaction.

5. In the areas where expansive soils are present, the backfill soils for the underground utilities under the floor slabs should consist of select fill and not sands or silts. The cohesionless backfill can act as a pathway for surface water to get under the foundation and resulting in subsoil swelling. In the event that a floating slab is used, on-site soils (not sands or silts), free of organics, can be used as structural fill.

6. Tree stumps should not be left under the slabs. This may result in future settlement and termite infestation.

## MATERIALS

The use of proper materials is crucial to the performance of a foundation system. Some of the relevant material issues is as follows:

- Inadequate concrete strength.

- Reinforcement, steel grade.

- o Improperly manufactured post-tensioned materials.

- o The geotechnical technician must check the earthwork testing, concrete pier, installation, and concrete placement.

## QUALITY CONTROL

General

Construction monitoring and quality control tests should be planned to verify materials and placement in accordance with the project design documents and specifications. Earthwork observations on the house pad, pad thickness measurements, drilled footing installation monitoring, and concrete placement monitoring should be performed. Details of each of these items is described in the following paragraphs.

Earthwork Observations

The subgrade and fill soils under the floor slabs should be compacted to about 95 percent of maximum dry density (ASTM D 698-91). Furthermore, the fill soils should be non-expansive. Atterberg limit tests should be performed on the fill soils, obtained from the borrow pit, to evaluate the suitability of these soils for use as structural fill and their shrink/swell potential. Expansive soils, of course, should not be used as structural fill. In the event that a floating slab foundation is used, on-site soils with the exception of sands/silts can be used as structural fill.

Field density tests should be conducted on the subgrade soils and any borrow fill materials in the floor slab and pavement areas. In the areas where expansive soils are present, about 18- to 36-inches of structural fill is placed under the floor slab areas. Laboratory proctor tests will also be performed on the on-site soils as well as off-site borrow fill materials to evaluate the moisture-density relationship of these soils.

Fill Thickness Verification

Fill soils may have to be placed on the lots to raise the lot or to provide a buffer zone in between the on-site expansive soils and the floor slabs. We recommend that the required thickness of the fill be verified after the completion of the building pad. This task can be accomplished by drilling two borings to a depth of two-feet in the building pad area, examining and testing the soils to verify the fill thickness.

Drilled Footing Observations

In the event that the structure is supported by drilled footings, we recommend that the installation of the footings be observed by a geotechnical technician.

The technician will conduct hand penetrometer tests on the soil cuttings to estimate the bearing capacity of the soil at each footing location. He will make changes to the foundation depth and dimensions if obstacles or soft soils are encountered. Therefore, minimizing costly construction delays. In addition, the technician must verify the bell size by a bell measurement tool. One set of concrete cylinders (four cylinders) will be made for each 50 yards of pour. Two cylinders will be broken at seven days, and two cylinders at 28 days.

Concrete Placement Monitoring

The concrete sampling and testing in the floor slab and placement areas will be conducted in accordance with ASTM standards. A technician will monitor batching and placing of the concrete. At least four concrete cylinders should be made for each 50 yards floor slab pour. Two concrete cylinders are tested at seven days and two cylinders at 28 days.

## HOMEOWNER MAINTENANCE PROGRAM

Introduction

Performance of residential structures depends not only on the proper design and construction, but also on the proper foundation maintenance program. Many residential foundations have experienced major foundation problems as a result of owner's neglect or alterations to the initial design, drainage, or landscaping. This has resulted in considerable financial loss to the homeowners, builders, and designers in the form of repairs and litigation.

A properly designed and constructed foundation may still experience distress from vegetation and expansive soil which will undergo volume change when correct drainage is not established or incorrectly controlled water source becomes available.

The purpose of this document is to present recommendations for maintenance of properly designed and constructed residential projects in Houston. It is recommended that the builder submit this document to his/her client at the time that the owner receives delivery of the house.

Drainage

The initial builder/developer site grading (positive drainage) should be maintained during the useful life of the residence. In general, a civil engineer develops a drainage plan for the whole subdivision. Drainage sewers or other discharge channels are designed to accommodate the water runoff. These paths should be kept clear of debris such as leaves, gravel, and trash.

In the areas where expansive soils are present, positive drainage should be provided away from the foundations. Changes in moisture content of expansive soils are the cause of both swelling and shrinking. Positive drainage should also be maintained in the areas where sandy soils are present.

Positive drainage is extremely important in minimizing soil-related foundation problems.

The homeowners berm the flowerbed areas, creating a dam between the berm and the foundation, preventing the surface water from draining away from the structure. This condition may be visually appealing, but can cause significant foundation damage as a result of negative drainage.

The most commonly used technique for grading is a positive drainage away from the structure to promote rapid runoff and to avoid collecting ponded water near the structure which could migrate down the soil/foundation interface. This slope should be about 3 to 5 percent within 10-feet of the foundation.

Should the owner change the drainage pattern, he should develop positive drainage by backfilling near the grade beams with fill compacted to 90 percent of the maximum dry density as determined by ASTM D 698-91 (standard proctor). This level of compaction is required to minimize subgrade settlements near the foundations and the subsequent ponding of the surface water. The fill soils should consist of silty clays and sandy clays with liquid limits less than 40 and plasticity index (PI) between 10 and 20. Bank sand or top soils are not a select fill. The use of Bank sand or top soils to improve drainage away from a house is discouraged; because, sands are very permeable. In the event that sands are used to improve drainage away from the structure, one should make sure the clay soils below the sands have a positive slope (3 - 5 Percent) away form the structure, since the clay soils control the drainage away from the house. The on-site soils (not sand or silts), free of organics, can be used as structural fill.

The author has seen many projects with an apparent positive drainage; however, since the drainage was established with sands on top of the expansive soils the drainage was not effective.

Depressions or water catch basin areas should be filled with compacted soil (sandy clays or silty clays not bank sand) to have a positive slope from the structure, or drains should be provided to promote runoff from the water catch basin areas. Six to twelve inches of compacted, impervious, nonswelling soil placed on the site prior to construction of the foundation can improve the necessary grade and contribute additional uniform surcharge pressure to reduce uneven swelling of underlying expansive soil.

Pets (dogs, etc.) sometimes excavate next to the exterior grade beams and created depressions and low spots in order to stay cool during the hot season. This condition will result in ponding of the surface water in the excavations next to the foundation and subsequent foundation movements. These movements can be in the form of uplift in the area with expansive soils and settlement in the areas with sandy soils. It is recommended as a part of the foundation maintenance program, the owner backfills all excavations created by pets next to the foundation with compacted clay fill.

Grading and drainage should be provided for structures constructed on slopes, particularly for slopes greater than nine percent, to rapidly drain off water from the cut areas and to avoid ponding of water in cuts or on the uphill side of the structure. This drainage will also minimize seepage through backfills into adjacent basement walls.

Subsurface drains may be used to control a rising water table, groundwater and underground streams, and surface water penetrating through pervious or fissured and highly permeable soil. Drains can help control the water table in the expansive soils. Furthermore, since drains cannot stop the migration of moisture through expansive soil beneath foundations, they will not prevent long-term swelling. Moisture barriers can be placed near the foundations to minimize moisture migration under the foundations. The moisture barriers should be at least five-feet deep in order to be effective.

Area drains can be used around the house to minimize ponding of the surface water next to the foundations. The area drains should be checked periodically to assure that they are not clogged.

The drains should be provided with outlets or sumps to collect water and pumps to expel water if gravity drainage away from the foundation is not feasible. Sumps should be located well away from the structure. Drainage should be adequate to prevent any water from remaining in the drain (i.e., a slope of at least 1/8 inch per foot of drain or 1 percent should be provided).

Positive drainage should be established underneath structural slabs with crawl space. This area should also be properly vented. Absence of positive drainage may result in surface water ponding and moisture migration through the slab. This may result in wood floor warping and tile unsticking. Furthermore, The crawl space area should be properly vented.

It is recommended that at least six-inches of clearing be developed between the grade and the wall siding. This will minimize surface water entry between the foundation and the wall material, in turn minimizing wood decay.

Poor drainage at residential projects in North and West Houston can result in saturation of the surficial sands and development of a perched water table. The sands, once saturated, can lose their load carrying capacity. This can result in foundation settlements and bearing capacity failures. Foundations in these areas should be designed assuming saturated subsoil conditions.

In general, roof drainage systems, such as gutters or rain dispenser devices, are recommended all around the roof line when gutters and downspouts should be unobstructed by leaves and tree limbs. In the area where expansive soils are present, the gutters should be connected to flexible pipe extensions so that the roof water is drained at least 10-feet away from the foundations. Preferably the pipes should direct the water to the storm sewers. In the areas where sandy soils are present, the gutters should drain the roof water at least five-feet away from the foundations.

If a roof drainage system is not installed, rain-water will drip over the eaves and fall next to the foundations resulting in subgrade soil erosion, and creating depression in the soil mass, which may allow the water to seep directly under the foundation and floor slabs.

The home owner must pay special attention to leaky pools and plumbing. In the event that the water bill goes up suddenly without any apparent reason, the owner should check for a plumbing leak.

The introduction of water to expansive soils can cause significant subsoil movements. The introduction of water to sandy soils can result in reduction in soil bearing capacity and subsequent settlement. The home owner should also be aware of water coming from the air conditioning drain lines. The amount of water from the condensating air conditioning drain lines can be significant and can result in localized swelling in the soils, resulting in foundation distress.

Landscaping

General. A house with the proper foundation, and drainage can still experience distress if the homeowner does not properly landscape and maintain his property. One of the most critical aspects of landscaping is the continual maintenance of properly designed slopes.

Installing flower beds or shrubs next to the foundation and keeping the area flooded will result in a net increase in soil expansion in the expansive soil areas. The expansion will occur at the foundation perimeter. It is recommended that initial landscaping be done on all sides, and that drainage away from the foundation should be provided and maintained. Partial landscaping on one side of the house may result in swelling on the landscaping side of the house and resulting differential swell of foundation and structural distress in a form of brick cracking, windows/door sticking, and slab cracking.

Landscaping in areas where sandy, non-expansive soils are present, with flowers and shrubs should not pose a major problem next to the foundations. This condition assumes that the foundations are designed for saturated soil conditions. Major foundation problems can occur if the planter areas are saturated as the foundations are not designed for saturated (perched water table) conditions. The problems can occur in a form of foundation settlement, brick cracking, etc.

Sprinkler Systems. Sprinkler systems can be used in the areas where expansive soils are present, provided the sprinkler system is placed all around the house to provide a uniform moisture condition throughout the year.

The use of a sprinkler system in parts of Houston where sandy soils are present should not pose any problems, provided the foundations are designed for saturated subsoil conditions with positive drainage away from the structure.

The excavations for the sprinkler system lines, in the areas where expansive soils are present, should be backfilled with impermeable clays. Bank sands or top soil should not be used as backfill. These soils should be properly compacted to minimize water flow into the excavation trench and seeping under the foundations, resulting in foundation and structural distress.

The sprinkler system must be checked for leakage at least once a month. Significant foundation movements can occur if the expansive soils under the foundations are exposed to a source of free water.

The homeowner should also be aware of damage that leaking plumbing or underground utilities can cause, if they are allowed to continue leaking and providing the expansive soils with the source of water.

Effect of Trees. The presence of trees near a residence is considered to be a potential contributing factor to the foundation distress. Our experience shows that the presence or removal of large trees in close proximity to residential structures can cause foundation distress. This problem is aggravated by cyclic wet and dry seasons in the area. Foundation damage of residential structures caused by the adjacent trees indicates that foundation movements of as much as 3- to 7-inches can be experienced in close proximity to residential foundations.

This condition will be more severe in the periods of extreme drought. Sometimes the root system of trees such as willow, elm, or oak can physically move foundations and walls and cause considerable structural damage. Root barriers can be installed near the exterior grade beams to a minimum depth of 36-inches, if trees are left in place in close proximity to foundations. It is recommended that trees not be planted closer than half the canopy diameter of the mature tree, typically 20-feet from foundations. Any trees in closer proximity should be thoroughly soaked at least twice a week during hot summer months, and once a week in periods of low rainfall. More frequent tree watering may be required.

Tree roots tend to desiccate the soils. In the event that the tree has been removed prior to house construction, subsoil swelling can occur for several years. Studies have shown that for certain types of trees this process can last as much as 20 years in the areas where highly expansive clays are present. In this case the foundation for the house should be designed for the anticipated maximum heave.

Furthermore, the drilled footings, if used, must be placed below the zone of influence of tree roots. In the event that a floating slab foundation is used, we recommend the slab be stiffened to resist the subsoil movements due to the presence of trees. In addition, the area within the tree root zone may have to be chemically stabilized to reduce the potential movements. Alternatively, the site should be left alone for several years so that the moisture regime in the desiccated areas of the soils (where tree roots used to be) become equal/stabilize to the surrounding subsoil moisture conditions.

Tree removal can be safe provided the tree is no older than any part of the house, since the subsequent heave can only return the foundation to its original level. In most cases there is no advantage to a staged reduction in the size of the tree and the tree should be completely removed at the earliest opportunity. The areas where expansive soils exist and where the tree is older than the house, or there are more recent extensions to the house, it is not advisable to remove the tree because the danger of inducing damaging heave; unless the foundation is designed for the total computed expected heave.

In general, in the areas where non-expansive soils are present, no foundation heave will occur as a result of the tree removal.

In the areas where too much heave can occur with tree removal, some kind of pruning, such as crown thinning, crown reduction or pollarding should be considered. Pollarding, in which ~~most~~ of the branches are removed and the height of the main trunk is reduced, is often mistakenly specified, because most published advice links the height of the tree to the likelihood of damage. In fact the leaf area is the important factor. Crown thinning or crown reduction, in which some branches are removed or shortened, is therefore generally preferable to pollarding. The pruning should be done in such a way as to minimize the future growth of the tree, without leaving it vulnerable to disease (as pollarding often does) while maintaining its shape. This should be done only by a reputable tree surgeon or qualified contractor working under the instructions of an arboriculturist.

You may find there is opposition to the removal or reduction of an offending tree; for example, it may belong to a neighbor or the local authority, or have a Tree Preservation Order on it. In such cases there are other techniques that can be used from within your own property.

One option is root pruning, which is usually performed by excavating a trench between the tree and the damaged property deep enough to cut most of the roots. The trench should not be so close to the tree that it jeopardizes its stability. In time, the tree will grow new roots to replace those that are cut; however, in the short term there will be some recovery as the degree of desiccation in the soil under the foundations reduces.

Where the damage has only appeared in a period of dry weather, a return to normal weather pattern may prevent further damage occurring. Permission from the local authority is required before pruning the roots of a tree with preservation order on it.

Root barriers are a variant of root pruning. However, instead of simply filling the trench with soil after cutting the roots, the trench is either filled with concrete or lined with an impermeable layer to form a "permanent" barrier to the roots. Whether the barrier will be truly permanent is questionable, because the roots may be able to grow round or under the trench. However, the barrier should at least increase the time it takes for the roots to grow back.

### Foundations/Flat Works

Every homeowner should conduct a yearly observation of foundations and flat works and perform any maintenance necessary to improve drainage and minimize infiltrations of water from rain and lawn watering. This is important especially during the first six years of a newly built home because this is usually the time of the most severe adjustment between the new construction and its environment. We recommend that all of the separations in the flat work and paving joints be immediately backfilled with joint sealer to minimize surface water intrusion and subsequent shrink/swell.

Some cracking may occur in the foundations. For example, most concrete slabs can develop hairline cracks. This does not mean that the foundation has failed. All cracks should be cleaned up of debris as soon as possible. The cracks should be backfilled with high-strength epoxy glue or similar materials. If a foundation experiences significant separations, movements, cracking, the owner must contact the builder and the engineer to find out the reason(s) for the foundation distress and develop remedial measures to minimize foundation problems.

## FOUNDATION STABILIZATION

### General

Several methods of foundation stabilization are presented here. These recommendations include foundation underpinning, using drilled footings or pressed piling, moisture barriers, moisture stabilization, and chemical stabilization. Some of these methods are being used in the Houston area. A description of each method is summarized in the following sections of this document.

### Foundation Underpinning

Foundation Underpinning, using drilled footings or pressed piling has been used in the Houston area for a number of years. The construction of a drilled footing consists of drilling a shaft, about 12-inches in diameter (or larger) constructed underneath the grade beam. The shaft is generally extended to depths ranging from 8 to 12-feet below existing grade. The bottom of the shaft is then reamed with an underreaming tool. The hole is then backfilled with steel, concrete, and the grade beams are jacked to a level position and shimmied to level the foundation system.

In a case of pressed piling, precast concrete piers are driven into the soils. These pier attain there bearing capacity based on the end bearing and the skin friction. In general, the precast concrete units are about 12-inches in height, six-inches in diameter and jacked into the soil. It is important the precast pier foundations are driven below the zero movement line to resist the uplift loads as a result of underlying expansive soils. Some of these jacked piles may consist of perma-piles, ultra piles, cable lock piles, etc.

The use of drilled footings/pressed piles should be determined by a geotechnical/structural engineer. Each one of these foundation systems have their pluses and minuses. Neither of these foundations can resist upward movement of the slabs. In general, they only limit the downward movement of the slabs. The pressed piles may not resist uplift loads as a result of skin friction of expansive soil if they are not connected with a cable or reinforcement. Therefore, if the units are not properly connected, they will not provide any tensile load transfer. The construction of each method should be monitored by an experienced geotechnical/structural engineer.

Helical piles which consist of Helical auger drilled into the soils provide a good method for underpinning, especially in the areas where sand, silts, shallow water table or caving clays are present. The helical piles are drilled into the soils until the desired resistance to resist the compressive loads are achieved. The augers are then fitted with a bracket and jacked against the grade beams to lift and to level the foundations.

Interior foundations may be required to level the interior of the residence. This can be accomplished by installing interior piers, tunneling under foundations and using pressed piling, or the use of polyurethane materials injected at strategic locations under the slab. The use of tunneling to install interior piers may introduce additional problems, such as inadequate compaction of backfill soils under the slab. However, the author has never encountered such a problem with pressed piling.

Partial underpinning is used in the areas where maximum distress is occurring under a slab. The use of full underpinning which includes placement of piers/pressed piling underneath all foundations is not necessarily a better method of stabilizing foundations. Many foundations are performing satisfactorily with partial underpinning. In the event that foundation underpinning is used, the home owners should put into place a foundation maintenance program to prevent additional foundation distress as a result of changes in subsoil moisture content.

Moisture Stabilization

Moisture Stabilization can be an effective method of stabilizing subsoil shrink swell movements in the ares where expansive soils are present. This method of stabilization is not effective in the areas where sands are present such as north of Harris County in areas such as Kingwood, Fairfield and The Woodlands. This method could be effective in the areas of highly expansive soils such as Tanglewood, Bellaire, West University, River Oaks, South Houston, and Southwest Houston. The method uses a porous pipe that is placed around the perimeter of the foundation and is connected to a water pressure system. A timer turns the water on and off depending on the subsoil moisture conditions, the moisture conditions around the perimeter of the house are monitored by moisture sensors. In general, the purpose of the system is to stabilize the moisture content around the slab to a uniform condition; therefore, minimizing the extremes of shrink and swelling problems. As it was mentioned earlier, the use of this method can result in major problems in the areas where sandy soils are present.

Moisture Barriers

Moisture barriers can be used to isolate subsoil moisture variations in the areas where expansive soils are present. This can be as a result of surface water, groundwater, and tree root systems. In general, a moisture barrier may consist of an impermeable filter fabric, placed just outside the grade beams to depths ranging from five- to seven-feet. The moisture barriers can be horizontal or vertical. A horizontal moisture may consist of a sidewalk attached the exterior grade beams. The waterproofing between the moisture barrier and the exterior grade beams are very important. The connection should be completely sealed so that surface water can not penetrate under the horizontal moisture barrier. In general, it may take several years for the moisture barriers to effectively stabilize the moisture content underneath the floor slabs. A minimum vertical moisture barrier depth of five-feet is recommended.

Chemical Stabilization

This method of foundation stabilization has not been used in the Houston area routinely; however, it has been used for many projects in Dallas and San Antonio areas. The purpose of chemical stabilization is to chemically alter the properties of expansive soils; thus, making it non-expansive. In a chemical stabilization technique, the chemicals which may consist of lime or other chemicals are injected into the soil to a depth of about 7-feet around the perimeter of the structure. The chemical stabilization may (a) chemically alter the soil properties, and (b) provide a moisture barrier around the foundation. In general, this type of stabilization is effective when the chemicals are in intimately mixed with the soil. This can occur in soils that exhibit fissured cracks and secondary structures. This method of stabilization is not effective in the areas where soils do not experience significant cracking.

Regardless of what method of foundation stabilization is used, the homeowner maintenance with respect to proper drainage and landscaping is extremely important for success of any method.

## RECOMMENDED QUALIFICATIONS FOR THE GEOTECHNICAL ENGINEER

We recommend that the geotechnical engineer should have the following qualifications:

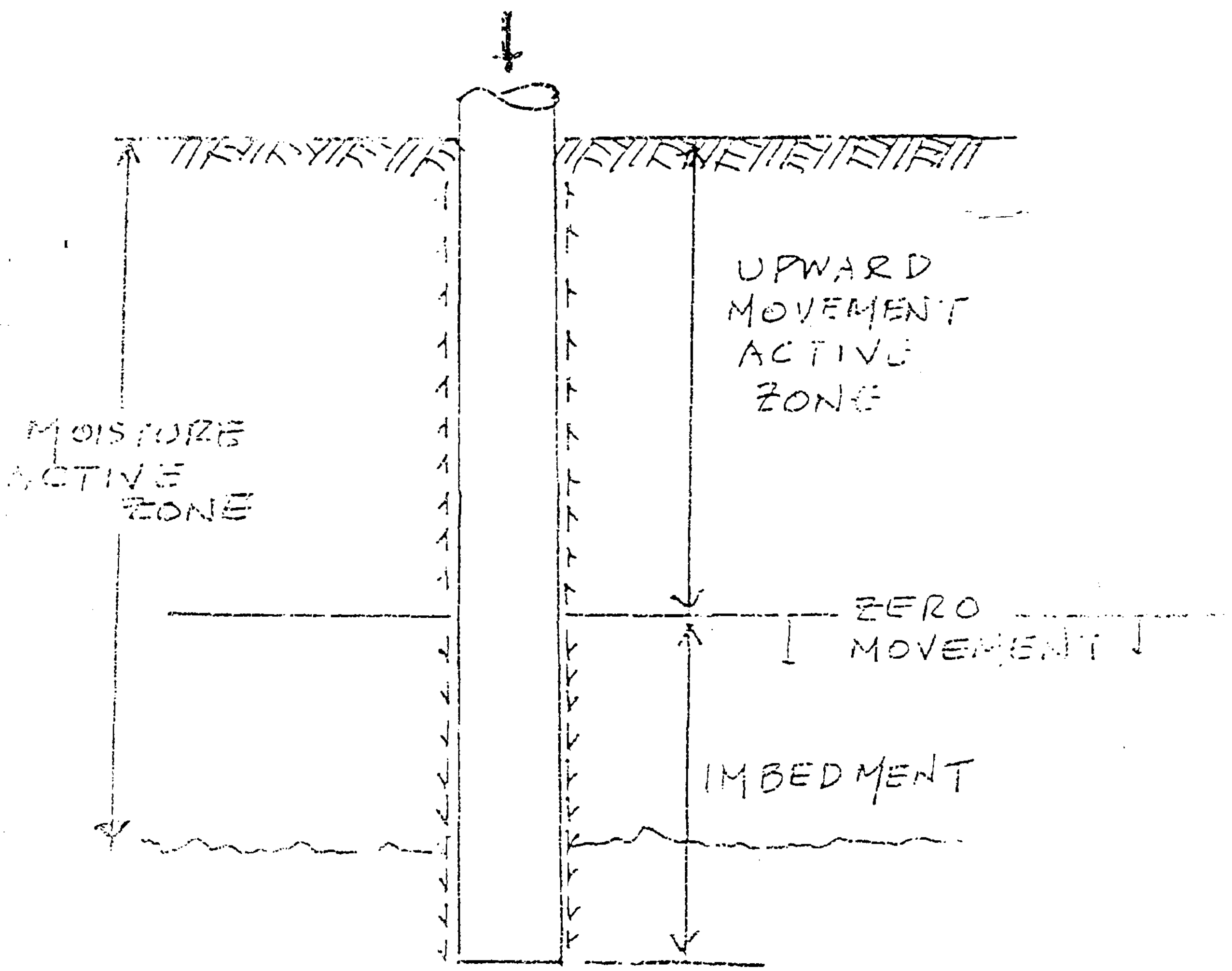
- Engineer must have several years experience in the same geographical area where the work will take place (i.e. proven designs in a given area).
- A Professional Engineer (P.E.) designation with a geotechnical engineering background should be required. A civil engineer with a master's degree or higher is preferred. The civil engineer must have a geotechnical engineering specialty.
- The geotechnical engineering firm must have a A2LA Laboratory certification in geotechnical engineering.

- The firm must have professional liability insurance with errors and omissions.

SLIDE NO.

10

# DESIGN CONDITIONS : UPLIFT




From Dr. Lytton

# FOUNDATION CONDITION SURVEY

## State Subcommittee Report

**Bill Lawson, PE**

# DEVELOPMENT OF DESIGN AND REMEDIAL MEASURES FOR LIGHTLY-LOADED STRUCTURES FOUNDED ON EXPANSIVE SOILS WITH TREES IN MIND

David A. Eastwood, P.E.
Geotech Engineering and Testing, Inc.
800 Victoria Drive
Houston, Texas 77022
713-699-4000

and

Richard W. Peverley, P.E.
Peverley Engineering, Inc.
7207 Regency Square, Ste. 108
Houston, Texas 77036
713-977-0328

## ABSTRACT

A review of the current literature in the United States shows there may be an absence of practical approaches for the design of lightly-loaded structures founded on expansive clay soils where trees are involved. As a result, many such structures which have experienced distress as a result of the lack of the consideration of the effect of trees in the design of these structures. The authors of this paper are a part of a Houston organization called the Foundation Performance Committee. One of the activities of this Committee is the investigation of the adverse effect that the presence and/or the removal of trees can have on lightly-loaded structures. This paper includes the results of this activity, and also includes the results of a literature survey, a review of the failures of some foundations whose cause has been attributed to the presence of and/or the removal of trees, design recommendations, and repairs where trees have been identified as the primary cause.

## INTRODUCTION

A significant number of residential buildings were constructed in the greater Houston are in the 1950 through 1970 time periods. Many of these buildings were founded on expansive soils and on building lots, which were void of vegetation. Trees were then planted after these building were sold and eventually the trees matured and caused foundation failure to occur. Corrective measures generally included the underpinning of the foundation perimeter beam using drilled piers and later pressed piles. When piers were used, this approach had, at best, a limited degree of success. The type of failure mode, whereby trees cause the settlement of the perimeter of residential, and other low-rise buildings, has received a significant amount of publicity in recent times and, as a result, home owners, building remodeling contractors, home builders, etc. have become aware of this problem. From this, an entire industry has grown which provides measures to control this problem.

Some notice of the manner in which trees can produce downward deflections in residential slab-on-ground foundations began to appear in the early 1970's. For example, in 1972, Davis(1) summarized existing papers which indicated that near-by trees could adversely affect foundation performance. Davis and Tucker(2) published a Technical Note which provided data which showed that Post Oak trees located South of Arlington, Texas, incurred vertical movements varying from 1.2 inches to 3.4 inches between the end of the summer months and the end of the winter months. The University of Texas at Arlington conducted an investigation of 69 abandoned residential buildings which were founded on clay soils and reported, among other things, that the extraction of moisture from the soil through the roots of trees, caused moderate to severe deflections in foundations.(3) In 1974, Buckley (4) presented data which showed damage to foundations caused by trees. Kramer and Kozlowski(5) identified the comparatively high transpiration rates of some trees in 1960. In 1987, Peverley and Hanys(6) provided measured deflections in a residential foundation which had been produced by near-by trees.

The manner in which trees can cause downward deflections in residential slab-on-ground foundations has, therefore, been well documented. Simply stated, in order to satisfy their need for soil water, trees can desiccate the soil upon which the outer edge of a foundation rests, resulting in the shrinkage of the soil with the attendant loss of soil support. Foundation distress attributable to such causes have occurred with such regularity in the greater Houston area as to have caused a major alteration to the fundamental foundation design and construction concepts

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with were in effect for years. Not as well understood, however, are the adverse effects that the removal of large trees can have on reconstructed foundations, even where they are resting on drilled piers. Equally misunderstood is the relationship between tree root growth and under-slab sewer leaks.

The purpose of this paper is to explore the adverse effects that trees can have on residential foundation performance based on the experience of others as documented in the literature, based on the personal experience of the authors, and based on an accumulation of information form the Foundation Performance Committee. This paper will be presented in the three following basic parts; foundation edge settlement produced by soil shrinkage, foundation edge heaving caused by soil swelling, and foundation center settlement caused by the interaction between tree roots and under-slab sewer leaks. The mechanics of such conditions along with proposed corrective measures will be discussed. Examples will be presented.

## SOIL MECHANICS AS AFFECTED BY TREES

### THE PHYSIOLOGY OF TREES

Trees have long been considered to be a benefit to mankind. Trees have been written about as many as 4000 years ago.[7] Trees absorb heat as they transporate, provide shade, and reduce solar radiation. They enhance air purification, aid in the control of erosion, and can, to a limited degree, provide some noise reduction benefits. Perhaps their most appreciated benefit is their ability to enhance the beauty of the surrounding landscape. One can appreciate the beauty of old oaks whose branches provide an umbrella for many of the roads in the old South or whose sculpture enhances the skyline of the Pacific Coast.

Trees do have their downside. Their limbs fall injuring property and people. In the Gulf coast, people have been injured or killed by falling trees. Tree roots clog sewers and break sidewalks. Trees can also increase the ozone content of the air, damage electrical power lines, and interfere with UHF reception. Perhaps the highest cost of trees is in their damage to residential foundations. In 1973, Jones and Holtz [8] estimated the annual cost of expansive soils in the US to be 2.2 billion dollars. In 1982, Peverley & Hanys [6] estimated the cost to repair only those residential foundations in the greater Houston, Texas real estate market for a 6-month period of time to be in excess of 28.5 million dollars. If one were to conservatively estimate that only 50% of these foundation failures were caused by trees, the costs would be obviously enormous.

Trees are the largest plants in the world.[9] Trees can generally be classified as needle-leaf or broad leaf (deciduous). The essential parts of a tree are the crown, the trunk, and the roots. The crown contains the leaves, which essentially absorbs sunlight and converts it into food. The roots are the fastest growing part of a tree. They collect water and transport it through the trunk to the leaves in the form of sap. The trunk provides the transporting mechanism between the leaves and the roots and is made up of the heartwood in the center, the cambium layer at the outer edge of the trunk, and the bark, which provides the primary protection. Roots grow only as fast as they are provided energy from the leaves. The tree system consists of the circulation of water from the roots upward through their trunk in the form of sap. When the sap reaches a leaf, the water evaporates into the air. The sap brings mineral salts from the earth to the leaves. The chlorophyll in the leaves acts with sunlight to convert the salts into food through photosynthesis. This food then flows back into the tree system through paths just below the bark. It is this system which makes a tree live.

In engineering terms, we are primarily concerned with the term evapotransporation; i.e., the withdrawal of moisture from the soil and its eventual transporation into the atmosphere. Attempts have been made to quantify this term; however, the results have not always been uniformly accepted in the engineering/ arborist communities, primarily because of the inability to accurately measure the moisture loss/replacements in the soil under most trees. Driscoll[10] presented a ranking of trees in terms of their damage potential. A modified copy of this ranking is contained in Table 1.[11]

In the greater Houston area, we do not have an abundance of Poplar trees; however, there well may be more Oak trees than any others. Also contained in this document is an example of seasonal moisture content variations, which is shown in Figure 1. Of interest is the identification of a zone of permanent moisture deficiency. This concept was further explored by Biddle[12] who measured the soil moisture content in the close proximity of various kinds of trees which were growing in a variety of clay soils, all of which were in England. A combined moisture reduction/moisture deficit curve for a Poplar tree growing

**Table 1. Risk of damage by different varieties of tree**

| Ranking | Species | Maximum height of tree (H): metres | Separation between tree and building for 75% of cases: metres | Minimum recommended separation in shrinkable clay: metres |
|---|---|---|---|---|
| 1 | Oak | 16-23 | 13 | 1H |
| 2 | Poplar | 24 | 15 | 1H |
| 3 | Lime | 16-24 | 8 | 0.5H |
| 4 | Common Ash | 23 | 10 | 0.5H |
| 5 | Plane | 25-30 | 7.5 | 0.5H |
| 6 | Willow | 15 | 11 | 1H |
| 7 | Elm | 20-25 | 12 | 0.5H |
| 8 | Hawthorn | 10 | 7 | 0.5H |
| 9 | Maple/ Sycamore | 17-24 | 9 | 0.5H |
| 10 | Cherry/Plum | 8 | 6 | 1H |
| 11 | Beech | 20 | 9 | 0.5H |
| 12 | Birch | 12-14 | 7 | 0.5H |
| 13 | White Beam/ Rowan | 8-12 | 7 | 1H |
| 14 | Cypress | 18-25 | 3.5 | 0.5H |

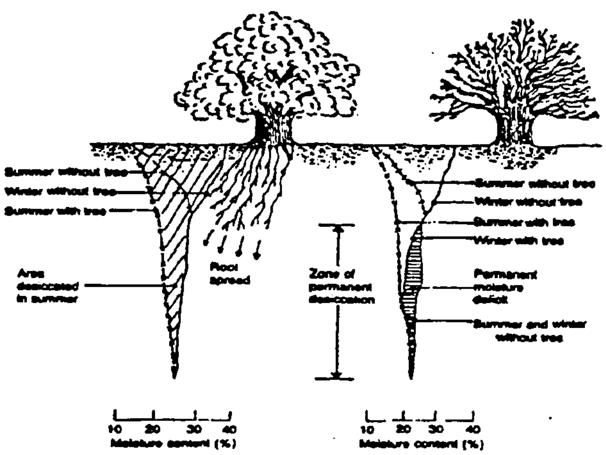

**Figure 1. Seasonal variation in moisture content with and without trees**

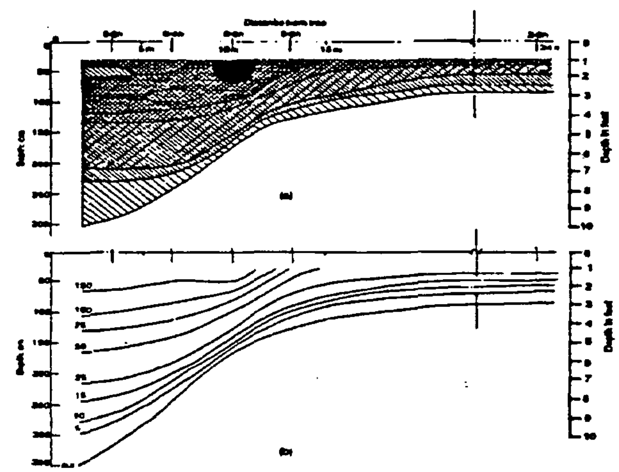

**Figure 2. An example of soil moisture and soil moisture deficit reduction curves for popular trees on a bolder clay soil in London, England**

in a Bolder Clay (PI = 29%) is shown in Figure 2. The moisture deficit curves are calculated by multiplying the change in moisture content by the appropriate layer thickness. In reviewing this curve, it is significant that it does represent the most severe condition for a tree which has been judged to be of a lesser threat than would be an Oak tree growing in a soil whose Plasticity Index is less than some of the major areas in the greater Houston, Texas area. The availability of such data in England has had significant impacts on the construction business. Whereas it was considered to be impractical to plant any tree closer to a foundation than its ultimate height, these data do provide some bases for the planting of certain trees closer to buildings, assuming the data provided for these curves are considered. They also demonstrate the folly of simply removing existing trees in expansive soils for the construction of a new residential structure.

## WHAT CAUSES EXPANSIVE SOILS TO SHRINK OR SWELL

As clay particles are formed, there are usually several points in the particle arrangement where there is an electrical imbalance; the electrical imbalance is increased whenever a "string" of clay particles is broken apart. Thus, the result is that a clay particle typically has a negative net electrical charge on its surface. Since nature likes all things to be balanced, whenever a water molecule drifts close enough to the surface of a clay particle, the negatively charged surface of the clay particle causes the positive end of the water molecule to turn toward the particle. If it is close enough to the particle, the water molecule is attracted to the clay particle surface sufficiently strongly that the water molecule becomes trapped. Also, unattached or "free" positively charged particles, called "cations", tend to acquire a spherical-shaped arrangement of water molecules which have their negative ends directed toward the positively charged cation (and their positive ends directed away from the cation). When the free cation is "captured", water molecules approach a clay particle. The attraction between the negatively charged clay particle surface and the positively charged outside of the cation sphere of water molecules causes the cation to be "captured" by the clay particle, thus increasing the amount of water associated with the clay particle.

Clay particles are very small. A typical kaolinite particle might have a total surface area (top, bottom and edges) of approximately 1 x 10-5 mm$^2$ (1 x 10-10 ft$^2$, or 0.0000000001 ft$^2$). As areas go, this is very small. Smectite particles have a diameter that is 100 to 1,000 times smaller than kaolinite particles and a thickness that is 10 to 400 times thinner than kaolinite[13] and, consequently, typically have a larger surface area per particle. Thus, a single pound of montmorillonite particles would have an incredible total surface area of approximately 800 acres (325 hectares) [13] with which to attract water.

Thus, expansive soils are very small in size and have a large surface area that attracts free water. Because of these characteristics, it is easy to see why it is said that expansive soils are those clays that exhibit an extreme change in volume.

Soil suction is a measure of free energy of the pore-water or tension stress exerted on the pore-water by soil matrix. Soil suction is, in practical terms, a measure of the affinity of the soils to retain water and can provide information on soil parameters that are influenced by soil water; e.g., volume change, deformation, and strength characteristics of soil. The soil suction is measured using the filter paper method in accordance with ASTM D-5298.

The soil suction is divided into two components; Matrix suction and osmotic suction. The matrix suction is the negative pressure (expressed as a positive value) relative to ambient atmospheric pressure on soil water, to which solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with soil water; pressure equivalent to permeable wall with the soil water; and pressure equivalent to that measured by test methods D2325 and D3152.

The osmotic suction is the negative pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semi-permeable membrane with a pool containing a solution identical in composition with soil water; decrease in relative humidity due to the pressure of dissolved salts in pore-water.

### FOUNDATIONS AND RISKS

Many lightly loaded foundations are designed and constructed on the basis of economics, risks, soil type, foundation shape and structural loading. Many times, due to economic considerations, higher risks are accepted in foundation design. Most of the time, the foundation types are selected by the owner/builder, etc. It should be noted that some levels of risk are associated with all types of foundations and there is no such thing as a zero risk foundation. All of these foundations must be stiffened in the areas where expansive soils are present and trees have been removed prior to construction. The foundation types typically used in the area with increasing levels of risk and decreasing levels of cost are discussed in Table II.

The above recommendations, with respect to the best foundation types and risks, are very general. The best type of foundation may vary as a function of structural loading and soil types. For example, in some cases, a floating slab foundation may perform better than a drilled footing type foundation.

## FOUNDATION PROBLEMS CAUSED BY TREES

### FOUNDATION SETTLEMENT PRODUCED BY SOIL SHRINKAGE

Several authors as far back as 1960 have documented this type of distress.[9,14] Buckley[3] proposed that trees be placed no closer to a residential foundation than its ultimate height. The basis for this recommendation is contained in Figure 3. It was not, however, widely recognized by the designers and constructors of the millions of

**Table II.**

| | |
|---|---|
| Structural Slab with Piers | This type of foundation (which also includes a pier and beam foundation with a crawl space) is considered to be a minimum risk foundation. A minimum crawl space of six-inches or larger is required. Using this foundation, the floor slabs are not in contact with the subgrade soils. This type of foundation is particularly suited for the area where expansive soils are present and where trees have been removed prior to construction. The drilled footings must be placed below the potential active zone to minimize potential drilled footing upheaval due to expansive clays. In the areas where non-expansive soils are present, spread footings can be used instead of drilled footings. |
| Slab-On-Fill-Foundation Supported on Piers | This foundation system is also suited for the area where expansive soils are present. This system has some risks with respect to foundation distress and movements, where expansive soils are present. However, if positive drainage and vegetation control are provided, this type of foundation should perform satisfactorily. The fill thickness is evaluated such that once it is combined environmental conditions (positive drainage, vegetation control) the potential vertical rise will be minimum. The structural loads can also be supported on spread footings, if expansive soils are not present. |
| Floating (Stiffened) Slab Supported on Piers. The Slab can either be Conventionally-Reinforced or Post-Tensioned. | The risk on this type of foundation system can be reduced sizably if it is built and maintained with positive drainage and vegetation control. Due to presence of piers, the slab can move up if expansive soils are present, but not down. In this case, the steel from the drilled piers should not be doweled into the grade beams. The structural loads can also be supported on spread footings if expansive soils are not present. |
| Floating Slab Foundation (Conventially-Reinforced or Post-Tensioned Slab) | The risk on this type of foundation can be reduced significantly if it is built and maintained with positive drainage and vegetation control. No piers are used in this type of foundation. Many of the lightly-loaded structures in the state of Texas are built on this type of foundation and are performing satisfactorily. In the area where trees have been removed prior to construction and where expansive clays exist, these foundations must be significantly stiffened to minimize the potential differential movements as a result of subsoil heave due to tree removal. |

homes built on concrete slab-on-ground foundations in the 1950 time period. In Houston, Texas, as an example, the post World War II building boom included upwards of 100,000 homes constructed in the Southwest part of the City, where the soils were typically very expansive. In most cases, these homes were initially constructed in subdivisions outside of the City limits, but were later annexed by the City. Thus, no building codes were applied. Since this real estate was largely farm land which was barren of trees, one of the things that were done by individual homeowners (and even some subdivision developers) was to plant trees in the yard close to the foundation. Trees such as Oaks, China Berry, and Pecan were popular because they were hardy. When the trees entered into their period of major growth, their water demands steadily increased and foundation problems began.

Studies[15] have shown that when a slab is placed on ground, evaporation of soil moisture is retarded. If the

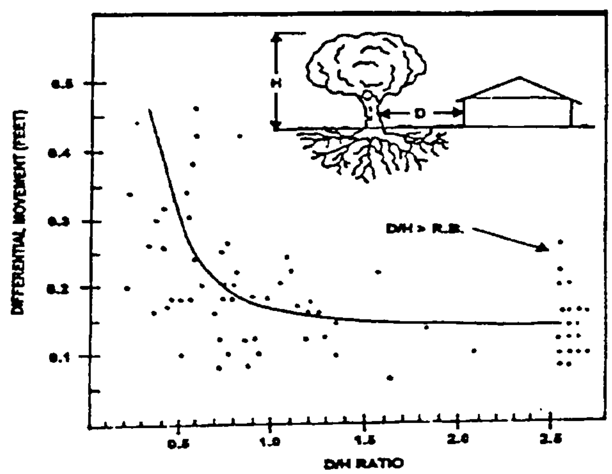


Figure 3. A curve of foundation vertical movement versus tree height

soil is desiccated at the time of construction, moisture will move toward the center of the slab and is going to be higher than at the edge. Trees use soil moisture for growth. Therefore, during wet periods, sufficient supply of mois-

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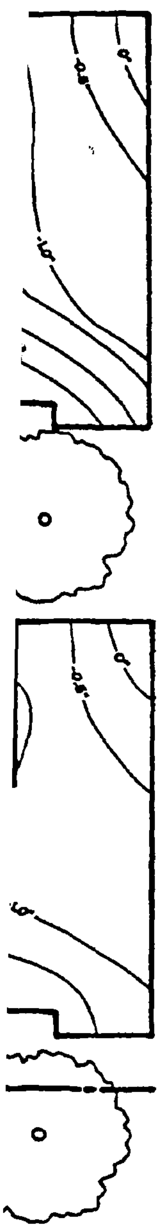

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Figure 4 shows an example of a home located in the Southwest part of Houston, Texas, whose slab-on-ground foundation was underpinned using drilled piers. During the 1988 to 1990 drought, the foundation incurred additional deflections. At our suggestion, a root barrier, combined with an automatically actuated soaker system was applied, and the foundation not only stabilized, but some rebound occurred. The example contained in Figure 4 is not, unfortunately, an isolated example. Better results have been obtained in the recent past using pressed segmented piles and helical piers.[16] Vertical moisture barriers have enjoyed a very limited use.

## FOUNDATION EDGE HEAVING CAUSED BY SOIL SWELLING

To compensate for the settlement of the outer edge of a foundation due to the extraction of moisture from the soil through the roots of nearby trees, an accepted preventative measure was to place the foundation on top of drilled piers. Most recently, however, a condition has occurred where the piers became a detriment. In the inter part of the City of Houston, Texas, there are subdivisions with comparatively small building lots, which often contained small, older homes, became very desirable because of their location. In many cases, the lots contained large, prolific trees, which were removed to make way for the construction of larger homes. In many such cases, nothing was done to compensate for the inevitable swelling of the soils which would occur when the tree, which had desiccated the soil in its near vicinity, was gone and soil suction forces moved soil water into the desiccated areas. It does not take much imagination to invision the mood of a homeowner who, in many cases, paid extra moneys to have a sturdy foundation constructed only to have it begin to move soon after the owner moved in. An example of such a condition is shown in Figure 5. The foundation, in this example, was founded on 10-foot deep drilled piers, which had 42-inch diameter bells that were inspected during construction. The pier shafts were tied to the concrete perimeter beam. The soils had a plasticity index in the 60% range. A Pecan tree was remove during the construction process, or shortly thereafter. Signs of foundation induced damage became manifested within the first year of construction and have, in the interim, worsened steadily. A literature search failed to reveal any documented discussion of this phenomenon, not only in the state of Texas, but in the United States, as well. Such discussions were, however, found in literature from outside of the United States. A listing of such sources is

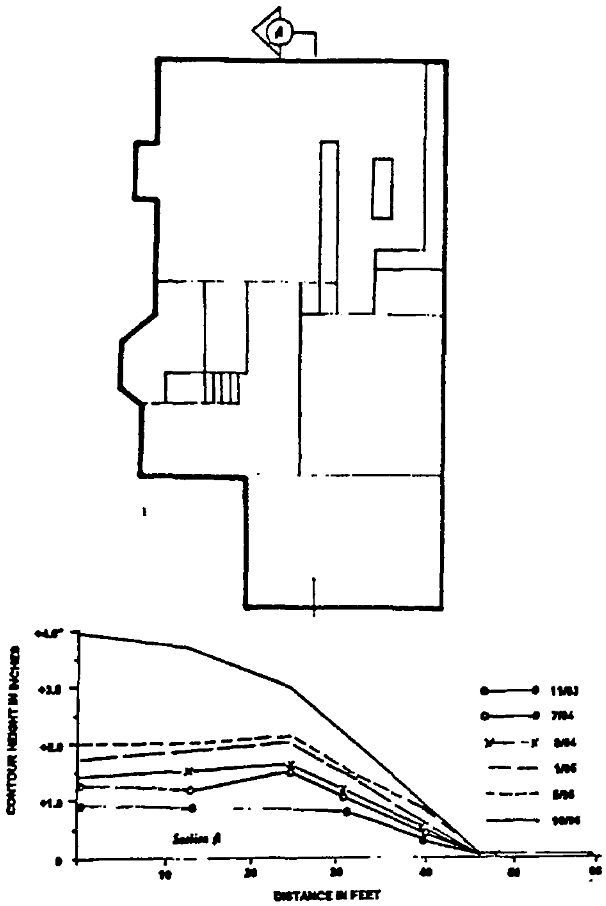


Figure 5. An example of a home which had been adversely effected by the removal of a Pecan tree

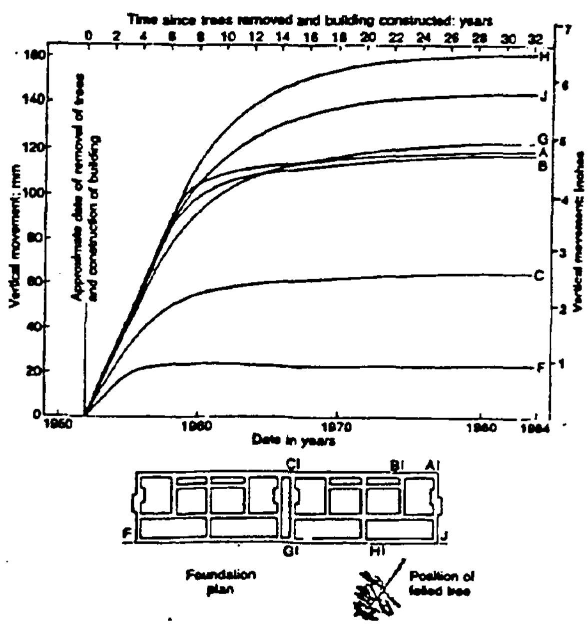


Figure 6. A copy of a vertical movement that occurred as a result of the removal of a Poplar tree in London, England

contained in the Bibliography Section of this paper. This significance of the tree removal situation is perhaps best illustrated in Figure 6[17]. In this case, the removal of a Poplar tree caused over 6 inches of heaving which was monitored over a 30-year period of time.

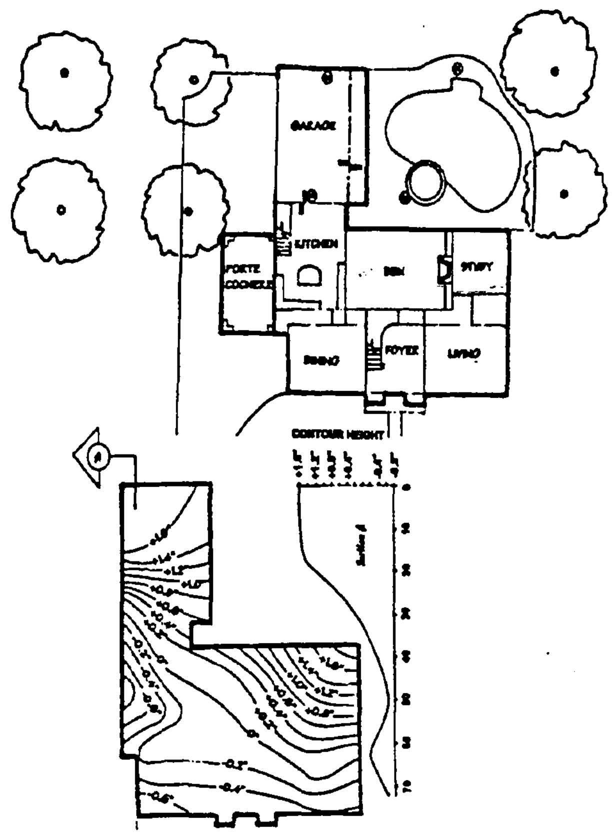


Figure 7. An example of a home in Houston, Texas, which had been constructed on a roadway that was lined with Post Oak trees

Figure 7 shows how a residential building was placed on the edge of a roadway, which, before the time of construction, had Post Oak trees on either side. The removal of the trees caused significant heaving of the foundation at the rear of this, and several other buildings, which were constructed along this roadway. We have been able to measure heaving in a limited number of cases and the results are contained in Figure 8. In comparing our data with that shown in Figure 9, we can see that the slope of our data is steeper, even though one of the curves seemed to level out after 7 years.

As mentioned earlier, tree roots tend to desiccate the soils. In the event that the tree has been removed prior to building construction, during the useful life of the structure, or if a tree dies, subsoil swelling can occur in the expansive

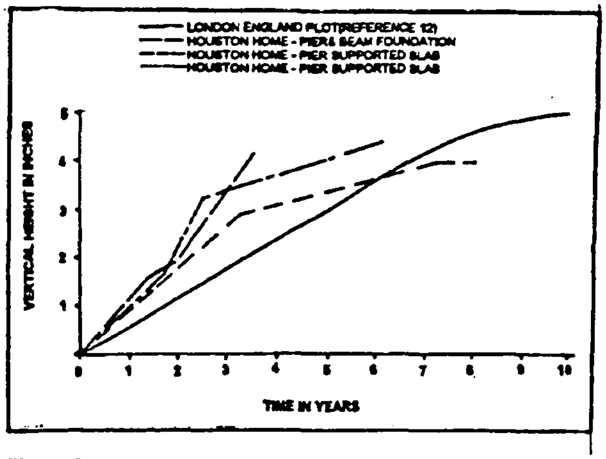


Figure 8.

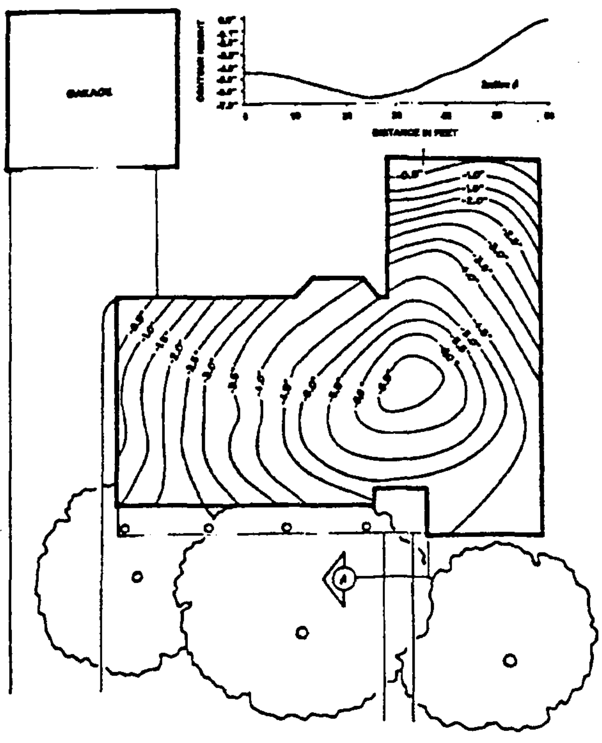

Figure 9.

soil areas for several years. Studies have shown that this process can take several years in the area where highly expansive clays are present. In this case, the foundation for the structure should be designed for the anticipated maximum heave. Furthermore, the drilled footings, if used, must be placed below the zone of influence of tree roots. This depth should be evaluated as follows:

a) The pier should be placed below the depth of constant suction or the zero movement line.

b) The pier depth should be such that it could resist the uplift loads due to expansive soils that extend along the shaft perimeter.

c) More extensive soil tests are required. Soil borings near a tree must be, as a minimum, 25 to 30 feet deep. The depth to which tree root fibers exist must be determined since they are a basis of identifying the depth of constant soil suction. Potential Vertical Movement values should also be calculated.

In the event that a floating slab foundation is used, we recommend the slab be stiffened to resist the subsoil movements due to the presence of trees. In addition, the area within the tree root zone may have to be chemically stabilized to reduce the potential movements. Alternatively, the site should be left alone for several years so that the moisture regime in the desiccated areas of the soils (where tree roots used to be) become equalized/stabilized to the surrounding subsoil moisture conditions. The length of time required for subsoils to regain their moisture is dependent on the tree species, soil type and the amount of rainfall. For most trees, one wet season may be enough for the subsoils to regain their moisture; however, removal of trees such as Live Oak, Poplar, etc. may result in moisture deficits in the soil profile that may require several years to stabilize.

Remedial measures to correct the adverse effects of this type of soil heaving are somewhat limited. One method is to raise the entire foundation out of the potential vertical rise of the soil using underpinning techniques. Soil testing will generally show the PVM (Potential Vertical Movement) values in the soil where the trees were removed. It may then become necessary to raise the foundation out of this zone. An alternative is to use a vertical moisture barrier. A combination of partial underpinning and the use of moisture barriers may also have to be used to stabilize the foundation system.

## FOUNDATION CENTER SETTLEMENT

There has been an ever-increasing problem with regard to the interaction between trees and foundation performance; i.e., foundation performance induced by under-slab sewer leaks. Many of the homes constructed in the 1950 time period had cast iron, under-slab sewer pipes buried in clay soil. Over the past 40 (+) years the effect of this unfortunate marriage has produced a proliferation of under-slab sewer leaks. Since most (if not all) of the home insurance policies allow a homeowner to collect on damages caused by such leaks, there has been an attendant number of such claims filed.

Typically, the insurance carrier hires a plumbing testing company and an Engineer to determine if the leak has caused any foundation related damage. It is likewise typical that this same Engineer will observe that the foundation has deflected downward in its center section, which is the opposite of what one might anticipate if water were to be induced into expansive soils. Other contradictions may be observed which included the following:

- o The timing of the damage appeared coincidental to the occurrence of the sewer leak.
- o There were plumbing leaks; yet, where soil tests were conducted, the soils were comparatively dry.
- o There was always a reasonable degree of correlation between the presence of the sewer leaks and the points of deflection.
- o In a majority of cases, the soils were expansive, the foundation was constructed on drilled piers or was underpinned using drilled piers subsequent to the time of original construction, and there were trees growing near the foundation which were almost always mature. It is a known fact that the water demands of mature trees tend to stabilize. Could these trees then suddenly become the source of additional foundation deflections?
- o In the 20(+) cases which we examined, the foregoing conditions existed and the foundation settled in the center instead of heaving, as was anticipated. An example is shown in Figure 9. In this case, the sewer system could not be tested since it would hold no water.

We are of the opinion that the introduction of the sewer water spurred the growth of the tree roots to grow towards the source.[18, 19] The tree roots then extracted not only the moisture provided from any sewer leaks which occurred, but also any moisture which was in the soil before the leak occurred. The presence of an under-slab sewer leak then resulted in a net soil moisture loss where large trees were growing adjacent to the foundation with the ultimate result that the foundation subsided instead of heaving, as one might anticipate.

This opinion does, of course, involve a number of assumptions for which no proof exists. In fact, there is little of no real proof that any sewer leak did, or did not, cause foundation deflections to occur. More testing and study is required.

## CONCLUSIONS AND RECOMMENDATIONS

The design and manufacture of most of the material things we use in our lives is based, to at least some degree, on some type of research. Automobiles are designed and extensively tested before they reach the market, manufacturers of appliances subject them to extensive testing before new models are put up for sale, new food products must be given extensive testing, etc. It is then somewhat ignominious that the design and construction of the fundamental part of what is perhaps the most expensive investment for most of us is based on little, if any, current research; at least in the United States. Instead, we tend to learn in the most fundamental, and in the crudest, of ways - by trial and error. The cost of this process is born by the builders, homeowners, and engineers much to the delight of many attorneys.

We have, in this paper, pointed out some of the problems that can be caused by our failure to learn to live with trees in an urban environment. Although much of this discussion was based on Houston, Texas, experience, this information certainly applies to much of Texas and to other parts of the country, as well. All of us who are involved in the design and construction of residential and other low-rise buildings need to be cognizant of these problems and to conduct ourselves accordingly. This may require additional pre-construction testing and may necessitate the need for more expensive designs. Some may say that our clients may not be willing to pay the price for such extra work. So long as there are engineers who are willing to do cheap work, the problems we discussed herein will recur and we will be left to ponder why some people are more willing to pay their attorneys more than they are their engineers and or builders.

We have pointed out the need for research. To the best of our knowledge, the last 2 large research studies conducted on residential foundation issues were the BRAB in the 1960's and the University of Texas at Arlington studies in the 1970's. We do know of some smaller studies, which have been conducted at some Universities, but we believe that larger studies are needed not only on the issues presented herein but on sister issues, as well. Some of the study areas are listed below:

- o A relationship needs to be developed that would address the tree type (species), distance from the foundation, and height of the tree.
- o Studies similar to those conducted by Biddle should be done using trees more typically found in the United States (Oaks, Pecans, China Berry, etc),in clay soils and varying weather patterns that are typical of this country.
- o How to better design floating slabs that would resist the effect of trees.
- o Develop a simple mathematical module that would relate sewer leaks, tree moisture removal, and subsoil movements.

Perhaps the information contained herein will help in the search for research dollars.

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# FOUNDATION DESIGN ISSUES

## State Subcommittee Report

Kirby Meyer

# POLICY ADVISORY

## 09-98-A
## Regarding Design, Evaluation and Repair of Residential Foundations

## Texas Board of Professional Engineers

### I. Background & Purpose

Under the exemptions of Section 20(d) of the Texas Engineering Practice Act, any person who designs, constructs or repairs engineering features for a Texas residence does not need to be licensed as a professional engineer to legally perform that task. However, licensed professional engineers are actually performing a large number of the residential foundation designs, evaluations and repairs performed in Texas each year. According to data collected by the Real Estate Center at Texas A&M University, approximately 76,000 single-family residential building permits were issued on an annual basis since 1995, representing a significant impact on Texas business.

The Board receives a disproportionately high number of complaints against license holders performing the design or evaluation of residential foundations. Since these complaints frequently appear to be a result of poor communications or procedures, the Board established the Residential Foundation Committee (RFC) to pinpoint some of the most common problems and offer a summary of concerns and/or recommendations for the Board's consideration. The RFC and a volunteer support team met in the fall of 1997 and spring of 1998, resulting in the issuance of two reports to the Board's General Issues Committee for staff use in drafting this policy statement. Although the RFC's reports are not a part of this policy, they provide an interesting and quite valuable commentary on various aspects of engineering related to residential foundations. Single copies of the RFC's reports are available by request or may be copied from the Board's home page at **http://www.main.org/peboard**

The purpose of this policy statement is twofold:

A. Provide recommendations to various non-engineering entities on how to minimize the probability that residential foundation problems, currently encountered by homeowners, will occur.
B. Provide practicing licensed professional engineers with guidance in the preparation of designs and evaluations of residential foundations to minimize the probability that problems, currently encountered by homeowners, will occur.

**While the Board may use this policy statement as a tool to evaluate specific complaints, this statement is not intended to replace professional engineering judgment.** This statement is intended to emphasize the professional judgment requirements of Board Rules 22 TAC 131.151-155, not to replace or modify them in any way. **Under no circumstances should a professional engineer use this statement as a "checklist" of activities needed to adequately perform an engineering assignment related to residential foundations.** In its evaluations of complaints, the Board has consistently been most concerned that the intent of the Board rules of conduct and ethics are followed and that the public and client interests are well served. This statement is designed to underscore that concern.

## II. Recommendations

While proper professional engineering practice on individual projects is integral to the success of the project, public policy alterations should be evaluated by the local government entities for probable positive impacts on the property interests of tax-paying homeowners.

The Board makes the following recommendations for consideration by the appropriate entities:

A. Where not already required by existing code, building code enforcement entities such as cities or special districts should require that a licensed professional engineer prepare the designs and directly supervise the construction of residential foundations in situations where soil conditions warrant the involvement of a professional engineer. The public entity should be concerned that warranting conditions may exist:

1. Where the weighted BRAB* equivalent plasticity index of the soil exceeds 20; or
2. Where the site settlement potential exceeds approximately one inch under expected loads; or
3. Where the structure will be supported by fill material; or
4. Where known geological hazards exist.

*Building Research Advisory Board Report #33

B. Warranting conditions should be established in one of two ways. First, licensed professional engineers can establish warranting conditions on a site-specific basis. Second, in areas where general soil conditions are sufficiently well known, licensed professional engineers familiar with local conditions can be requested to aid public entities in the establishment of geographic boundaries where warranting conditions exist.

C. Purchasers of forensic foundation evaluations from licensed professional engineers should base their purchase request on one of three levels of evaluation described in section IV of this statement and understand the scope and limitations associated with that level. The requested level of evaluation to be purchased for the foundation should match the level of analysis of any other evaluations to which it may be compared if a direct comparison is desired. If a particular purpose is intended for the evaluation (such as the development of a repair plan or a forensic report), the engineer must establish the minimum level of evaluation required to adequately accomplish that purpose.

## III. Practice Guidance for Licensed Engineers: Design and Repair

Professional engineers designing residential foundations or designing repairs for residential foundations will meet the requirements of all of the applicable Board rules of professional conduct and ethics in their practice. Special emphasis is placed upon:

A. Board Rule 22 TAC 131.151(a) - Engineers have an obligation to protect the property interests of the future homeowner, the builder, the lender and all other parties involved. Inherent in this rule is the notion that an engineer is to provide an optimized, cost-effective design.

B. Board Rule 22 TAC 131.151(b) - Engineers must perform their design in a manner which can be favorably measured against generally accepted standards or procedures. A design or repair plan should include all information needed to delineate its scope, intended use, limitations, client contract requirements or other factors that can impact its proper implementation. If called upon to evaluate a complaint under this rule, the Board will assess engineers' work against design procedures such as the

Post-Tensioning Institute's design guideline, the Building Research Advisory Board Report #33, or other similar procedures. Engineers' work will be expected to address significant design issues that may include (but may not be limited to):

1. collection of sufficient geotechnical data;
2. selection of reasonable sample locations and testing activities for geotechnical data;
3. completion of a site characterization activity, denoting key feature such as the presence of water or fill material;
4. inclusion of all needed specification documentation for adequate construction of the foundation;
5. inclusion of a plan for supervising or inspecting the foundation construction; and
6. documentation of all engineering functions in a suitable manner for clients, code officials, etc.

C. Board Rule 22 TAC 131.166 - Engineers must only seal work that they have personally performed or has been performed under their direct supervision. Direct supervision as defined under 22 TAC 131.18 requires the engineer to provide some acceptable combination of exertion of control over the work, regular personal presence, reasonable geographic proximity to the work being performed, and an acceptable employment relationship with the person(s) being supervised. If called upon to evaluate a complaint under this rule, the Board will evaluate all work attributed to an engineer (including post-tension designs, pier layouts, repair details, etc.) for conformity to these direct supervision requirements.

D. Engineers in responsible charge of this type of work must be competent to perform it adequately. Competence is established through education, training or experience in appropriate areas of endeavor; these areas might include residential foundation design, structural engineering, soils and geotechnical engineering, materials engineering and general civil engineering.

**IV. Practice Guidance for Licensed Engineers: Evaluations of Existing Foundations**

A. When evaluating an existing residential foundation, engineers will be expected to report their findings in a manner that clearly identifies:

1. the purpose of the evaluation;
2. the level of evaluation at which the work was performed; and
3. limitations regarding the conclusions that are drawn given the level of evaluation used.

All evaluations, regardless of the level at which they are performed must be of professional quality as evidenced by sufficient and appropriate data, careful analyses, and disciplined and unbiased judgment when drawing conclusions and stating opinions. In accordance with Board Rule 22 TAC 131.152(b) engineers must communicate using clear and concise language that can be readily understood by their client or other expected audiences.

B. In certain cases, the level of evaluation is established by the client. However, in most cases involving the potential for repair of a condition, the engineer will recommend an appropriate level of evaluation for the situation. Under Board Rule 22 TAC 131.155(a), the engineer is expected to recommend and perform the lowest level of evaluation needed for adequate analysis of the situation. For the purpose of aiding the client in determining the type of evaluation performed (or desired), the Board recommends the use of the following three levels of evaluation designations:

1. Level A - This level of evaluation will be clearly identified as a report of first impression conclusions and/or recommendations and will not imply any higher level of evaluation has been performed. Level A

evaluations will typically:

a. define the scope, expectations, exclusions, and other available options;
b. interview the home owner and/or client if possible;
c. document visual observations personally made by the engineer during a physical walk-through;
d. describe the analysis process used to arrive at any performance conclusion; and
e. provide a report containing one or more of the following: observations, opinions, performance conclusions, and recommendations based on the engineer's first impressions of the condition of the foundation.

2. Level B - This level builds upon the elements found in a Level A evaluation. In addition to the items included in Level A, a Level B evaluation will typically:

a. request and review available documents such as geotechnical reports, construction drawings, field reports, prior additions to the foundation and frame structure, etc.;
b. determine relative foundation elevations to assess levelness at the time of evaluation and to establish a datum;
c. if appropriate, perform non-invasive plumbing tests, recognizing that additional invasive testing is also available;
d. document the analysis process, data and observations;
e. provide conclusions and/or recommendations; and
f. document the process with references to pertinent data, research, literature and the engineer's relevant experience.

3. Level C - This level builds upon the elements found in the Level B evaluation. In addition to the items included in Levels A and B, a Level C evaluation will typically:

a. conduct non-invasive and invasive plumbing tests as required by the engineer;
b. conduct site specific geotechnical investigations as required by the engineer;
c. conduct materials tests as required by the engineer to reach a conclusion;
d. obtain other data and perform analyses as required by the engineer;
e. document the analysis processes, data and observations; and
f. provide conclusions and/or recommendations.

C. Engineers performing evaluations of residential foundations should be especially aware of their obligations under Board Rules 22 TAC 131.153(c), 22 TAC 131.151(b), and 22 TAC 131.152(b) as they report their findings. They should substantiate all assumptions, conclusions, and recommendations using appropriate references. Terms such as "failure", "distress", "damage", etc. must be clearly defined. When an evaluation is to be used in comparison with another report, the engineers should make every effort to provide a correlation to the definition used in the previous report in addition to any other definitions used in their own report. Engineers must draw any needed distinctions between "failures" discussed from a structural aspect and "failures" discussed from a performance aspect.

D. As previously noted in section III (D), engineers in responsible charge of this type of work must be competent to perform it adequately. Competence is established through education, training or experience in appropriate areas of endeavor; these areas might include specific residential foundation design, structural engineering, soils and geotechnical engineering, materials engineering and general civil engineering.

## V. Related Advisories & Updates

There are no related advisories at this time. Updates may be made periodically by the board. Date of this advisory: **09/11/98.**

**Questions regarding this advisory may be sent to:**

*Hali Ummel, Public Information Coordinator*
*Texas Board of Professional Engineers*
*P.O. Drawer 18329*
*Austin, Texas 78760-8329*
*(512) 440-7723*

**Email: peboard@mail.capnet.state.tx.us**

**home page: http://www.main.org/peboard**

**last updated 10/06/98**

# FOUNDATION DESIGN ISSUES

## A State of the Art Review of Tract Home Foundations

Lowell Brumley, PE

# CURRENT PRACTICES FOR DESIGN OF RESIDENTIAL TRACT HOME FOUNDATIONS IN HOUSTON

By Lowell Brumley, P.E.

# CURRENT PRACTICES FOR DESIGN OF RESIDENTIAL TRACT HOME FOUNDATIONS IN HOUSTON

## INTRODUCTION

A tract home is typically defined as a single family residence which is constructed from a plan out of a series of floor plans which is used repetitively to construct a substantial number of homes in a given tract of land or in a subdivision. The purpose of this is to allow the builder to reduce costs through repetitive construction procedures and the purchasing of materials and services in bulk.

As we all realize, today, many of our home builders in Houston, which build "tract" houses, build houses ranging from 1,000 to 1,700 sq. ft. "starter homes" to homes in the 4,000 to 6,000 sq.ft. range. Most builders also have a variety of floor plans and elevations which they will mix into a subdivision to give a more "custom" look to the neighborhood. (TSG has clients which will have foundation designs for as many as 20 floor plans with three to four elevations each, for a 60 lot subdivision.) This leads us to the realization that "tract home" construction has evolved and matured considerably, with many of the changes coming in the last 10 to 15 years. As construction requirements have changed, a need has been brought about for new design criteria and procedures. In foundations, the biggest change which has occurred has come with the use of post tension slab-on-ground foundations.

Today, approximately 85% to 90% of the homes in Houston which are built by repetitive home or "tract" builders are built with post-tensioned slabs-on-grade. The predominance of this type of foundation has occurred over approximately the last 15 to 20 years. The design procedures for post-tension foundations have been developed and evolved through a number of studies, as well as the interacting of various parties involved, the design engineer, the soils engineer, the builders, the post-tension suppliers and to some degree the home buyers.

The first widely accepted analytical design procedure used for slab-on-ground design was the B.R.A.B. Report #33, which was issued in 1968 and contained procedures for developing soil support criteria and design stresses on the foundation. The foundations designed, using B.R.A.B., were normally reinforced with conventional mild steel reinforcement. In 1975 a procedure was developed for designing slab-on-grade foundations reinforced with post-tension tendons. This procedure was developed and prepared by an Ad-Hoc Committee of the P.C.I. Post Tensioning Committee. It used many of the same assumptions and derivations used in the B.R.A.B. Report.

Then in 1980 the Post Tensioning Institute published a document titled "Design and Construction of Post-Tensioned Slabs-on-Ground" which outlined procedures and recommendations for the design and construction of post-tension slabs-on-grade. The second edition, which made some minor changes, was released in 1996. This is the design standard currently in use today for the design of post tension residential S.O.G., and it is accepted by the majority of engineers familiar with residential design and construction. The PTI method is currently under reevaluation to see what changes and improvements can be made. However, this procedure is currently outlined in the 1997 Uniform Building Code and as such has become the official standard for the design of slab-on-grade foundations.

TRENDS IN DESIGN:

The basic design of post-tension slab-on-grade foundations has become more conservative in the last 10 to 15 years for several reasons:

- A. The average home today is typically larger and/or more expensive than the average home of 20 years ago. It is generally more complicated and contains more upgrades.
- B. Homebuyers today expect more out of their homes.
- C. Tougher code requirements have been implemented by local municipalities.
- D. The soil data and recommendations received from the soils engineers give more conservative design criteria.
- E. Our field experience has increased significantly with post tension S.O.G.
- F. Mitigation of Litigation.

Each of these items has had an affect on the home building industry and foundation design. Typically the designs you see today have more concrete, deeper beams, more post-tension tendons and an increased amount of mild steel reinforcement.

DESIGN CONCEPT:

Conceptually a slab-on-grade foundation acts as a buffer between the active soils beneath the home and the structure above. However it is allowed to move and float on top of the bearing soils. The idea is for the foundation to have sufficient strength and rigidity to control, dissipate and distribute the actual loads. In this way, it should control movements in the upper structure such that a home does not have a significant number of reoccurring problems such as sticking doors, sheetrock cracks, brick cracks, etc. and it should not move in such a manner that the structure itself becomes unsafe.

REQUIREMENTS:

For a proper design to be successful several items must be considered:

A. Good geotechnical information is important. A proper soil study and report, which include PTI design parameters, allowable soil bearing values, and plasticity indices, is recommended.
B. Proper lot preparation and building pad placement are necessary.
C. Proper foundation makeup and reinforcement placement must be done.
D. Good quality concrete and proper placement of the concrete are performance related items, which must be controlled.
E. Correct curing procedures are important.
F. Proper grading and drainage around the home are a must!!!
G. Proper homeowner maintenance is also required.

It has been our experience that good construction procedures and proper drainage have the largest overall effect on long term performance of residential foundations.

AREAS FOR IMPROVEMENT

Areas we see which consistently can use improvement are lot and pad preparation, curing procedures and homeowner maintenance.

A. If a lot or pad is improperly placed, initial deflection and settlement can occur which can crack the foundation system, which is unreinforced for a minimum of 4 days after placement. Tendon stressing cannot be done until the concrete strength has increased sufficiently to accept the stressing forces. Once a crack has formed in the foundation, the slab will "in effect" hinge at that location and it is considerably less rigid than at the adjacent areas of the remaining foundation system. If the foundation is subjected to some amount of movement, problems in the upper structure can be concentrated in the vicinity of the crack.
B. Curing and shrinkage cracks can occur which can give the same condition as a settlement crack. During the summer months, curing cracks become more prevalent.
C. Once a foundation is placed, the home constructed, and the buyer has moved in, improper homeowner maintenance can cause a properly performing foundation to be stressed beyond the design values and this can cause the foundation to move abnormally. Therefore, homeowners must be informed as to their responsibilities in the care of their home and its foundation.

FINAL COMMENTS

Even though "tract" homes have become larger and more complicated to build, their time from start to completion has remained unchanged for the last several years (90 to 120 days is what most builders project to complete a new home). This has required builders in Houston to become more efficient in coordinating and organizing their material suppliers and subcontractors, however it has also caused them to allow less time for such things as quality inspections, inclement weather delays, and buyer changes, which affect the foundations. At the same time the design engineer has been asked to design the most economical foundation possible. This is typically accomplished by assuming that the weather and soil conditions are ideal, and the foundation makeup and concrete placement will be perfect. What this leads to are conditions which at some point in time can cause a problem to arise in some percentage of the homes constructed by the builder.

Consistent communication between the builder, the design engineer and the soils engineer, from the beginning of the project (lot acquisition) to the final construction of all homes in the tract or subdivision is a must. Without this type of coordinated cooperation, conditions can easily arise which will be detrimental to the performance of a home's foundation and upper structure.

# CURRENT DESIGN PROCEDURES RECOMMENDED BY THE POST TENSION INSTITUTE (REFORMATTED AND PRESENTED IN THE 1997 UNIFORM BUILDING CODE)

**References:** **Design and Construction of Post-Tensioned Slabs-On-Ground - First Addition, Post Tensioning Institute Copyright 1980, Second Edition 1996. Reprinted with the permission of Post-Tensioning Institute.**

# 4.0 DESIGN COMMENTARY

## 4.1 General

The design method developed herein for slabs on expansive soils is based upon a working stress, or serviceability method. Moments, shears, and differential deflections under the action of applied service loads (including soil loading resulting from changes in climatic moisture) are predicted using equations developed from empirical data and a computer study of a plate on an elastic foundation. Concrete stresses caused by those moments and shears are limited to specific allowable values. Differential deflections in the slab are limited to specific values which are a function of the deformation compatibility of the superstructure.

Although the design is based upon an assumed uncracked section, the effects of cracking were studied and found to be of no significant consequence, due to a post-cracking increase in slab support provided by the soil (see Section 5.2.F). This increase in soil support also prevented the rotations necessary to develop conventional cracked section "ultimate" strengths, thus ultimate strengths are not considered in the design procedure.

Ground-supported post-tensioned foundations addressed by this document are not specifically excluded from the requirements of the ACI Building Code[3]. Therefore, many ACI 318 requirements arguably apply to the design of post-tensioned ground-supported foundations. However, since this entire design method was uniquely developed for post-tensioned ground-supported applications and is supported by the successful performance of many thousands of existing foundations, it is intended that this document be independent of ACI 318 and shall govern in areas of conflict.

## 4.2 Design Parameters

A set of design parameters must be known to successfully design a slab-on-ground. These include data relating to climate, soil, and structure. The design parameters discussed below are applicable to both prestressed and non-prestressed slabs-on-ground. An outline of procedures that may be used by geotechnical engineers to evaluate design properties of an expansive soil mass is presented in Appendix A.3.

### (A) Climate

Clay soils with the potential to shrink or swell are found in almost all parts of the United States (Figure 4.1), but this potential is only realized in climates that have periods of rainfall followed by extended periods without rainfall. These semi-arid conditions are particularly evident in California and Texas, and to a slightly lesser degree in many of the great plains and other western states of the United States.

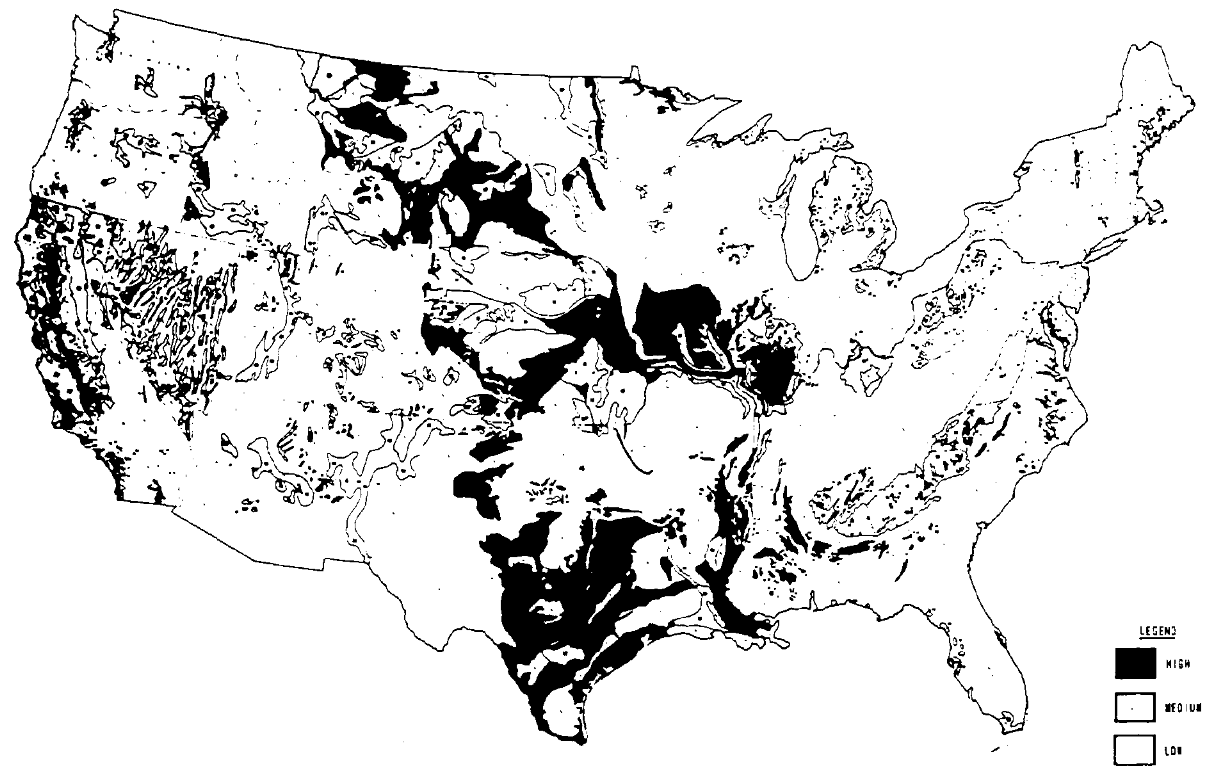



**Fig. 4.1** Distribution of expansive soils in the United States and their relative activity after Wiggins (104)*.

*Numbers in parenthesis refer to references in Appendix A.11

When designing foundations for use on expansive soils, the engineer must realize that damaging soil movement is not a necessary consequence of construction. If the structure is to be located at a site which has a high swelling potential but the climate is such that little change in the soil moisture content occurs, then there is little opportunity for detrimental swelling or shrinking to occur. If the site is in an area that has high rainfall or the climate remains relatively wet throughout the year, then the soil has probably already experienced considerable expansion; application of additional soil moisture will produce only a very small amount of additional swell.

The danger of a potentially high swelling soil in a region of wet climate derives less from swelling and more from soil shrinkage during periods of little or no rainfall. Conversely, if the site remains relatively dry throughout the year, then there is more opportunity for large differential swelling to occur. Thus, to arrive at a proper design, the engineer needs some environmental indicator or knowledge of the climate at the project site in order to estimate the severity of the shrink-swell activity of the soil on which the foundation will reside.

One such environmental indicator is the index of potential evapotranspiration which was introduced by Thornthwaite[88]. The Thornthwaite Moisture Index is defined as the amount of water which would be returned to the atmosphere by evaporation from the ground surface and transpiration by plants if there was an unlimited supply of water to the plants and soil. A map of this quantity as it is distributed across the United States is shown in Figure A.3.2 in Appendix A.3, and a larger scale map of the State of Texas and California is shown in Figure A.3.3.a. and A.3.3.b., respectively. The maps in Figures A.3.2, A.3.3.a and A.3.3.b., represent twenty year average values of the Thornthwaite Moisture Index for the period 1955-1974. A positive Thornthwaite Moisture Index $I_m$ value indicates a net surplus of soil moisture while a negative number indicates a net soil moisture deficit.

**(B) Soil Parameters**

(1) **Swelling Mode:** If the soil beneath the slab experiences a change in its moisture content after construction of the slab, it will distort into either a center lift mode (also termed "edge drying", "center heave" or "doming") or an edge lift mode (also called "edge swell", "edge heave" or "dishing"). The term "curling" is also occasionally used, mostly by non-technical or lay persons, to describe the edge lift condition. Center lift is a long term condition which occurs when the moisture content of the soil around the perimeter of the slab gradually decreases and the soil shrinks relative to the soil beneath the interior of the slab, or when the moisture content of the soil beneath the interior of the slab increases and the soil expands. Conversely, edge lift is, in general, a seasonal or short term condition and occurs when the soil beneath the perimeter becomes wetter than the soil beneath the interior of the slab. These two distortion (swelling) modes are depicted in Figure 4.2

(2) **Edge Moisture Variation Distance, $e_m$**: Also known as the edge penetration distance, $e_m$ is the distance measured inward from the edge of the slab over which the moisture content of the soil varies. An increasing moisture content at increasing distances inside the slab perimeter is indicative of a center lift condition whereas a decreasing moisture content indicates an edge lift situation. The magnitude of the moisture variation distance is dependent to a large degree upon the climate. For example, when center lift distortion occurs, a slab in a drier climate would tend to experience larger distances of drying soil around its edges than would a slab in a wetter climate. Drier climates would tend to experience smaller distances of moisture variation during edge lift swelling than during center lift distortion. This is due to the

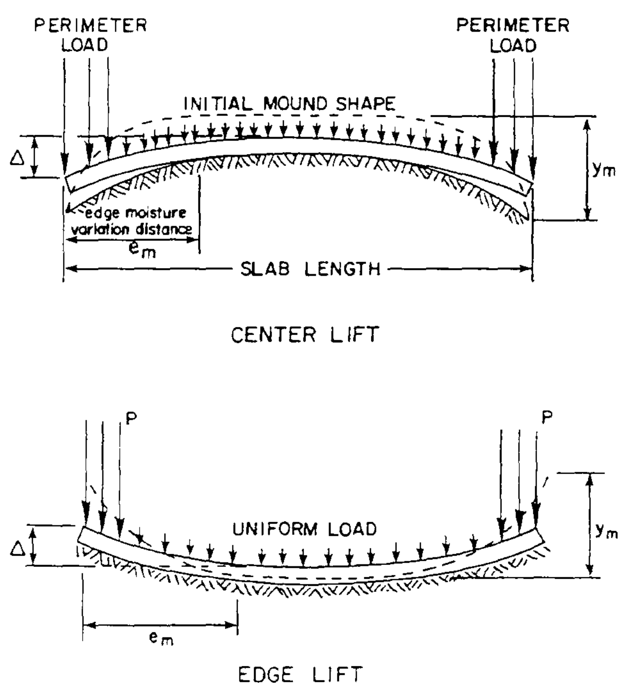



Fig. 4.2 Soil-structure interaction models (108)

strong evapotranspiration influences that tend to retard or reverse the moisture migration beneath the slab. Slabs constructed in wetter climates would have larger moisture variation distances during edge lift swelling due to the strong influence of the wetter environmment. The value of $\mathbf{e_m}$ to be used in structural design calculations should be provided in the soils investigation report submitted by the geotechnical engineer. An approximate procedure for evaluating the edge moisture variation distance on the basis of the Thornthwaite Moisture Index is presented in Appendix A.3, Figure A.3.4.

During the development of the PTI design procedure, banded curves were selected as aids to determine edge moisture variation distance, $\mathbf{e_m}$ in Figure A.3.4.

The lower lines of the bands were determined by back-calculation, using the PTI equations applied to post-tensioned residential slabs that had been in place for up to 10 years, including wet and dry years, and which were performing satisfactorily. These slabs represented sites with average conditions. Non climatic conditions such as vegetation, slope or poor drainage were not encountered.

However, because of the uncertainty of applying these curves over different soil conditions, upper parallel band curves were selected. The permeability of the soil is one factor which contributes to this uncertainty. On sites with more pervious soil, values closer to the upper lines could be chosen. On sites with less pervious soil, values closer to the lower lines could be selected. The banded curves represented primarily climatic conditions and a return weather pattern period of approximately 10 years.

Guidance on determining $\mathbf{e_m}$ values associated with return period weather patterns up to 50 years is provided in Lytton[117]. Slopes can also affect the selection of the $\mathbf{e_m}$ value and this relationship requires further study, see Lytton[118,119].

In choosing the appropriate $\mathbf{e_m}$ value, the designer is not limited to values within the band width. Severe non-climatic conditions could require a value of $\mathbf{e_m}$ to be selected that is greater than the upper line value, to adequately reflect the particular site conditions. However, the selection of $\mathbf{e_m}$ greater than the upper line value must not be considered as an alternate to permit substandard site preparation, particularly in the areas of grading, drainage and irrigation.

Once a value of $\mathbf{e_m}$ is selected, the VOLFLO computer program can be used to estimate the magnitude of $\mathbf{y_m}$ including the effects of trees and drainage[120].

Designers should ensure that calculations of center lift moments based on values of $\mathbf{e_m}$ greater than 5 ft. should not be less than those generated for the 5 ft. threshold. It should be recognized that better accuracy than the nearest 0.5 ft. (e.g. 4.5 ft) is not warranted when estimating $\mathbf{e_m}$ from Fig A.3.4.

(3) **Differential Soil Movement, $\mathbf{y_m}$** : Also known as Differential Swell, the amount of differential soil movement $\mathbf{y_m}$ to be expected depends upon a number of conditions, including the type and amount of clay mineral, depth of clay layers, uniformity of clay layers, the initial wetness, the depth of the active zone (depth of soil suction variation), the velocity of moisture infiltration or evaporation as well as other less easily measured and controlled effects. Effects which are more difficult to measure may include the type and amount of site post-construction and pre-construction vegetation cover, slope of the site, drainage conditions, irrigation, substantial local water delivery such as downspouts or leaking water supplies, and others. If these site conditions have been corrected so that soil moisture conditions are controlled **by the climate alone**, the amount of differential movement may be estimated by a geotechnical engineer. A procedure that may be used by geotechnical engineers to evaluate the **climate-controlled** differential soil movement is presented in Appendix A.3.

It must be emphasized that the determination of ym, and therefore the entire design method presented herein, is based solely upon climate-controlled soil conditions and is invalid when influenced to any significant degree by other conditions, including but not limited to those mentioned above and expanded upon in the following Section 4.2.(B)(4).

The design method is valid for $\mathbf{y_m}$ values up to and including 4 in. For $\mathbf{y_m}$ substantially over 4 in. a different type of foundation design method, such as finite element, should be considered.

(4) **Factors Not Related to Climate**: The use of an environmental indicator such as the Thornthwaite Moisture Index as an aid in estimating the amount of shrink-swell that a soil will exhibit does not account for factors causing soil movement that are not related to climate. Factors not related to climate may

induce soil movements many times larger than those resulting from climatic influences alone. While it may be possible to quantify the effects of many non-climatic factors, their presence or absence is often beyond the direct control of the structural and/or geotechnical engineer. In general, an effective means for mitigating non-climatic factors is to provide detailed limitations on construction and use on the plans and/or contract documents. Some designers and builders actually prepare "user's manuals" for the owners of homes on expansive soils, with detailed guidelines on irrigation, drainage, vegetation, slopes, and other non-climatic factors which may affect the performance of the foundation. The major factors influencing soil movement that are not related to climate are:

(a) **Pre-Vegetation.** Large individual trees, thickets or other vegetation requiring large amounts of moisture from the soil tend to make the soil in the areas reached by their roots drier than adjacent areas. These dessicated pockets have a much higher potential for swelling than do the adjacent, less dessicated areas.

(b) **Fence Lines, Trails, and Tracks.** These surface features typically have the vegetation worn away, leaving only bare or thinly covered strips which are much drier than the soil on either side. Like the dessicated areas caused by pre-construction vegetation, these areas will swell more than other areas.

(c) **Slopes.** Slopes comprised of active expansive soil have a tendency to migrate downhill as the soil experiences shrink-swell cycles.

(d) **Cut and Fill Sections.** Cut and fill sections will experience differential soil movement because of variations of compacted densities.

(e) **Drainage.** If rainfall runoff is allowed to pond or collect adjacent to a structure built on expansive soil, the structure may be subjected to distress caused by the soil beneath the structure swelling as a direct result of increased soil moisture content. Lot surfaces must be graded to drain away from the structure. Excess runoff should not be collected and disposed of by carrying a discharge pipe beneath the structure. Care should also be taken with sewage and water utility lines to ensure that leaks do not develop beneath the slab.

(f) **Time of Construction.** If the slab is cast at the end of a lengthy dry period, it may experience greater uplift around the edges when the soil becomes wetter at the conclusion of the dry period. Similarly, a slab cast at the end of a wet period, may experience greater drying around the edges during the subsequent period of dryness.

(g) **Post-Construction.** A number of post-construction practices beyond the control of the design engineer can occur to cause distress to structures founded on expansive clay. Planting flower beds or shrubs next to the foundation and keeping these areas flooded will generally cause a net increase in soil moisture content and result in soil expansion around the foundation perimeter in that vicinity. Planting shade trees closer to the structure than a distance equal to half the mature height of the tree will allow the tree roots to penetrate beneath the foundation and withdraw moisture from the soil; the result will be a soil shrinkage in the region of the roots. Redirecting surface runoff channels or swales by the owner can result in improper drainage as detailed above. To minimize movements in soils due to post-construction factors that are not climate related, the following home owners maintenance procedures are recommended:

(i) Initial landscaping should be done on all sides adjacent to the foundation and drainage away from the foundation should be provided and maintained.

(ii) Watering should be done in a uniform, systematic manner as equally as possible on all sides of the foundation to keep the soil moist. Areas of soil that do not have ground cover may require more moisture as they are more susceptible to evaporation. Ponding or trapping of water in localized areas adjacent to the foundations can cause differential moisture levels in subsurface soils.

(iii) Studies have shown that trees within 20 feet of foundations have caused differential movements in foundations. These will require more water in periods of extreme drought and in some cases a root injection system may be required to maintain moisture equilibrium.

(iv) During extreme hot and dry periods, close observations should be made around foundations to insure that adequate watering is being provided to keep soil from separating or pulling back from the foundation.

**(C) Structural Parameters**

(1) **Slab Shape**: The slab plan geometry is generally fixed by functional and architectural requirements.

(2) **Applicable Structural Systems**: The design procedure presented herein can be used for ribbed foundations (consisting of a uniform thickness slab with stiffening beams projecting from the bottom of the slab in both directions) and uniform thickness foundations (a solid slab with uniform thickness and no interior stiffening beams).

(a) **Ribbed Foundations:**

(i) **Stiffening Beam Spacing:** For ribbed foundations, the location of stiffening beams is dictated mainly by the configuration of the foundation system, the structural design requirements, and the wall layout of the superstructure. **Beam spacing S shall be a maximum of 17 feet.** A minimum beam spacing **S** of 6 feet shall be used in the design of ribbed slabs, however, the actual spacing may be less than that if desired. Additional beams may be required where heavy loads are applied to the foundation, as in the case of a fireplace or an interior column.

When beam spacings vary, the average spacing may be used for design unless the ratio between the largest and smallest spacing exceeds 1.5. In that case, the design spacing shall be 0.85 times the largest spacing.

Corners of ribbed foundations require special consideration. Bending moments are biaxial near corners, affected by both long and short direction bending. For foundations with widely spaced ribs, the line of maximum moment around a corner may not cross a rib. Additional ribs, or a diagonal rib extending from the corner to the intersection of the first orthogonal ribs, may be advisable to insure proper performance at corners.

(ii) **Stiffening Beam Depth**: The depth of stiffening beams **h** is usually the controlling parameter in the structural design of ribbed foundations. Beam depth is the structural parameter which most influences the moment capacity, shear capacity, and deflections in the ribbed foundation. Frost depth, where applicable, may be a controlling factor for determining minimum edge beam depth. For consistency with the computer study used to develop this design method, the design must be limited to a constant beam depth for all beams in both directions. Different beam depths may, of course, be used in the as-built foundation (such as a deeper edge beam), however, the design in that case must be based upon the smallest beam depth used. In addition, the total beam depth **h** shall be in no case less than 12", and the beam must extend at least 7" below the bottom of the slab (**h ≥ t + 7"**).

(iii) **Stiffening Beam Width**: The width of stiffening beams **b** affects the soil bearing capacity, the applied shear stress, and all section properties. To insure the accuracy of equations for applied service moments, shears, and deflections (in which **b** does not appear), the beam width used in section property calculations must be limited to a range of 8" to 14". Within this range the flexural design is virtually unaffected by the beam width. Beam widths less than 8" wide are impractical due to excavation considerations. Beam widths greater than 14" may be used if required for bearing, however, in that case a width of 14" shall be used in section property calculations. Stiffening beam widths most commonly found in practice are 10" to 12". Observations of numerous slabs built on soils with low bearing values and using larger bearing areas (containing a portion of the slab in addition to the beam width) have shown satisfactory performance.

(b) **Uniform Thickness Foundations**:

To design a uniform thickness foundation the designer must first design a ribbed foundation for moment, shear, and differential deflection, and then convert the ribbed foundation to a uniform thickness foundation using a conversion equation. The original ribbed foundation must conform to all of the moment, shear, and differential deflection requirements for ribbed foundations, including the limitations on beam spacing, depth, and width listed above in Sections 4.2(C)(2)(a)(i) through (iii). The uniform thickness of this type of foundation should be limited to a minimum of 7.5", unless a continuous stiffening beam, conforming to the requirements of Section 4.2(C)(2)(a)(ii) and (iii), is provided at the entire perimeter of the foundation.

(3) **Loading**: The loading applied to the foundation is governed by applicable building codes, the architecture of the building, framing, and the materials of construction. The design procedure developed herein assumes the following loadings on the foundation, the first two constant (built into the procedure), the third variable and determined by the designer:

(a) A uniform 40 psf live load applied over the entire plan area of the foundation.

(b) A uniform 65 psf dead load applied over the entire plan area of the foundation (representing the weight of an assumed 4" slab plus 15 psf for partitions and other interior dead loads).

(c) A uniform unfactored service line load **P** acting along the entire length of the perimeter stiffening beams representing the weight of the exterior building material and that portion of the superstructure dead and live loads which frame into the exterior wall. **P** does not include any portion of the foundation concrete, (e.g., the weight of the stiffening beam concrete).

The actual perimeter line loadings **P** used to develop this method ranged between 600 and 1,500 plf. This procedure does not apply for slabs with interior uniform loads substantially in excess of those described above. Unusually heavy concentrated loads, such as fireplaces, post loads, or interior bearing walls, must be evaluated on an individual basis. A formula for calculation of tensile stresses beneath concentrated loads is presented in Chapter 6, Section 6.14. If the slab stresses produced by concentrated loads exceed those permissible, the loads should be framed to adjacent stiffening beams in ribbed foundations, or a footing should be placed below them in uniform thickness foundations.

The structural engineer must carefully evaluate the assumption of uniform perimeter line loading as it applies to the modeling of the specific foundation under design. Actual framing can produce loads substantially different from this assumption. For example, in a rectangular building framed in the short direction, the perimeter load on the short edges will be very small (only the weight of the walls themselves) while the long edges will carry substantially all of the superstructure load (half on each side). In this case the assumption of uniform perimeter loading would not accurately represent the moments produced by the actual loading. In any case, the largest load intensity occurring anywhere on the perimeter should be used for center lift design and the smallest load intensity for edge lift design.

(4) **Allowable Shear Stress**: The equation for allowable shear stress (Equation (9) in Section 6.5(D)) has been revised from the previous edition to reflect both concrete strength and prestress compression. In developing the current equation, the Committee researched the relationship between the vertical shear stress and the principal tension stress, documented recommended values which have been used for the permissible principal tension stress, and the relationship of these values to current ACI Code equations for permissible vertical shear stresses.

# 6.0 STRUCTURAL DESIGN PROCEDURE FOR SLABS ON EXPANSIVE SOILS

## 6.1 General

On the basis of the design parameters discussed in Chapter 4, and the results of the soil-structure interaction analysis described in Chapter 5, specific structural design formulas and procedures for moment, shear, deflection and slab-subgrade friction are presented in this chapter for slabs on expansive soils. An equation is developed for calculation of the stress due to concentrated line loads on slabs. This chapter also addresses some slab-on-ground applications other than those on expansive soils, including non-post-tensioned slabs on ground, slabs subject to frost heave, and slabs constructed on compressible soils. These applications are found in Section 6.13.

This procedure can be used for slabs with stiffening beams (ribbed foundations) or uniform thickness foundations. To design a uniform thickness foundation the designer must first design a ribbed foundation which satisfies all requirements of the design procedure for ribbed foundations. The fully conformant ribbed foundation is then converted to an equivalent uniform thickness foundation.

The design procedure for post-tensioned foundations constructed over expansive clays should include the following steps, with the pertinent sections of this chapter shown in parenthesis:

(A) Assemble all the known design data (6.2).
(B) Divide an irregular foundation plan into overlapping rectangles and design each rectangular section separately (6.3).
(C) Assume a trial section for a ribbed foundation in both the long and short directions of the design rectangle (6.4).
(D) Calculate the applied service moment the section will be expected to experience in each direction for either the center lift or edge lift condition (6.8).
(E) Determine the flexural concrete stresses by the applied service moments and compare to the allowable flexural concrete stresses (6.8 and 6.5).
(F) Determine the expected differential deflections and compare with the allowable differential deflections (6.10).
(G) Calculate the applied service shear force and shear stress in the assumed sections and compare the applied shear stress with the allowable shear stress (6.11).
(H) Convert the ribbed foundation to an equivalent uniform thickness foundation, if desired (6.12).
(I) Repeat Steps 4 through 8 for the opposite swelling condition.
(J) Check the design for the first swelling condition to ascertain if adjustments are necessary to compensate for any design changes resulting from the second design swelling condition addressed in Step 9.
(K) Check the effect of slab-subgrade friction to assure a residual compressive stress of 50 psi at the center of each design rectangle in both directions. Adjust post-tensioning force if necessary (6.7).
(L) Calculate stresses due to any heavy concentrated loads on the slab and provide special load transfer details when necessary (6.14).

The design procedure for foundations on compressible soils is similar to the above except that different equations are used and the primary bending deformation is usually similar to that shown in Figure 4.2 for the edge lift loading case (refer to Section 6.13).

## 6.2 Required Design Data

The soils and structural properties needed for design are as follows:

(A) Soils Properties
   (1) Allowable soil bearing pressure, $q_{allow}$, in pounds per square foot.
   (2) Edge moisture variation distance, $e_m$, in feet.
   (3) Differential soil movement, $y_m$, in inches.
   (4) Slab-subgrade friction coefficient, $\mu$.
(B) Structural data and materials properties
   (1) Slab length, **L**, in feet (both directions).
   (2) Perimeter loading, **P**, in pounds per foot.
   (3) Average stiffening beam spacing, **S**, in feet (both directions).
   (4) Beam depth, **h**, in inches.
   (5) Compressive strength of the concrete, $f'_c$, in pounds per square inch.
   (6) Allowable flexural tensile stress in the concrete, $f_t$, in pounds per square inch.
   (7) Allowable compressive stress in the concrete, $f_c$, in pounds per square inch.

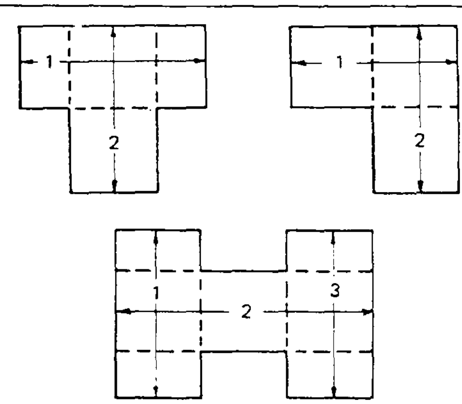



Fig. 6.1 Design rectangles for slabs of irregular shape

(8) Type, grade, and strength of the prestressing steel.
(9) Prestress losses, ksi.

### 6.3 Slabs of Irregular Shape

Slabs of irregular shape should be divided into overlapping rectangles so that the resulting boundary provides complete congruence with the slab perimeter. See Figure 6.1 for examples. A separate design must be made for each of the component rectangles of the slab (except for instances where the overlapping rectangles are of nearly similar dimensions).

### 6.4 Trial Section Assumptions

(A) **Assume Beam Depth and Spacing**: An initial estimate of the depth of the stiffening beams can be obtained from solving either Equation (24) or Equation (25) for the beam depth yielding the maximum allowable differential deflection. The procedure is as follows:

(1) Determine the maximum distance over which the allowable differential deflection will occur, **L** or **6β** whichever is smaller. As a first approximation, use $\beta$ = 8 feet.

(2) Select the allowable differential deflection:

(a) Center Lift (assume $\mathbf{C_\Delta}$ = 360):

$$\Delta_{allow} = \frac{12(L \text{ or } 6\beta)}{C_\Delta} = \frac{12(L \text{ or } 6\beta)}{360} \quad (1)$$

(b) Edge Lift (assume $\mathbf{C_\Delta}$ = 720):

$$\Delta_{allow} = \frac{12(L \text{ or } 6\beta)}{C_\Delta} = \frac{12(L \text{ or } 6\beta)}{720} \quad (2)$$

Alternatively, $\mathbf{C_\Delta}$ may be selected from Table 6.2, which presents sample $\mathbf{C_\Delta}$ values for various types of superstructures.

(3) Assume a beam spacing, **S**, and solve for beam depth, **h**:

(a) Center Lift (from Equation 24):

$$h^{1.214} = \frac{(y_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{380\Delta_{allow}} \quad (3a)$$

$$h = \left[\frac{(y_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{380\Delta_{allow}}\right]^{0.824} \quad (3b)$$

(b) Edge Lift (from Equation 25):

$$h^{0.85} = \frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(y_m)^{0.76}}{15.9\Delta_{allow}(P)^{0.01}} \quad (4a)$$

$$h = \left[\frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(y_m)^{0.76}}{15.9\Delta_{allow}(P)^{0.01}}\right]^{1.176} \quad (4b)$$

Select the larger **h** from Equation (3b) or (4b). In the analysis procedure, the beam depth **h** must be the same for all beams in both directions (see Section 4.2(C)(2)(a)(ii)). If different beam depths are selected for the actual structure (such as a deeper edge beam), the analysis shall be based upon the smallest beam depth actually used.

(B) **Determine Section Properties**: The moment of inertia, section modulus, and cross-sectional area of the slabs and beams, and eccentricity of the prestressing force may be calculated for the trial beam depth determined above in accordance with normal structural engineering procedures. These procedures are illustrated in the design examples presented in Appendices A.5, A.6, A.7, and A.8.

### 6.5 Allowable Stresses

The following allowable stresses are recommended:

(A) Allowable Concrete Flexural Tensile Stress:

$$f_t = 6\sqrt{f'_c} \quad (5)$$

(B) Allowable Concrete Flexural Compressive Stress:

$$f_c = 0.45f'_c \quad (6)$$

(C) Allowable Concrete Bearing Stress at Anchorages:

(1) At Service Load:

$$f_{bp} = 0.6f'_c\sqrt{\frac{A'_b}{A_b}} \le f'_c \quad (7)$$

(2) At Transfer:

$$f_{bp} = 0.8f'_{ci}\sqrt{\frac{A'_b}{A_b} - 0.2} \le 1.25f'_{ci} \quad (8)$$

(D) Allowable Concrete Shear Stress:

$$v_c = 1.7\sqrt{f'_c} + 0.2f_p \quad (9)$$

(E) Allowable Stresses in Prestressing Steel:

(1) Allowable stress due to tendon jacking force:

$$f_{pj} = 0.8f_{pu} \le 0.94f_{py} \quad (10)$$

(2) Allowable stress immediately after prestress transfer:

$$f_{pi} = 0.7f_{pu} \quad (11)$$

### 6.6 Prestress Losses

Loss of prestress due to friction, elastic shortening, creep and shrinkage of the concrete, and steel relaxation shall be calculated in accordance with "Estimating Prestress Losses" by Zia, Preston, Scott and Workman[118]. Using loss terminology from this document, the effective prestress force $\mathbf{P_e}$ is:

$$P_e = P_i - ES - CR - SH - RE \qquad (12)$$

### 6.7 Slab-Subgrade Friction

The effective prestressing force in post-tensioned slabs-on-ground is further reduced by the frictional resistance to movement of the slab on the subgrade during stressing as well as the frictional resistance to dimensional changes due to concrete shrinkage, creep, and temperature variations. The resultant prestress force Pr is the difference between the effective prestress force and the losses due to subgrade friction:

$$P_r = P_e - SG \qquad (13)$$

where SG can be conservatively taken as:

$$SG = \frac{W_{slab}}{2{,}000}\mu \qquad (14)$$

The largest amount of prestress loss due to slab-subgrade friction occurs in the center regions of the slab. The greatest structural requirement for prestress force, however, is at the location of the maximum moment, which occurs at approximately one β-length inward from the edge of the slab. For normal construction practices, the value of the coefficient of friction $\mu$ should be taken as 0.75 for slabs on polyethylene and 1.0 for slabs cast directly on a sand base (also see Section 5.5 for additional discussion of slab-subgrade friction).

The maximum spacing of tendons should not exceed that which would produce a minimum average effective prestress compression of 50 psi after allowance for slab-subgrade effects. The Engineer of Record should limit the value of maximum spacing based on local experience. Common practice indicates this value normally falls in the 5 ft. to 6 ft. range.

### 6.8 Maximum Applied Service Moments

The maximum moment will vary depending upon the swelling mode and the slab direction being designed. For design rectangles with a ratio of long side to short side less than 1.1, the equations for $\mathbf{M_L}$ (Equations (15) and (19)) shall be used for moments in both directions.

(A) Center Lift Moment

(1) <u>Long Direction</u>

$$M_L = A_o\left[B(e_m)^{1.238} + C\right] \qquad (15)$$

where:

$$A_o = \frac{1}{727}\left[(L)^{0.013}(S)^{0.306}(h)^{0.688}(P)^{0.534}(y_m)^{0.193}\right] \qquad (16)$$

and for:

$$0 \le e_m \le 5 \qquad B = 1,\ C = 0 \qquad (17a)$$

$$e_m > 5 \qquad B = \left(\frac{y_m - 1}{3}\right) \le 1.0 \qquad (17b)$$

$$C = \left[8 - \frac{P - 613}{255}\right]\left[\frac{4 - y_m}{3}\right] \ge 0 \qquad (17c)$$

(2) <u>Short Direction</u>

For $L_L / L_S \ge 1.1$

$$M_S = \left[\frac{58 + e_m}{60}\right]M_L \qquad (18)$$

For $L_L / L_S < 1.1$

$$M_S = M_L$$

(B) Edge Lift Moment:

(1) <u>Long Direction</u>

$$M_L = \frac{(S)^{0.10}(he_m)^{0.78}(y_m)^{0.66}}{7.2(L)^{0.0065}(P)^{0.04}} \qquad (19)$$

(2) <u>Short Direction</u>

For $L_L / L_S \ge 1.1$

$$M_S = h^{0.35}\left[\frac{19 + e_m}{57.75}\right]M_L \qquad (20)$$

For $L_L / L_S < 1.1$

$$M_S = M_L$$

Concrete flexural stresses produced by the applied service moments can be calculated with the following equation:

$$f = \frac{P_r}{A} \pm \frac{M_{L,S}}{S_{t,b}} \pm \frac{P_r e}{S_{t,b}} \qquad (21)$$

The applied concrete flexural stresses **f** must be limited to $\mathbf{f_t}$ in tension and $\mathbf{f_c}$ in compression.

## 6.9 Cracked Section Considerations

This design method limits concrete flexural tensile stresses to $\mathbf{6\sqrt{f'_c}}$ . Since the modulus of rupture of concrete is commonly taken as $\mathbf{f_{cr} = 7.5\sqrt{f'_c}}$ , slabs designed with this method will theoretically have no flexural cracking. Some cracking from restraint to slab shortening is inevitable in post-tensioned slabs on ground, as it is in elevated post-tensioned concrete members. Nonetheless, the limitation of flexural tensile stresses to a value less than the modulus of rupture justifies the use of the gross effective concrete cross-section for calculating all section properties. Refer to Section 5.2 for additional discussion on the effects of cracking in post-tensioned ground-supported foundations.

## 6.10 Differential Deflections

Allowable and expected differential deflections may be calculated from the equations presented in the following sections.

(A) Relative stiffness length, **β**, may be calculated as follows:

$$\beta = \frac{1}{12}\sqrt[4]{\frac{E_c I}{E_s}} \qquad (22)$$

If the creep modulus of elasticity of the concrete $\mathbf{E_c}$ is not known, it can be closely approximated by using half of the normal or early life concrete modulus of elasticity. If the modulus of elasticity of the clay soil $\mathbf{E_s}$ is not known, use 1,000 psi. **I** in Equation (22) is the gross moment of inertia for the entire slab cross-section of width **W**, in the appropriate direction (long or short).

(B) Differential Deflection Distance:
The differential deflection may not occur over the entire length of the slab, particularly if the slab is longer than approximately 50 feet. Thus, the effective distance for determining the allowable differential deflection is the smaller of the two distances, **L** or **6β**, both expressed in feet.

(C) Allowable Differential Deflection $\Delta_{allow}$:
(1) Center Lift or Edge Lift:

$$\Delta_{allow} = \frac{12(L \text{ or } 6\beta)}{C_\Delta} \qquad (23)$$

The coefficient $\mathbf{C_\Delta}$ is a function of the type of superstructure material and the swelling condition (center or edge lift). Sample values of $\mathbf{C_\Delta}$ for both swelling conditions and various superstructure materials are shown in Table 6.2.

(D) Expected Differential Deflection Without Prestressing, $\mathbf{\Delta_o}$:
(1) Center Lift:

$$\Delta_o = \frac{(y_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{380(h)^{1.214}} \qquad (24)$$

(2) Edge Lift:

$$\Delta_o = \frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(y_m)^{0.76}}{15.9(h)^{0.85}(P)^{0.01}} \qquad (25)$$

(E) Deflection Caused by Prestressing, $\Delta_p$:
Additional slab deflection is produced by prestressing if the prestressing force at the slab edge is applied at any point other than the **CGC**. The deflection caused by prestressing can be approximated with reasonable accuracy by assuming it is produced by a concentrated moment of $\mathbf{P_e e}$ applied at the end of a cantilever with a span length of **β**. The deflection is:

$$\Delta_p = \frac{P_e e \beta^2}{2E_c I} \qquad (26)$$

If the tendon **CGS** is higher than the concrete **CGC** (a typical condition), $\Delta_p$ increases the edge lift deflection and decreases the center lift deflection. Deflection caused by prestressing is normally small and can justifiably be ignored in the design of most post-tensioned slabs on ground.

(F) Compare Expected to Allowable Differential Deflection:
If the expected differential deflection as calculated by either Equations (24) or (25), adjusted for the effect of prestressing, exceeds that determined from Equation (23) for the appropriate swelling condition, the assumed section must be stiffened.

$$\Delta = \Delta_o \pm \Delta_p \qquad (27)$$

### 6.11 Shear

(A) Applied Service Load Shear:
Expected values of service shear forces in kips per foot of width of slab and stresses in kips per square inch may be calculated from the following formulas:

(1) Center Lift:
(a) Long Direction Shear

$$V_L = \frac{1}{1940}\left[(L)^{0.09}(S)^{0.71}(h)^{0.43}(P)^{0.44}(y_m)^{0.16}(e_m)^{0.93}\right] \tag{28}$$

(b) Short Direction Shear

$$V_S = \frac{1}{1350}\left[(L)^{0.19}(S)^{0.45}(h)^{0.20}(P)^{0.54}(y_m)^{0.04}(e_m)^{0.97}\right] \tag{29}$$

(2) Edge Lift (for both directions):

$$V_L \text{ or } V_S = \frac{(L)^{0.07}(h)^{0.4}(P)^{0.03}(e_m)^{0.16}(y_m)^{0.67}}{3.0(S)^{0.015}} \tag{30}$$

(B) Applied Service Load Shear Stress, **v**:
Only the beams are considered in calculating the cross-sectional area resisting shear force in a ribbed slab:

(1) Ribbed Foundations:

$$v = \frac{VW}{nhb} \tag{31}$$

(2) Uniform Thickness Foundations:

$$v = \frac{V}{12H} \tag{32}$$

(C) Compare **v** to $\mathbf{v_c}$:
If **v** exceeds $\mathbf{v_c}$, shear reinforcement in accordance with ACI 318 must be provided. Possible alternatives to shear reinforcement include:
(1) Increasing the beam depth,
(2) Increasing the beam width;
(3) Increasing the number of beams (decrease the beam spacing).

### 6.12 Uniform Thickness Conversion

Once the ribbed foundation has been designed to satisfy moment, shear, and differential deflection requirements, it may be converted to an equivalent uniform thickness foundation with thickness **H**, if desired. The following equation for **H** shall be used for the conversion:

$$H = \sqrt[3]{\frac{I}{W}} \tag{33}$$

### 6.13 Other Applications of Design Procedure

The design procedure presented in this manual has other practical slab-on-ground applications besides construction on expansive clays, as discussed below:

(A) Design of Non-Prestressed Slabs-on-Ground:
Equations (15), (18)-(20), (24), (25), and (28)-(30) predict the values of bending moment, shear, and differential deflection expected to occur using a given set of soil and structural parameters. These design values may be calculated for slabs reinforced with unstressed as well as stressed reinforcement. Once these design parameters are known, design of either type of slab can proceed. This design manual does not provide design procedures for non-prestressed slabs-on-ground. However, to conform to the same deflection criteria, non-prestressed slabs designed on the basis of cracked sections will need significantly deeper beam stems than prestressed slabs.

(B) Design of Slabs Subject to Frost Heave:
The applied moments, shears and deflections due to frost heave can be approximated by substituting anticipated frost heave for expected swell of an expansive clay. The value of $\mathbf{e_m}$ for frost heave must be estimated from values comparable to those for expansive soils.

(C) Slabs-on-Ground Constructed on Compressible Soils:
Design of slabs constructed on compressible soils can be done in a manner similar to that of the edge lift condition for slabs on expansive soils. Compressible soils are normally assumed to have allowable values of soil bearing capacity, $\mathbf{q_{allow}}$, equal to or less than 1,500 pounds per square foot. Special design equations are necessary for this problem due to the expected in situ elastic property differences between compressible soils and the stiffer expansive soils. This procedure is illustrated in the design example presented in Appendix A.8. These equations are:

(1) Moment:
(a) Long Direction

$$M_{cs_L} = \left[\frac{\delta}{\Delta_{ns_L}}\right]^{0.5} M_{ns_L} \tag{34}$$

where:

$$M_{ns_L} = \frac{(h)^{1.35}(S)^{0.36}}{80(L)^{0.12}(P)^{0.10}} \quad (35)$$

$$\Delta_{ns_L} = \frac{(L)^{1.28}(S)^{0.80}}{133(h)^{0.28}(P)^{0.62}} \quad (36)$$

And $\delta$ = Expected settlement, reported by the geotechnical engineer, occurring in compressive soil due to the total load expressed as a uniform load, in.

(b) Short Direction

$$M_{cs_S} = \left(\frac{970 - h}{880}\right) M_{cs_L} \quad (37)$$

(2) Differential Deflection:

$$\beta = \frac{1}{12}\sqrt[4]{\frac{E_c I}{E_s\left(\frac{\delta}{\Delta_{ns}}\right)}} \quad (38)$$

$$\Delta_{cs} = \delta e_n\left[1.78 - 0.103(h) - 1.65 \times 10^{-3}(P) + 3.95 \times 10^{-7}(P^2)\right] \quad (39)$$

(3) Shear:

(a) Long Direction

$$V_{cs_L} = \left[\frac{\delta}{\Delta_{ns_L}}\right]^{0.30} V_{ns_L} \quad (40)$$

where:

$$V_{ns_L} = \frac{(h)^{0.90}(PS)^{0.30}}{550(L)^{0.10}} \quad (41)$$

(b) Short Direction:

$$V_{cs_S} = \left[\frac{116 - h}{94}\right] V_{cs_L} \quad (42)$$

## 6.14 Calculation of Stress in Slabs Due to Load Bearing Partitions

The equation for the tensile stress in a slab beneath a bearing partition may be derived from beam-on-elastic foundation theory. The maximum moment directly under a point load **P** in such a beam is:

$$M_{max} = -\frac{P\beta}{4} \quad (43)$$

where:

$$\beta = \left[\frac{4E_c I}{k_s B_w}\right]^{0.25} \leq S_{t,b} \quad (44)$$

with $\mathbf{E_c}$ = 1,500,000 psi and $\mathbf{k_s}$ = 4 pci:

$$\frac{I}{B_w} = \frac{B_w t^3}{12 B_w} = \frac{t^3}{12}$$

$$\beta = \left[\frac{4(1{,}500{,}000)t^3}{4(12)}\right]^{0.25} = 18.8t^{0.75} \quad (45)$$

therefore:

$$M_{max} = -\frac{18.8Pt^{0.75}}{4} = -4.7Pt^{0.75} \quad (46)$$

The equation for applied tensile stress **f** is:

$$f = \frac{P_r}{A} - \frac{M_{max}c}{I} \quad (47)$$

and since:

$$\frac{I}{c} = \frac{B_w t^3}{12}\left(\frac{2}{t}\right) = \frac{B_w t^2}{6} = \frac{12t^2}{6} = 2t^2$$

the applied tensile stress is:

$$f = \frac{P_r}{A} - \frac{4.7Pt^{0.75}}{2t^2} = \frac{P_r}{A} - 2.35\frac{P}{t^{1.25}} \quad (48)$$

$$= \frac{P_r}{A} - C_P\frac{P}{t^{1.25}}$$

For uniform thickness foundations substitute **H** for **t** in Equations (45), (46) and (48). The value of $\mathbf{C_p}$ depends upon the assumed value of the subgrade modulus $\mathbf{k_s}$. Table 6.1 illustrates the variation in $\mathbf{C_p}$ for different values of $\mathbf{k_s}$:

| Type of Subgrade | $k_s$, lb/in³ | $C_p$ |
|---|---|---|
| Lightly compacted, high plastic, compressible soil | 4 | 2.35 |
| Compacted, low plastic soil | 40 | 1.34 |
| Stiff, compacted, select granular or stabilized fill | 400 | 0.74 |

**Table 6.1**

If the allowable tensile stress (say $6\sqrt{f'_c}$ ) is exceeded by the results of the above analysis, a thicker slab section should be used under the loaded area, or a stiffening beam should be placed directly beneath the concentrated line load.

The following values are intended for **design** criteria only. The evaluation of existing slabs for deflection involves considerable engineering judgment because flexural deflection must be separated from construction effects (built-in out-of-levelness, for example.) Ideally, this can be done using an initial level survey made immediately after the slab is cast. Lacking an initial survey, accepted construction tolerances (such as those found in ACI 302) must be used to estimate construction effects.

**Sample Values of $C_\Delta$**

| Material | Center Lift | Edge Lift |
|---|---|---|
| Wood Frame | 240 | 480 |
| Stucco or Plaster | 360 | 720 |
| Brick Veneer | 480 | 960 |
| Concrete Masonry Units | 960 | 1920 |
| Prefab Roof Trusses* | 1000 | 2000 |

**Table 6.2**

* Trusses which clearspan the full length or width of the foundation from edge to edge.

# EXCERPTS FROM 1997 UNIFORM BUILDING CODE SHOWING THE LATEST VARIATIONS ON THE POST-TENSIONING INSTITUTE'S DESIGN EQUATIONS AND PROCEDURES

# Division III—DESIGN STANDARD FOR DESIGN OF SLAB-ON-GROUND FOUNDATIONS TO RESIST THE EFFECTS OF EXPANSIVE SOILS AND COMPRESSIBLE SOILS

## SECTION 1815 — DESIGN OF SLAB-ON-GROUND FOUNDATIONS [BASED ON DESIGN OF SLAB-ON-GROUND FOUNDATIONS OF THE WIRE REINFORCEMENT INSTITUTE, INC. (AUGUST, 1981)]

**1815.1 Scope.** This section covers a procedure for the design of slab-on-ground foundations to resist the effects of expansive soils in accordance with Division I. Use of this section shall be limited to buildings three stories or less in height in which gravity loads are transmitted to the foundation primarily by means of bearing walls constructed of masonry, wood or steel studs, and with or without masonry veneer.

1815.2 Symbols and Notations.

$1\text{-}c$ = soil/climatic rating factor. See Figure 18-III-8.

$A_s$ = area of steel reinforcing (square inch per foot) (mm$^2$ per m) in slab. See Figure 18-III-1.

$C_o$ = overconsolidation coefficient. See Figure 18-III-2.

$C_s$ = soil slope coefficient. See Figure 18-III-3.

$C_w$ = climatic rating. See Figure 18-III-4.

$E_c$ = creep modulus of elasticity of concrete.

$f_y$ = yield strength of reinforcing.

$I_c$ = cracked moment of inertia of cross section.

$k_l$ = length modification factor-long direction. See Figure 18-III-5.

$k_s$ = length modification factor-short direction. See Figure 18-III-5.

$L$ = total length of slab in prime direction.

$L'$ = total length of slab (width) perpendicular to $L$.

$L_c$ = design cantilever length ($l_c k$)—See Figures 18-III-5 and 18-III-6.

$l_c$ = cantilever length as soil function.

$M_l$ = design moment in long direction.

$M_s$ = design moment in short direction.

$PI$ = plasticity index.

$S$ = maximum spacing of beams. See Figure 18-III-7.

$V$ = design shear force (total).

$w$ = weight per square foot (N/m$^2$) of building and slab.

$q_u$ = unconfined compressive strength of soil.

$\Delta$ = deflection of slab, inch (mm).

**1815.3 Foundation Investigation.** A foundation investigation of the site shall be conducted in accordance with the provisions of Section 1804.

**1815.4 Design Procedure.**

**1815.4.1 Loads.** The foundation shall be designed for a uniformly distributed load which shall be determined by dividing the actual dead and live loads for which the superstructure is designed, plus the dead and live loads contributed by the foundation, by the area of the foundation.

**EXCEPTIONS:** 1. For one-story metal and wood stud buildings, with or without masonry veneer, and when the design floor live load is 50 pounds per square foot (2.4 kN/m$^2$) or less, a uniformly distributed load of 200 pounds per square foot (9.6 kN/m$^2$) may be assumed in lieu of calculating the effects of specific dead and live loads.

2. Those conditions where concentrated loads are of such magnitude that they must be considered are not covered by this section.

**1815.4.2 Determining the effective plasticity index.** The effective plasticity index to be used in the design shall be determined in accordance with the following procedures:

1. The plasticity index shall be determined for the upper 15 feet (4572 mm) of the soil layers and where the plasticity index varies between layers shall be weighted in accordance with the procedures outlined in Figure 18-III-9.

2. Where the natural ground slopes, the plasticity index shall be increased by the factor $C_s$ determined in accordance with Figure 18-III-3.

3. Where the unconfined compressive strength of the foundation materials exceeds 6,000 pounds per square foot (287.4 kPa), the plasticity index shall be modified by the factor $C_o$ determined in accordance with Figure 18-III-2. Where the unconfined compressive strength of the foundation materials is less than 6,000 pounds per square foot (287.4 kPa), the plasticity index may be modified by the factor $C_o$ determined in accordance with Figure 18-III-2.

The value of the effective plasticity index is that determined from the following equation:

Effective plasticity index = weighted plasticity index $\times C_s \times C_o$

Other factors that are capable of modifying the plasticity index such as fineness of soil particles and the moisture condition at the time of construction shall be considered.

**1815.5 Beam Spacing and Location.** Reinforced concrete beams shall be provided around the perimeter of the slab, and interior beams shall be placed at spacings not to exceed that determined from Figure 18-III-7. Slabs of irregular shape shall be divided into rectangles (which may overlap) so that the resulting overall boundary of the rectangles is coincident with that of the slab perimeter. See Figure 18-III-10.

**1815.6 Beam Design.** The following formulas shall be used to calculate the moment, shear and deflections, and are based on the assumption that the zone of seasonal moisture changes under the perimeter of the slab is such that the beams resist loads as a cantilever of length $L_c$:

$$M = \frac{wL'(L_c)^2}{2}$$

$$V = wL'L_c$$

$$\Delta = \frac{wL'(L_c)^4}{4E_cI_c}$$

The calculations shall be performed for both the long and short directions. Deflection shall not exceed $L_c/480$.

**1815.7 Slab Reinforcing.** The minimum slab thickness shall be 4 inches (102 mm), and the maximum spacing of reinforcing bars shall be 18 inches (457 mm). The amount of reinforcing shall be determined in accordance with Figure 18-III-1. Slab reinforcing shall be placed in both directions at the specified amounts and spacing.

2.2 Edge lift (assume $C_\Delta = 720$):

$$\Delta_{allow} = \frac{12(L \text{ or } 6\beta)}{C_\Delta} = \frac{12(L \text{ or } 6\beta)}{720} \quad (16\text{-}2)$$

**For SI:** 1 inch = 25.4 mm.

Alternatively, $C_\Delta$ may be selected from Table 18-III-GG, which presents sample $C_\Delta$ values for various types of superstructures.

3. Assume a beam spacing, $S$, and solve for beam depth, $h$:

3.1 Center lift (from Formula 16-20):

$$h^{1.214} = \frac{(y_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{380\Delta_{allow}} \quad (16\text{-}3\text{-}1)$$

$$h = \left(\frac{(y_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{380\Delta_{allow}}\right)^{0.824} \quad (16\text{-}3\text{-}2)$$

**For SI:** 1 inch = 25.4 mm.

3.2 Edge lift (from Formula 16-21):

$$h^{0.85} = \frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(y_m)^{0.76}}{15.9\Delta_{allow}(P)^{0.01}} \quad (16\text{-}4\text{-}1)$$

$$h = \left(\frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(y_m)^{0.76}}{15.9\Delta_{allow}(P)^{0.01}}\right)^{1.176} \quad (16\text{-}4\text{-}2)$$

**For SI:** 1 inch = 25.4 mm.

Select the larger $h$ from Formula (16-3-2) or (16-4-2). In the analysis procedure, the beam depth $h$ must be the same for all beams in both directions. If different beam depths are selected for the *actual* structure (such as a deeper edge beam), the analysis shall be based on the smallest beam depth actually used.

**1816.4.3.2 Determine section properties.** The moment of inertia, section modulus, and cross-sectional area of the slabs and beams, and eccentricity of the prestressing force shall be calculated for the trial beam depth determined above in accordance with normal structural engineering procedures.

**1816.4.4 Allowable stresses.**

The following allowable stresses are recommended:

1. **Allowable concrete flexural tensile stress:**

$$f_t = 6\sqrt{f'_c} \quad (16\text{-}5)$$

**For SI:** $f_t = 0.5\sqrt{f'_c}$

2. **Allowable concrete flexural compressive stress:**

$$f_c = 0.45f'_c \quad (16\text{-}6)$$

3. **Allowable concrete bearing stress at anchorages.**

3.1 At service load:

$$f_{bp} = 0.6f'_c\sqrt{\frac{A'_b}{A_b}} \leq f'_c \quad (16\text{-}7)$$

3.2 At transfer:

$$f_{bp} = 0.8f'_{ci}\sqrt{\frac{A'_b}{A_b} - 0.2} \leq 1.25f'_{ci} \quad (16\text{-}8)$$

4. **Allowable concrete shear stress:**

$$v_c = 1.7\sqrt{f'_c} + 0.2f_p \quad (16\text{-}9)$$

**For SI:** $v_c = 0.14\sqrt{f'_c} + 0.2f_p$

5. **Allowable stresses in prestressing steel.**

5.1 Allowable stress due to tendon jacking force:

$$f_{pj} = 0.8f_{pu} \leq 0.94f_{py} \quad (16\text{-}10)$$

5.2 Allowable stress immediately after prestress transfer:

$$f_{pi} = 0.7f_{pu} \quad (16\text{-}11)$$

**1816.4.5 Prestress losses.** Loss of prestress due to friction, elastic shortening, creep and shrinkage of the concrete, and steel relaxation shall be calculated in accordance with Section 1918.6.

**1816.4.6 Slab-subgrade friction.** The effective prestressing force in posttensioned slabs-on-ground is further reduced by the frictional resistance to movement of the slab on the subgrade during stressing as well as the frictional resistance to dimensional changes due to concrete shrinkage, creep and temperature variations. The resultant prestress force, $P_r$, is the difference between the effective prestress force and the losses due to subgrade friction:

$$P_r = P_e - SG \quad (16\text{-}12\text{-}1)$$

**For SI:** 1 pound = 4.45 kN.

where $SG$ can be conservatively taken as:

$$SG = \frac{W_{slab}}{2000}\mu \quad (16\text{-}12\text{-}2)$$

**For SI:** 1 pound = 4.45 kN.

The largest amount of prestress loss due to slab-subgrade friction occurs in the center regions of the slab. The greatest structural requirement for prestress force, however, is at the location of the maximum moment, which occurs at approximately one β-length inward from the edge of the slab. For normal construction practices, the value of the coefficient of friction μ should be taken as 0.75 for slabs on polyethylene and 1.00 for slabs cast directly on a sand base.

The maximum spacing of tendons shall not exceed that which would produce a minimum average effective prestress compression of 50 psi (0.35 MPa) after allowance for slab-subgrade friction.

**1816.4.7 Maximum applied service moments.** The maximum moment will vary, depending on the swelling mode and the slab direction being designed. For design rectangles with a ratio of long side to short side less than 1.1, the formulas for $M_L$ [Formulas (16-13-1) and (16-15)] shall be used for moments in both directions.

1. **Center lift moment.**

1.1 Long direction:

$$M_L = A_o\left[B(e_m)^{1.238} + C\right] \quad (16\text{-}13\text{-}1)$$

**For SI:** 1 ft.·kips/ft. = 4.45 kN·m/m.

**WHERE:**

$$A_o = \frac{1}{727}\left[(L)^{0.013}(S)^{0.306}(h)^{0.688}(P)^{0.534}(y_m)^{0.193}\right] \quad (16\text{-}13\text{-}2)$$

and for:

$$0 \leq e_m \leq 5 \quad B = 1, C = 0 \tag{16-13-3}$$

$$e_m > 5 \quad B = \left(\frac{y_m - 1}{3}\right) \leq 1.0 \tag{16-13-4}$$

$$C = \left[8 - \frac{P - 613}{255}\right]\left[\frac{4 - y_m}{3}\right] \geq 0 \tag{16-13-5}$$

1.2 Short direction.

For $L_L/L_S \geq 1.1$:

$$M_S = \left[\frac{58 + e_m}{60}\right] M_L \tag{16-14}$$

For **SI:** 1 ft.·kips/ft. = 4.45 kN·m/m.

For $L_L/L_S < 1.1$:

$$M_S = M_L$$

2. **Edge lift moment.**

2.1 Long direction:

$$M_L = \frac{(S)^{0.10}(he_m)^{0.78}(y_m)^{0.66}}{7.2(L)^{0.0065}(P)^{0.04}} \tag{16-15}$$

For **SI:** 1 ft.·kips/ft. = 4.45 kN·m/m.

2.2 Short direction.

For $L_L/L_S \geq 1.1$:

$$M_S = h^{0.35}\left[\frac{19 + e_m}{57.75}\right] M_L \tag{16-16}$$

For **SI:** 1 ft.·kips/ft. = 4.45 kN·m/m.

For $L_L/L_S < 1.1$:

$$M_S = M_L$$

Concrete flexural stresses produced by the applied service moments shall be calculated with the following formula:

$$f = \frac{P_r e}{A} \pm \frac{M_{LS}}{S_{t,b}} \pm \frac{P_r e}{S_{t,b}} \tag{16-17}$$

For **SI:** 1 pound per square inch = 0.0069 MPa.

The applied concrete flexural stresses $f$ shall not exceed $f_t$ in tension and $f_c$ in compression.

**1816.4.8 Cracked section considerations**. This design method limits concrete flexural tensile stresses to $6\sqrt{f'_c}$ (For **SI:** $0.5\sqrt{f'_c}$). Since the modulus of rupture of concrete is commonly taken as $f_{cr} = 7.5\sqrt{f'_c}$ (For **SI:** $f_{cr} = 0.625\sqrt{f'_c}$), slabs designed with this method will theoretically have no *flexural* cracking. Some cracking from restraint to slab shortening is inevitable in posttensioned slabs on ground, as it is in elevated posttensioned concrete members. Nevertheless, the limitation of flexural tensile stresses to a value less than the modulus of rupture justifies the use of the gross concrete cross section for calculating all section properties. This is consistent with standard practices in elevated posttensioned concrete members.

**1816.4.9 Differential deflections**. Allowable and expected differential deflections may be calculated from the formulas presented in the following sections.

**1816.4.9.1 Relative stiffness length.** $\beta$ may be calculated as follows:

$$\beta = \frac{1}{12}\sqrt[4]{\frac{E_c I}{E_s}} \tag{16-18}$$

For **SI:** $$\beta = \frac{1}{1000}\sqrt[4]{\frac{E_c I}{E_s}}$$

If the creep modulus of elasticity of the concrete $E_c$ is not known, it can be closely approximated by using half of the normal or early life concrete modulus of elasticity. If the modulus of elasticity of the clay soil $E_s$ is not known, use 1,000 psi (6.89 MPa). $I$ in Formula (16-18) is the gross moment of inertia for the entire slab cross section of width $W$, in the appropriate direction (short or long).

**1816.4.9.2 Differential deflection distance.** The differential deflection may not occur over the entire length of the slab, particularly if the slab is longer than approximately 50 feet (15.24 m). Thus, the effective distance for determining the allowable differential deflection is the smaller of the two distances, $L$ or $6\beta$, both expressed in feet (meters).

**1816.4.9.3 Allowable differential deflection, $\Delta_{allow}$ (in inches) (mm).**

1. Center lift or edge lift:

$$\Delta_{allow} = \frac{12(L \text{ or } 6\beta)}{C_\Delta} \tag{16-19}$$

For **SI:** $$\Delta_{allow} = \frac{1000(L \text{ or } 6\beta)}{C_\Delta}$$

The coefficient $C_\Delta$ is a function of the type of superstructure material and the swelling condition (center or edge lift). Sample values of $C_\Delta$ for both swelling conditions and various superstructure materials are shown in Table 18-III-GG.

**1816.4.9.4 Expected differential deflection without prestressing, $\Delta_o$ (in inches) (mm):**

1. Center lift:

$$\Delta_o = \frac{(y_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{380(h)^{1.214}} \tag{16-20}$$

For **SI:** 1 inch = 25.4 mm.

2. Edge lift:

$$\Delta_o = \frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(y_m)^{0.76}}{15.9(h)^{0.85}(P)^{0.01}} \tag{16-21}$$

For **SI:** 1 inch = 25.4 mm.

**1816.4.9.5 Deflection caused by prestressing, $\Delta_p$ (in inches) (mm).** Additional slab deflection is produced by prestressing if the prestressing force at the slab edge is applied at any point other than the *CGC*. The deflection caused by prestressing can be approximated with reasonable accuracy by assuming it is produced by a concentrated moment of $P_e e$ applied at the end of a cantilever with a span length of $\beta$. The deflection is:

$$\Delta_p = \frac{P_e e \beta^2}{2E_c I} \tag{16-22}$$

For **SI:** 1 inch = 25.4 mm.

If the tendon *CGS* is higher than the concrete *CGC* (a typical condition), $\Delta_p$ increases the edge lift deflection and decreases the

# SAMPLE TABLES FOR Ym AND Em VALUES

### TABLE 18-III-CC—DIFFERENTIAL SWELL OCCURRING AT THE PERIMETER OF A SLAB FOR AN EDGE LIFT SWELLING CONDITION IN PREDOMINANTLY MONTMORILLONITE CLAY SOIL (60 PERCENT CLAY)

| PERCENT CLAY (%) | DEPTH TO CONSTANT SUCTION (ft.) | CONSTANT SUCTION (pF) | VELOCITY OF MOISTURE FLOW (inches/month) | DIFFERENTIAL SWELL (inch) × 25.4 for mm | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Edge Distance Penetration × 304.8 for mm | | | | | | | |
| | × 304.8 for mm | | × 25.4 for mm/month | 1 ft. | 2 ft. | 3 ft. | 4 ft. | 5 ft. | 6 ft. | 7 ft. | 8 ft. |
| 60 | 3 | 3.2 | 0.5 | 0.014 | 0.027 | 0.041 | 0.054 | 0.066 | 0.079 | 0.091 | 0.104 |
| | | | 0.7 | 0.019 | 0.038 | 0.056 | 0.074 | 0.091 | 0.109 | 0.125 | 0.142 |
| | | 3.4 | 0.5 | 0.029 | 0.057 | 0.084 | 0.110 | 0.135 | 0.160 | 0.183 | 0.206 |
| | | | 0.7 | 0.041 | 0.079 | 0.116 | 0.151 | 0.184 | 0.216 | 0.247 | 0.277 |
| | | 3.6 | 0.5 | 0.071 | 0.136 | 0.195 | 0.251 | 0.303 | 0.352 | 0.399 | 0.433 |
| | | | 0.7 | 0.098 | 0.186 | 0.264 | 0.336 | 0.402 | 0.463 | 0.521 | 0.575 |
| | | 3.8 | 0.5 | 0.169 | 0.309 | 0.428 | 0.533 | 0.627 | 0.712 | 0.790 | 0.863 |
| | | | 0.7 | 0.233 | 0.413 | 0.562 | 0.690 | 0.802 | 0.903 | 0.994 | 1.077 |
| | 5 | 3.2 | 0.5 | 0.030 | 0.060 | 0.090 | 0.118 | 0.146 | 0.174 | 0.202 | 0.229 |
| | | | 0.7 | 0.042 | 0.083 | 0.124 | 0.163 | 0.202 | 0.240 | 0.278 | 0.314 |
| | | 3.4 | 0.5 | 0.065 | 0.127 | 0.188 | 0.246 | 0.303 | 0.359 | 0.413 | 0.465 |
| | | | 0.7 | 0.090 | 0.177 | 0.259 | 0.339 | 0.415 | 0.488 | 0.559 | 0.628 |
| | | 3.6 | 0.5 | 0.160 | 0.308 | 0.446 | 0.575 | 0.697 | 0.812 | 0.921 | 1.025 |
| | | | 0.7 | 0.224 | 0.425 | 0.607 | 0.775 | 0.931 | 1.077 | 1.214 | 1.343 |
| | | 3.8 | 0.5 | 0.395 | 0.724 | 1.009 | 1.261 | 1.488 | 1.695 | 1.886 | 2.063 |
| | | | 0.7 | 0.551 | 0.984 | 1.345 | 1.657 | 1.933 | 2.181 | 2.407 | 2.614 |
| | 7 | 3.2 | 0.5 | 0.052 | 0.104 | 0.155 | 0.205 | 0.254 | 0.303 | 0.351 | 0.398 |
| | | | 0.7 | 0.073 | 0.145 | 0.215 | 0.284 | 0.352 | 0.419 | 0.484 | 0.548 |
| | | 3.4 | 0.5 | 0.113 | 0.223 | 0.330 | 0.434 | 0.535 | 0.633 | 0.729 | 0.823 |
| | | | 0.7 | 0.159 | 0.311 | 0.458 | 0.598 | 0.734 | 0.865 | 0.992 | 1.115 |
| | | 3.6 | 0.5 | 0.286 | 0.551 | 0.799 | 1.031 | 1.251 | 1.459 | 1.658 | 1.847 |
| | | | 0.7 | 0.402 | 0.764 | 1.095 | 1.400 | 1.684 | 1.950 | 2.200 | 2.437 |

### TABLE 18-III-DD—DIFFERENTIAL SWELL OCCURRING AT THE PERIMETER OF A SLAB FOR AN EDGE LIFT SWELLING CONDITION IN PREDOMINANTLY MONTMORILLONITE CLAY SOIL (70 PERCENT CLAY)

| PERCENT CLAY (%) | DEPTH TO CONSTANT SUCTION (ft.) | CONSTANT SUCTION (pF) | VELOCITY OF MOISTURE FLOW (inches/month) | DIFFERENTIAL SWELL (inch) × 25.4 for mm | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Edge Distance Penetration × 304.8 for mm | | | | | | | |
| | × 304.8 for mm | | × 25.4 for mm/month | 1 ft. | 2 ft. | 3 ft. | 4 ft. | 5 ft. | 6 ft. | 7 ft. | 8 ft. |
| 70 | 3 | 3.2 | 0.5 | 0.016 | 0.032 | 0.048 | 0.064 | 0.079 | 0.094 | 0.109 | 0.123 |
| | | | 0.7 | 0.023 | 0.045 | 0.067 | 0.088 | 0.109 | 0.129 | 0.149 | 0.169 |
| | | 3.4 | 0.5 | 0.035 | 0.068 | 0.100 | 0.131 | 0.161 | 0.190 | 0.219 | 0.246 |
| | | | 0.7 | 0.048 | 0.094 | 0.138 | 0.179 | 0.219 | 0.258 | 0.294 | 0.330 |
| | | 3.6 | 0.5 | 0.084 | 0.162 | 0.233 | 0.299 | 0.361 | 0.420 | 0.475 | 0.528 |
| | | | 0.7 | 0.117 | 0.221 | 0.314 | 0.400 | 0.479 | 0.552 | 0.620 | 0.684 |
| | | 3.8 | 0.5 | 0.202 | 0.368 | 0.510 | 0.635 | 0.747 | 0.849 | 0.942 | 1.028 |
| | | | 0.7 | 0.277 | 0.492 | 0.669 | 0.822 | 0.955 | 1.075 | 1.184 | 1.284 |
| | 5 | 3.2 | 0.5 | 0.036 | 0.071 | 0.106 | 0.140 | 0.174 | 0.207 | 0.240 | 0.273 |
| | | | 0.7 | 0.050 | 0.099 | 0.147 | 0.195 | 0.241 | 0.286 | 0.331 | 0.374 |
| | | 3.4 | 0.5 | 0.077 | 0.151 | 0.223 | 0.293 | 0.361 | 0.427 | 0.492 | 0.554 |
| | | | 0.7 | 0.108 | 0.210 | 0.309 | 0.403 | 0.494 | 0.582 | 0.666 | 0.748 |
| | | 3.6 | 0.5 | 0.191 | 0.367 | 0.531 | 0.685 | 0.830 | 0.967 | 1.097 | 1.221 |
| | | | 0.7 | 0.267 | 0.506 | 0.724 | 0.924 | 1.110 | 1.283 | 1.446 | 1.600 |
| | | 3.8 | 0.5 | 0.470 | 0.862 | 1.262 | 1.502 | 1.773 | 2.020 | 2.247 | 2.458 |
| | | | 0.7 | 0.656 | 1.172 | 1.603 | 1.974 | 2.303 | 2.598 | 2.867 | 3.114 |
| | 7 | 3.2 | 0.5 | 0.062 | 0.124 | 0.184 | 0.244 | 0.303 | 0.361 | 0.418 | 0.475 |
| | | | 0.7 | 0.087 | 0.173 | 0.256 | 0.339 | 0.419 | 0.499 | 0.577 | 0.653 |
| | | 3.4 | 0.5 | 0.135 | 0.266 | 0.393 | 0.517 | 0.637 | 0.754 | 0.869 | 0.980 |
| | | | 0.7 | 0.189 | 0.371 | 0.545 | 0.713 | 0.875 | 1.031 | 1.182 | 1.329 |
| | | 3.6 | 0.5 | 0.341 | 0.656 | 0.951 | 1.229 | 1.490 | 1.739 | 1.975 | 2.200 |
| | | | 0.7 | 0.479 | 0.910 | 1.304 | 1.668 | 2.006 | 2.323 | 2.621 | 2.903 |

## TABLE 18-III-C—DIFFERENTIAL SWELL OCCURRING AT THE PERIMETER OF A SLAB FOR A CENTER LIFT SWELLING CONDITION IN PREDOMINANTLY KAOLINITE CLAY SOIL (50 PERCENT CLAY)

| PERCENT CLAY (%) | DEPTH TO CONSTANT SUCTION (ft.) × 304.8 for mm | CONSTANT SUCTION (pF) | VELOCITY OF MOISTURE FLOW (inches/month) × 25.4 for mm/month | DIFFERENTIAL SWELL (inch) × 25.4 for mm — Edge Distance Penetration × 304.8 for mm: 1 ft. | 2 ft. | 3 ft. | 4 ft. | 5 ft. | 6 ft. | 7 ft. | 8 ft. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 3 | 3.2 | 0.5 | 0.004 | 0.009 | 0.014 | 0.019 | 0.025 | 0.030 | 0.036 | 0.042 |
| | | | 0.7 | 0.006 | 0.013 | 0.020 | 0.028 | 0.036 | 0.044 | 0.053 | 0.063 |
| | | 3.4 | 0.5 | 0.009 | 0.020 | 0.031 | 0.043 | 0.057 | 0.073 | 0.091 | 0.113 |
| | | | 0.7 | 0.013 | 0.028 | 0.044 | 0.064 | 0.086 | 0.115 | 0.153 | 0.207 |
| | | 3.6 | 0.5 | 0.022 | 0.049 | 0.083 | 0.127 | 0.196 | 0.321 | 0.620 | 1.480 |
| | | | 0.7 | 0.031 | 0.070 | 0.124 | 0.210 | 0.380 | 0.818 | 2.103 | 5.926 |
| | 5 | 3.2 | 0.5 | 0.011 | 0.022 | 0.034 | 0.046 | 0.059 | 0.072 | 0.086 | 0.100 |
| | | | 0.7 | 0.015 | 0.031 | 0.048 | 0.066 | 0.084 | 0.104 | 0.125 | 0.148 |
| | | 3.4 | 0.5 | 0.023 | 0.047 | 0.073 | 0.101 | 0.133 | 0.168 | 0.209 | 0.258 |
| | | | 0.7 | 0.031 | 0.066 | 0.105 | 0.148 | 0.200 | 0.264 | 0.347 | 0.469 |
| | | 3.6 | 0.5 | 0.051 | 0.113 | 0.188 | 0.288 | 0.434 | 0.694 | 1.303 | 3.018 |
| | | | 0.7 | 0.071 | 0.160 | 0.281 | 0.465 | 0.817 | 1.696 | 4.196 | — |
| | 7 | 3.2 | 0.5 | 0.021 | 0.042 | 0.064 | 0.086 | 0.110 | 0.134 | 0.159 | 0.186 |
| | | | 0.7 | 0.028 | 0.058 | 0.090 | 0.122 | 0.157 | 0.194 | 0.233 | 0.276 |
| | | 3.4 | 0.5 | 0.041 | 0.085 | 0.133 | 0.185 | 0.243 | 0.308 | 0.383 | 0.472 |
| | | | 0.7 | 0.057 | 0.120 | 0.191 | 0.272 | 0.366 | 0.483 | 0.636 | 0.864 |
| | | 3.58 | 0.5 | 0.086 | 0.186 | 0.306 | 0.457 | 0.666 | 1.001 | 1.690 | 3.499 |
| | | | 0.7 | 0.119 | 0.263 | 0.452 | 0.723 | 1.194 | 2.260 | 5.172 | — |

## TABLE 18-III-D—DIFFERENTIAL SWELL OCCURRING AT THE PERIMETER OF A SLAB FOR A CENTER LIFT SWELLING CONDITION IN PREDOMINANTLY KAOLINITE CLAY SOIL (60 PERCENT CLAY)

| PERCENT CLAY (%) | DEPTH TO CONSTANT SUCTION (ft.) × 304.8 for mm | CONSTANT SUCTION (pF) | VELOCITY OF MOISTURE FLOW (inches/month) × 25.4 for mm/month | DIFFERENTIAL SWELL (inch) × 25.4 for mm — Edge Distance Penetration × 304.8 for mm: 1 ft. | 2 ft. | 3 ft. | 4 ft. | 5 ft. | 6 ft. | 7 ft. | 8 ft. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 60 | 3 | 3.2 | 0.5 | 0.006 | 0.012 | 0.018 | 0.024 | 0.030 | 0.037 | 0.044 | 0.052 |
| | | | 0.7 | 0.008 | 0.017 | 0.025 | 0.034 | 0.044 | 0.054 | 0.065 | 0.077 |
| | | 3.4 | 0.5 | 0.012 | 0.024 | 0.038 | 0.053 | 0.069 | 0.088 | 0.110 | 0.136 |
| | | | 0.7 | 0.016 | 0.033 | 0.053 | 0.077 | 0.104 | 0.138 | 0.184 | 0.249 |
| | | 3.6 | 0.5 | 0.026 | 0.059 | 0.099 | 0.154 | 0.236 | 0.386 | 0.745 | 1.779 |
| | | | 0.7 | 0.036 | 0.083 | 0.149 | 0.252 | 0.455 | 0.983 | 2.527 | 7.124 |
| | 5 | 3.2 | 0.5 | 0.013 | 0.027 | 0.041 | 0.056 | 0.071 | 0.087 | 0.103 | 0.120 |
| | | | 0.7 | 0.019 | 0.038 | 0.058 | 0.080 | 0.102 | 0.126 | 0.151 | 0.178 |
| | | 3.4 | 0.5 | 0.028 | 0.056 | 0.087 | 0.122 | 0.160 | 0.202 | 0.252 | 0.310 |
| | | | 0.7 | 0.037 | 0.078 | 0.125 | 0.177 | 0.240 | 0.316 | 0.417 | 0.564 |
| | | 3.6 | 0.5 | 0.062 | 0.135 | 0.226 | 0.345 | 0.521 | 0.834 | 1.566 | 3.628 |
| | | | 0.7 | 0.086 | 0.193 | 0.337 | 0.559 | 0.982 | 2.039 | 5.049 | — |
| | 7 | 3.2 | 0.5 | 0.025 | 0.050 | 0.077 | 0.104 | 0.132 | 0.161 | 0.192 | 0.224 |
| | | | 0.7 | 0.034 | 0.070 | 0.108 | 0.147 | 0.189 | 0.233 | 0.281 | 0.332 |
| | | 3.4 | 0.5 | 0.050 | 0.103 | 0.160 | 0.223 | 0.292 | 0.371 | 0.461 | 0.568 |
| | | | 0.7 | 0.069 | 0.144 | 0.229 | 0.326 | 0.440 | 0.580 | 0.765 | 1.038 |
| | | 3.58 | 0.5 | 0.103 | 0.223 | 0.367 | 0.549 | 0.800 | 1.203 | 2.031 | 4.205 |
| | | | 0.7 | 0.142 | 0.316 | 0.543 | 0.870 | 1.436 | 2.717 | 6.217 | — |

# CHARTS AND GRAPHS

1. **Climate Rating Chart.**
2. **Thornwaite Moisture Indices for Texas.**
3. **New Slope Coefficient Criteria Incorporated in the 1997 UBC.**
4. **New Deflection Criteria Set for By PTI and Incorporated in the 1997 UBC.** (Note: These values when placed in the new PTI Manual were to be instituted as guidelines and not absolute control values.)

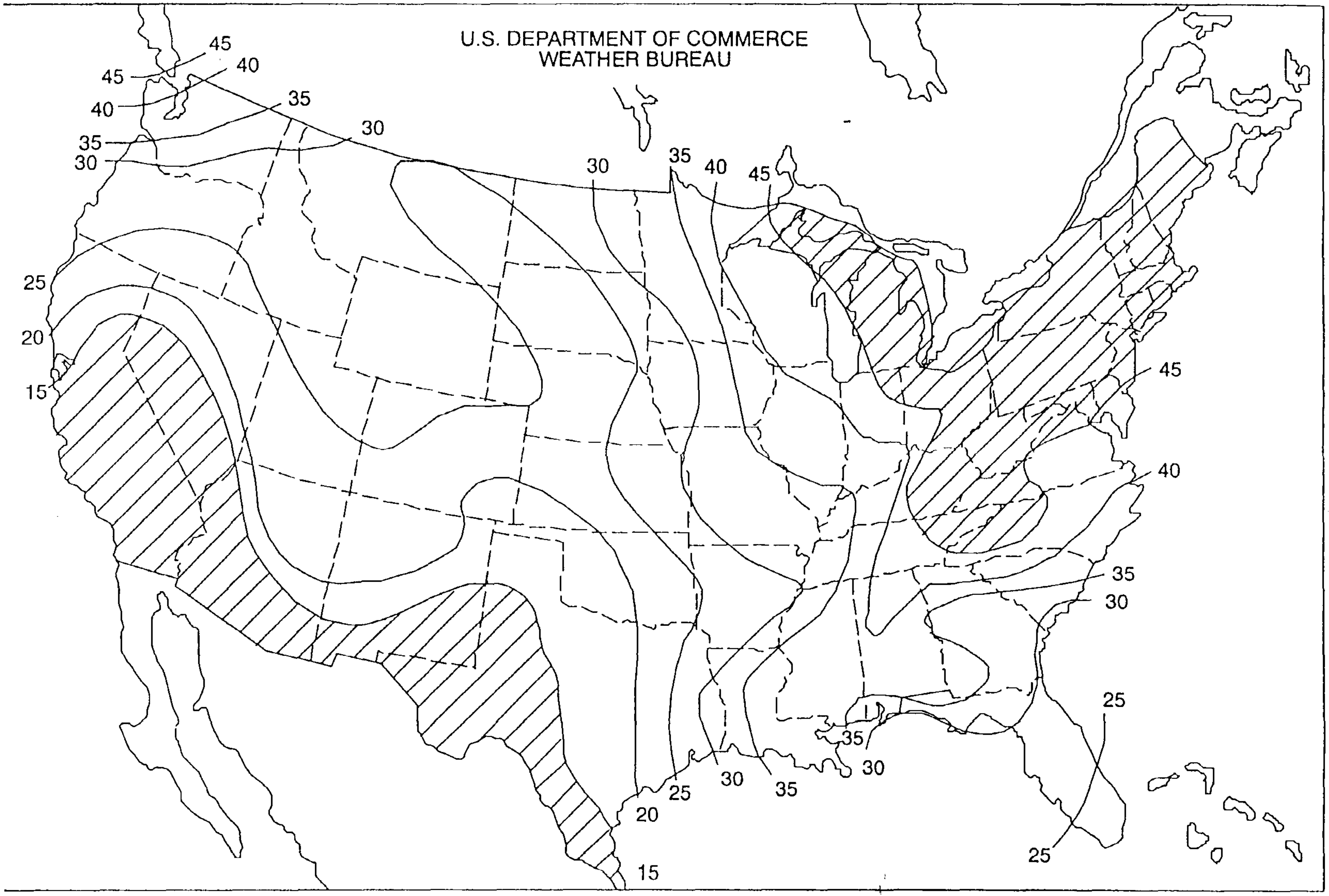



**FIGURE 18-III-4—CLIMATIC RATING ($C_W$) CHART**

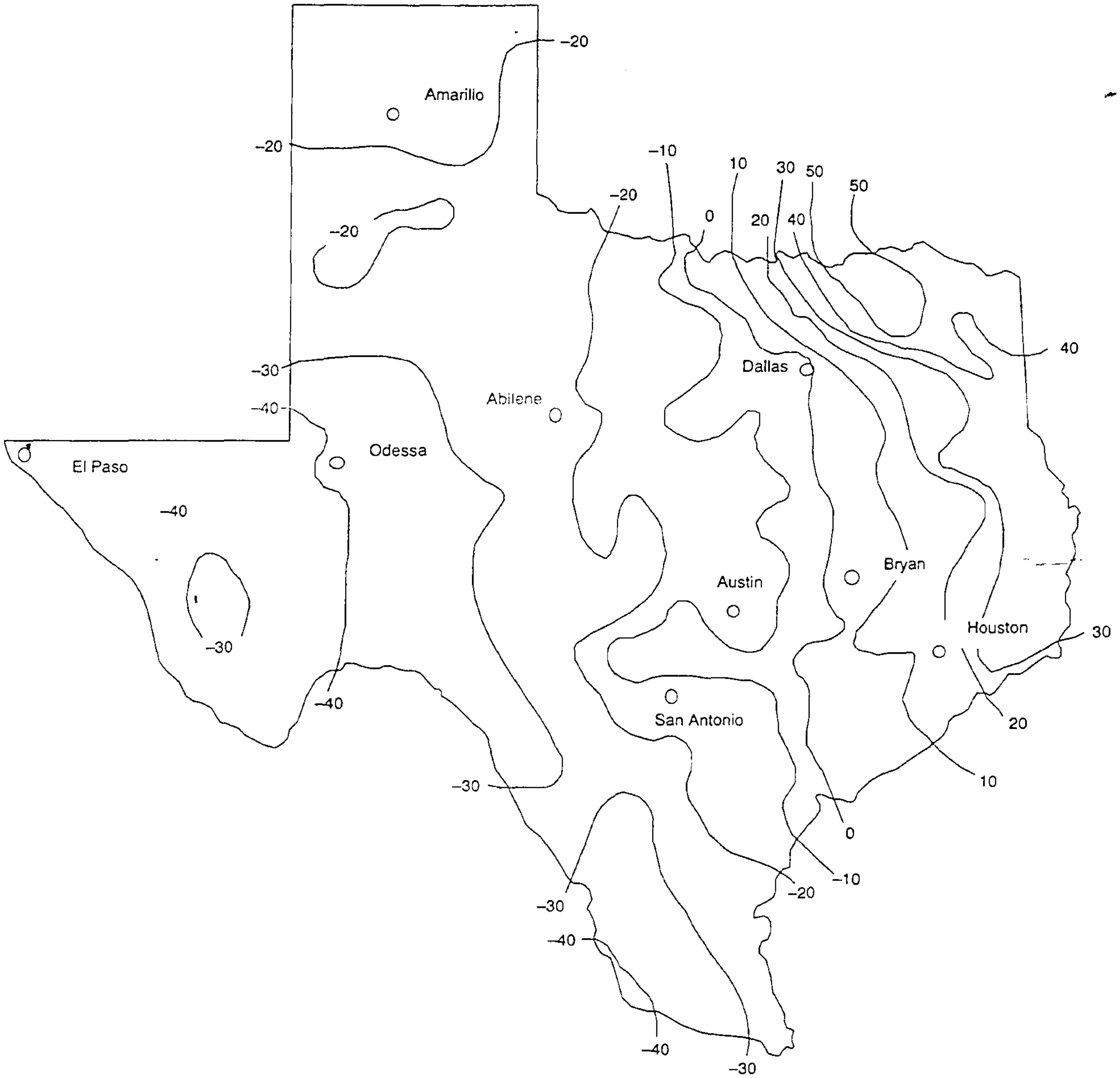



FIGURE 18-III-13-1—THORNTHWAITE MOISTURE INDEX DISTRIBUTION FOR TEXAS
(20-YEAR AVERAGE, 1955-1974)

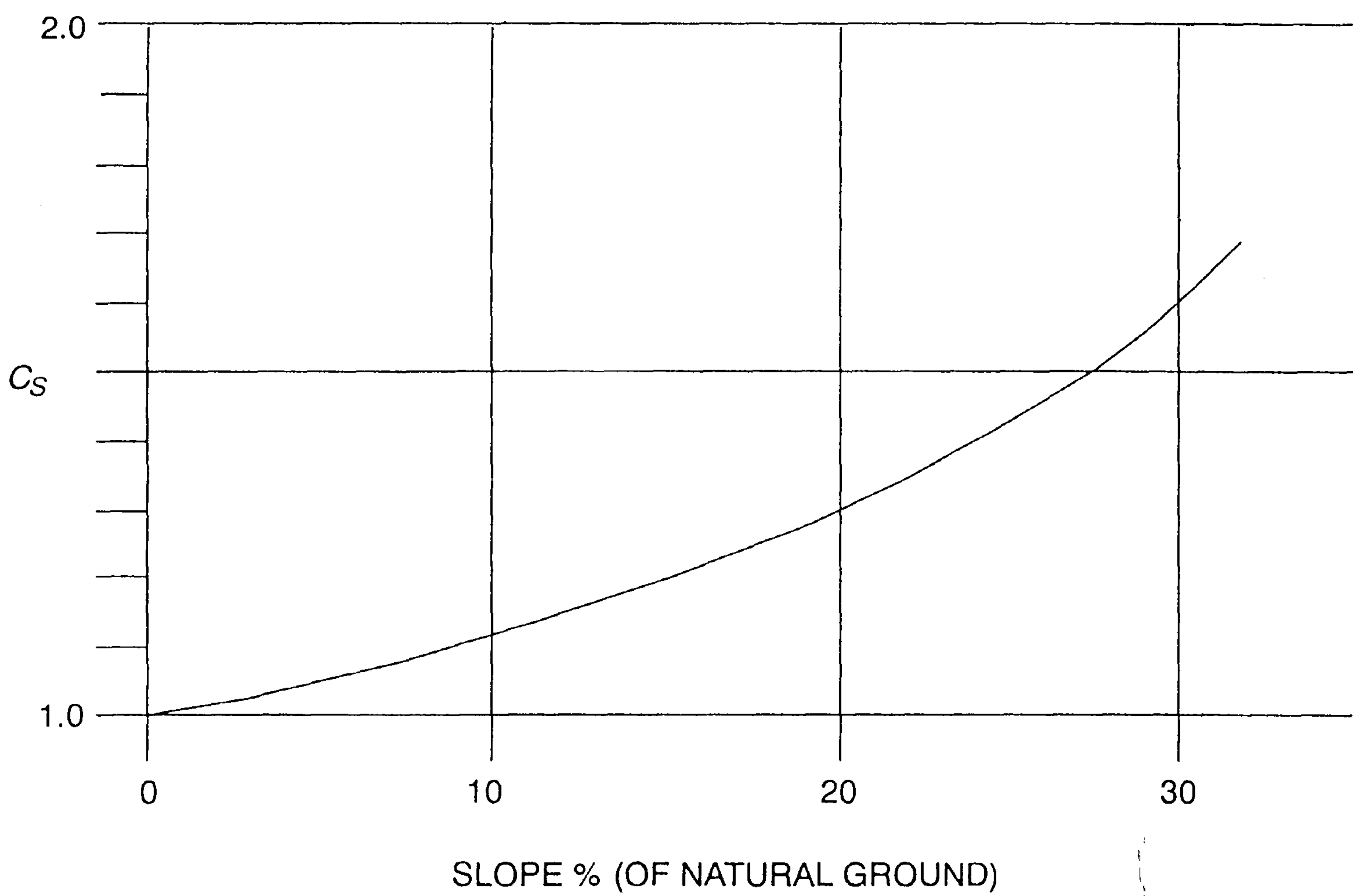



**FIGURE 18-III-3—SLOPE OF NATURAL GROUND VERSUS SLOPE CORRECTION COEFFICIENT**

## TABLE 18-III-GG—SAMPLE VALUES $C_\Delta$

| MATERIAL | CENTER LIFT | EDGE LIFT |
|---|---|---|
| Wood Frame | 240 | 480 |
| Stucco or Plaster | 360 | 720 |
| Brick Veneer | 480 | 960 |
| Concrete Masonry Units | 960 | 1,920 |
| Prefab Roof Trusses[1] | 1,000 | 2,000 |

[1]Trusses that clearspan the full length or width of the foundation from edge to edge.

# THE PTI'S EXAMPLE OF BENDING MOMENT CURVE ALONG THE PLANAR AXIS OF A RECTANGULAR FOUNDATION

## 5.0 SOIL-STRUCTURE INTERACTION ANALYSIS

### 5.1 General

An extensive computer study of the soil-structure interaction occurring between a slab-on-ground and the underlying swelling soil was conducted at Texas A & M University. The purpose of the study was to determine the relationship between the various design parameters and their effect on the basic design quantities of moment, shear, and differential deflection as each parameter was varied.[108] The results of this PTI-sponsored research are presented in Appendices A.9-A.10. Some of the observations resulting from the study regarding the basic design parameters of moment, shear, and deflection are presented in this Chapter as introductory to the rational design procedure that resulted from the study of this problem.

The results of an extensive review of technical literature concerning the magnitude of slab-subgrade friction effects are also presented in Section 5.5.

### 5.2 Bending Moment

As a result of the computer study the following observations can be made concerning the magnitude and distribution of bending moments:

A. The magnitude of the moment in either the long or the short direction typically varied as shown in Figure 5.1. This variation was similar for both edge lift and center lift conditions.

B. The typical distribution of moment, as shown in the contours of Figure 5.2, indicates a moment "peak" occurring, with moment contours sloping toward or away from the location of the maximum moment. This distribution pattern was typical for both swelling modes.

C. The maximum moment does not occur at the point of actual soil-slab separation but at some distance further toward the interior. The location of the maximum moment can be closely estimated by $\beta$, a length which depends upon the relative stiffness of the soil and the stiffened slab.

D. The moment increases rapidly from the edge of the slab until it reaches a maximum at approximately $\beta$. The magnitude then begins to reduce towards the midpoint of the slab. The amount of this reduction is dependent upon the slab length for slabs approximately 48 feet or less. For longer slabs, increased length does not offer further moment reduction in the mid-region of the overall slab length. Figure 5.3 illustrates the effect of slab length on moment reduction.

E. In all cases of edge lift studied, and in most cases of center lift, the magnitude of the moment in the short direction was either approximately equal to, or larger than, the moment in the long direction. Additionally, with other variables held constant, deeper stiffening beams produced an increase in bending moment. This observation confirms the maxim that "stiffness attracts moment".

F. Prestressed slabs with beam depths varying from 18 inches to 30 inches were loaded by computer model until tensile failure of the concrete occurred. Perimeter loading in the center lift model was increased far beyond the cracking load, and loading and soil swelling conditions in the edge lift model were gradually increased far beyond the tensile cracking point in order to study the post-cracking response of the slab-soil system. For both loading cases, the response of the slab up to the cracking moment was elastic. At the point of cracking, a substantial deflection took place for center lift loading, and a small deflection occurred for edge lift loading. As a result of these deflections, the slab experienced addi-

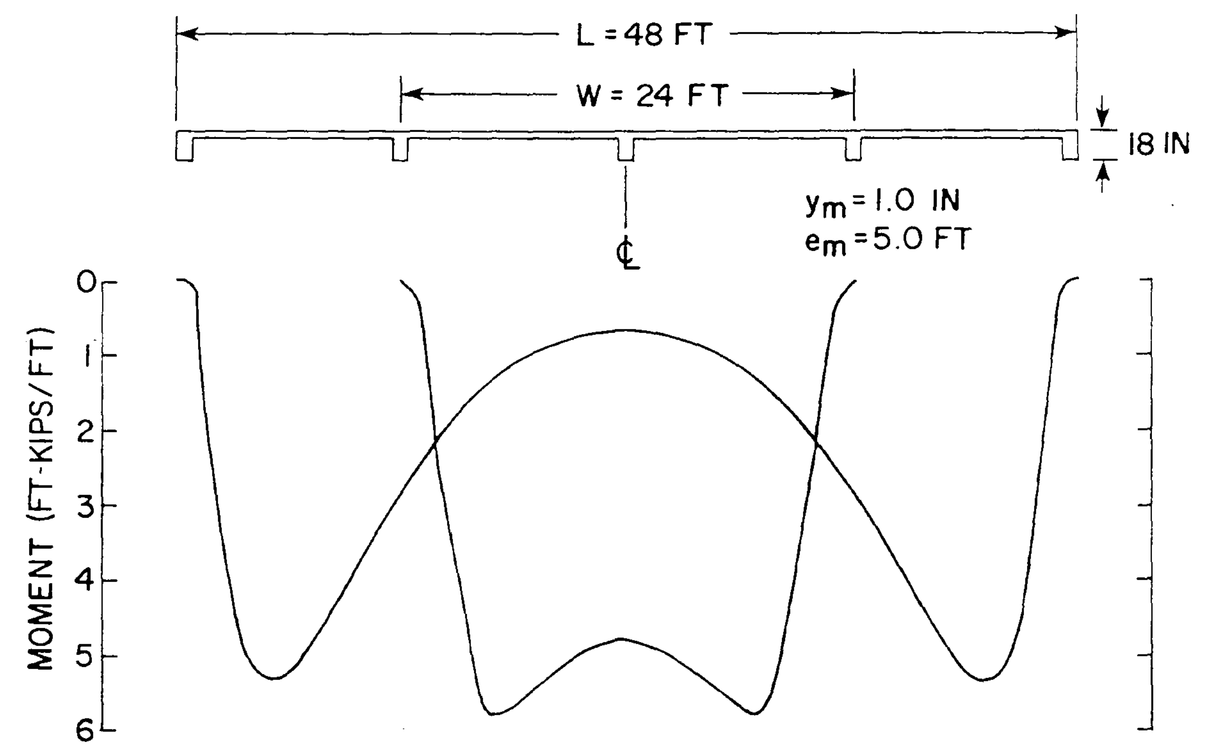



**Fig. 5.1 Typical variation of moment along the longitudinal and transverse axes of a rectangular slab (108)**

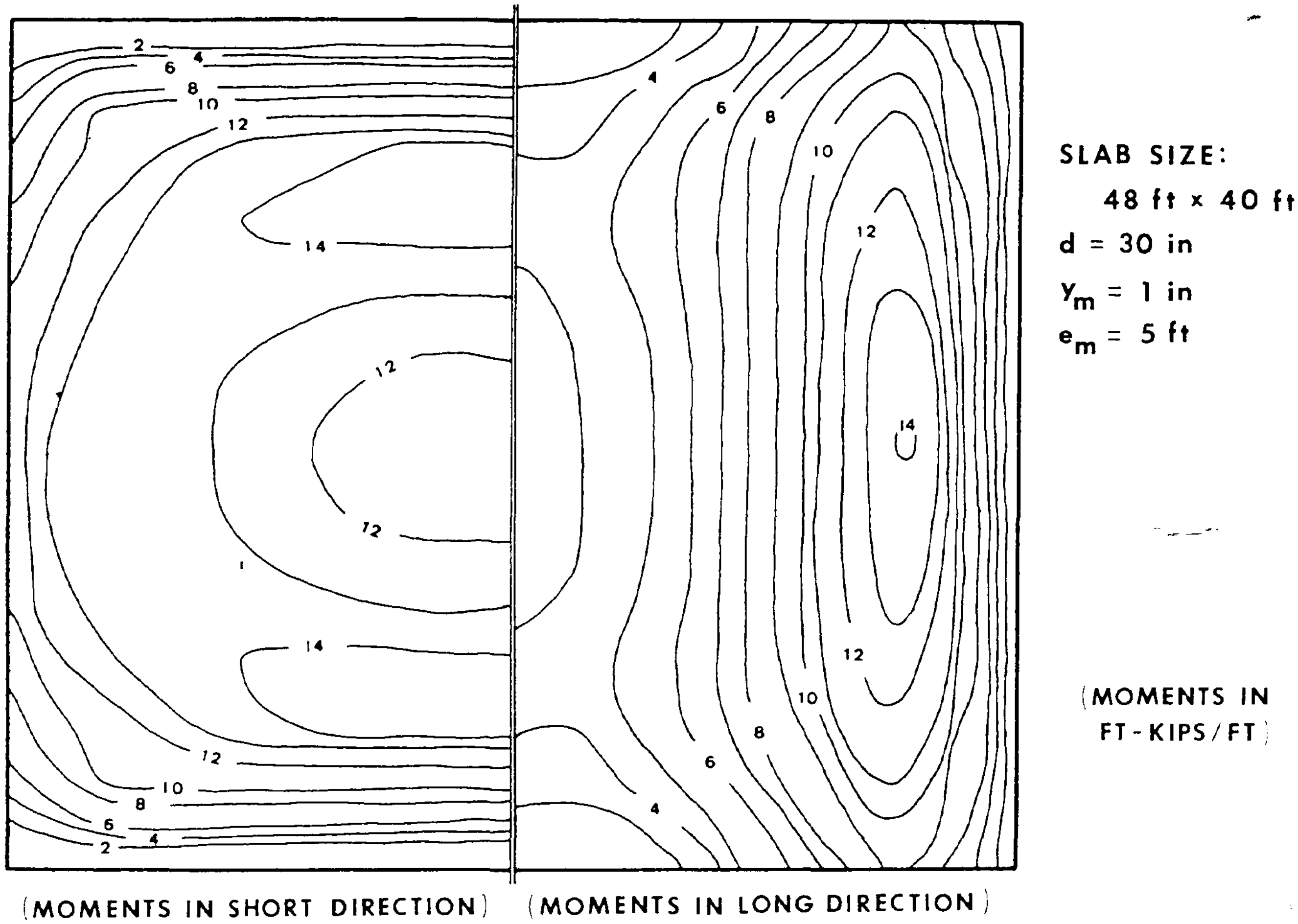



Fig. 5.2 Typical distribution of center lift bending moment over surface of slab (108)

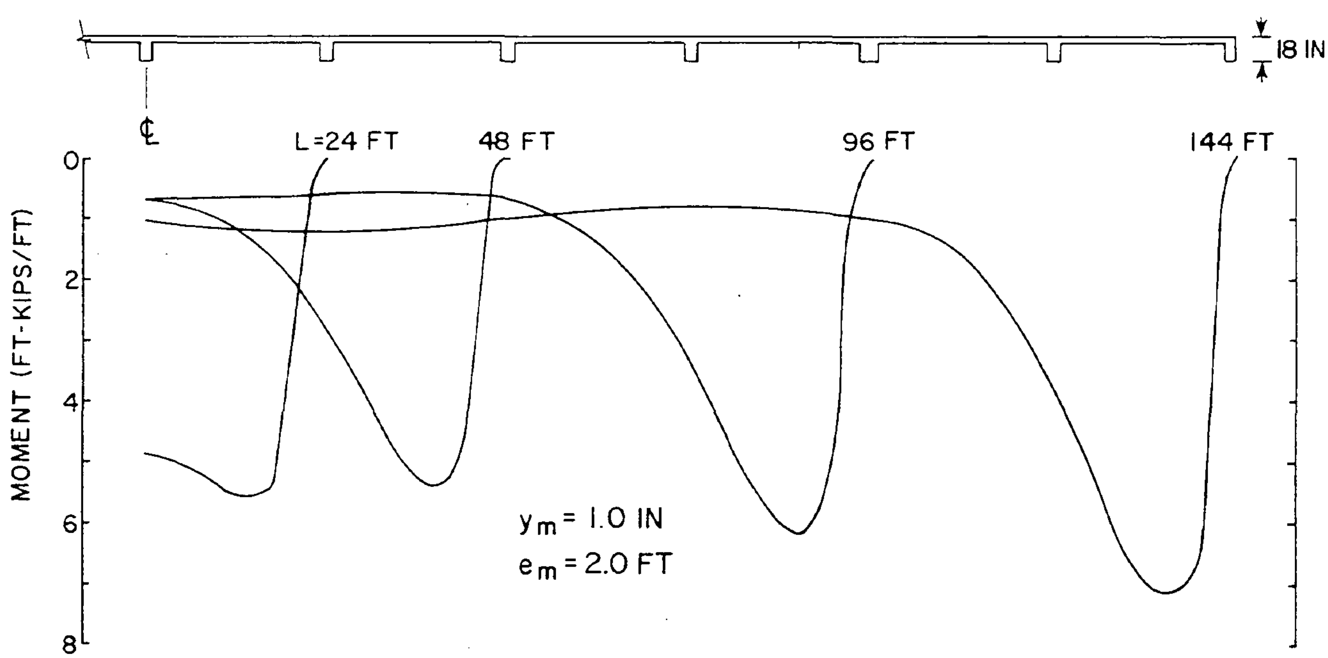


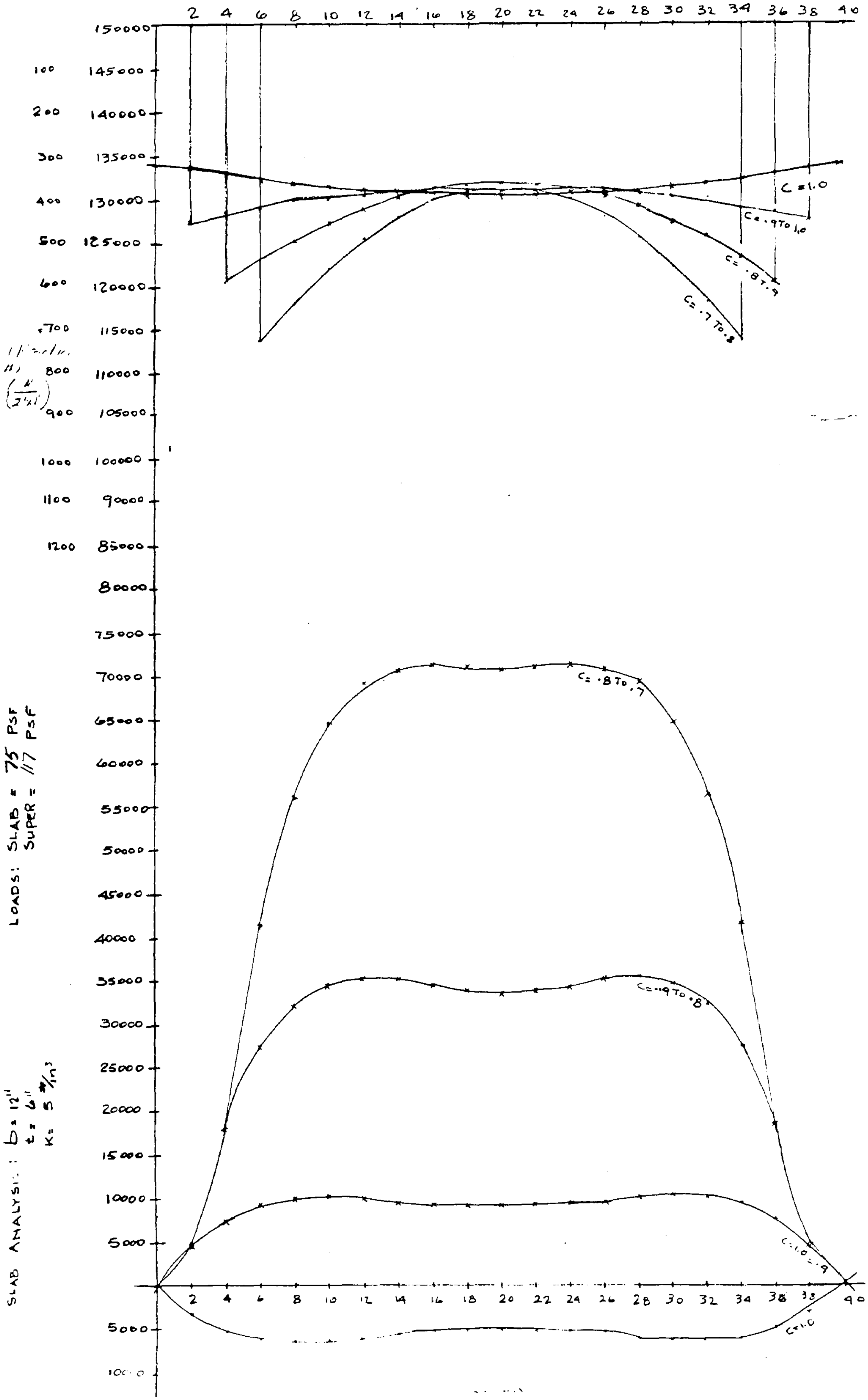

Fig. 5.3 Typical variation of moment along the longitudinal axis as slab length increases (108)

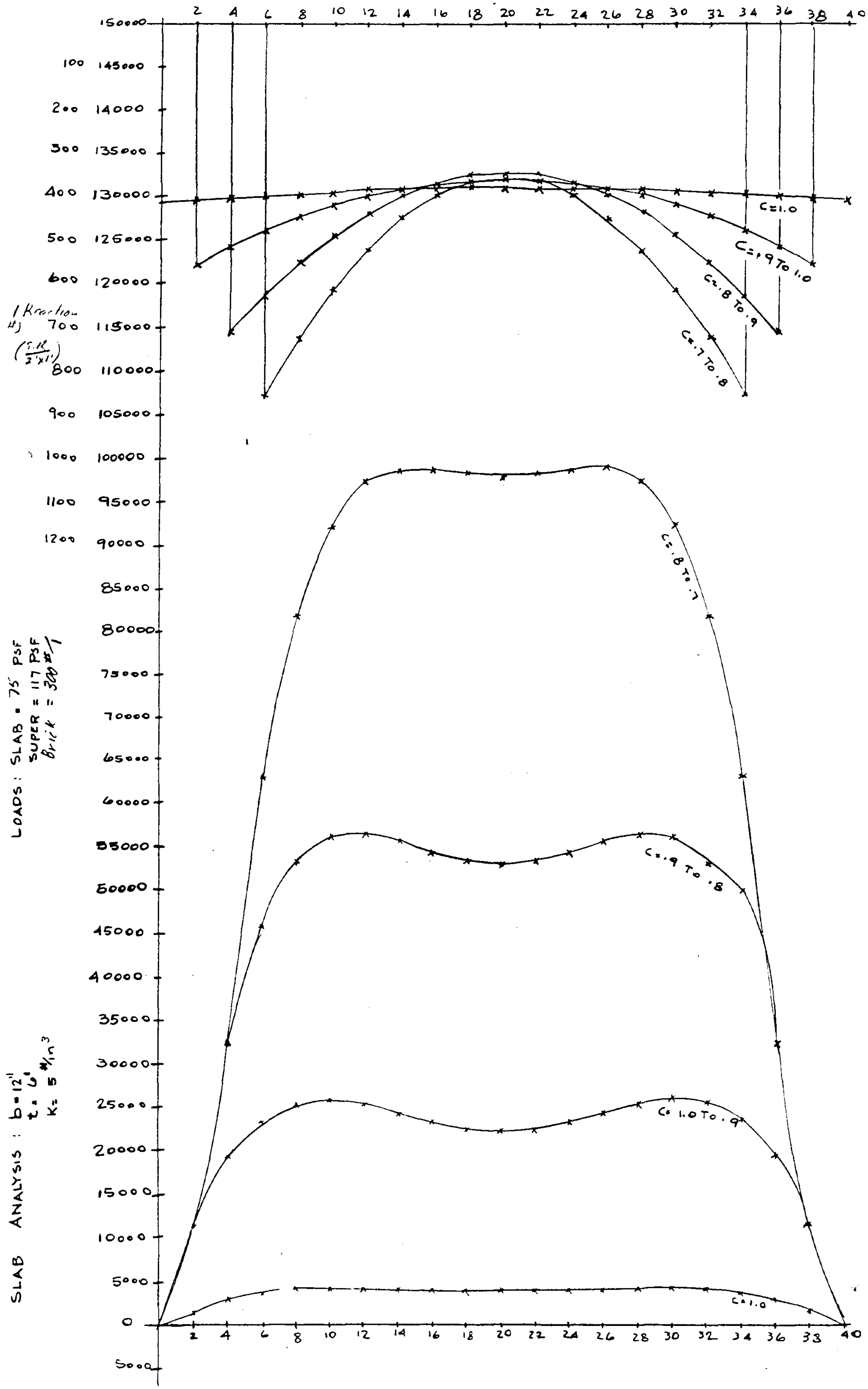
GRAPHS SHOWING REACTIONS AND BENDING MOMENT CURVES FOR A LINEAR-PLANE SLAB SECTION 40' IN LENGTH USING AN AVERAGE THICKNESS OF 6" WITH VARYING LOAD CONDITIONS.

WINKLER SPRINGS WERE USED TO ANALYZE A FOUNDATION SECTION SUPPORTED ON AN ELASTIC SOIL SUPPORT SYSTEM.

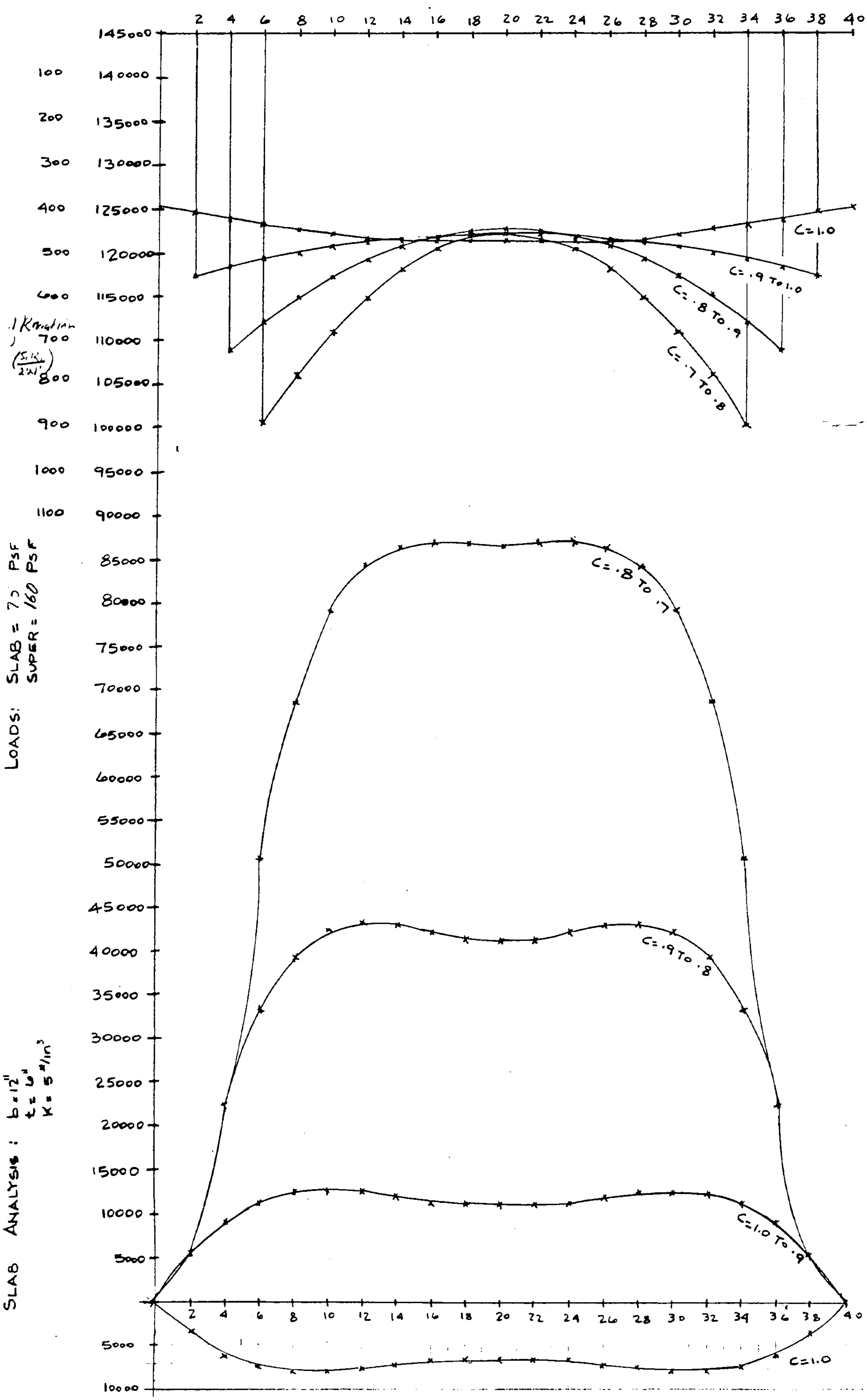
ITEMS WHICH WERE VARIED:

1. LOAD CONDITIONS.
2. THE MODULUS OF SUBGRADE REACTION.
3. THE SUPPORT INDEX COEFFICIENT.
4. THE FOUNDATION RIGIDITY.

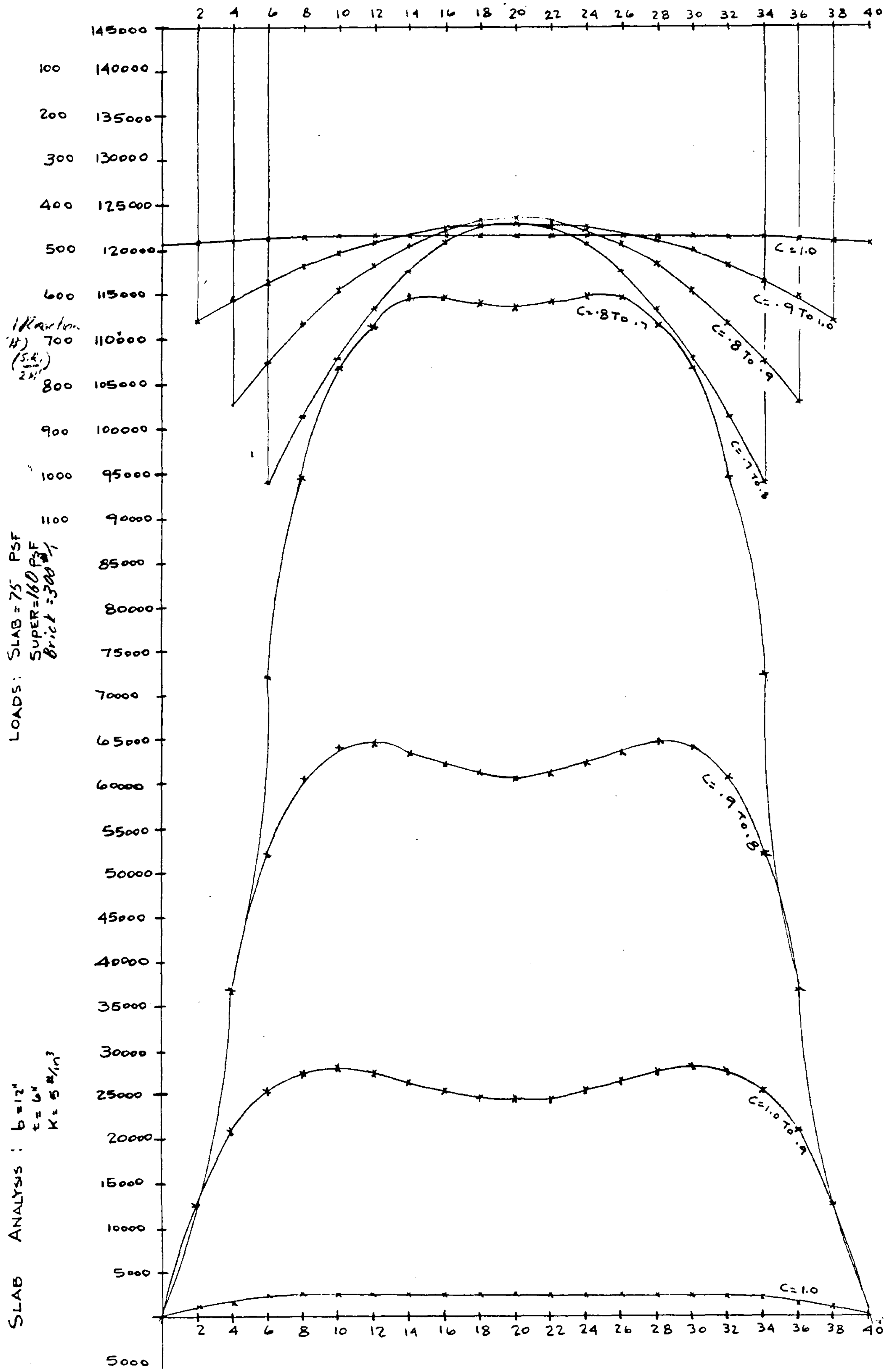

SLAB ANALYSIS: b = 12" t = 6" K = 5 #/in^3
LOADS: SLAB = 75 PSF SUPER = 117 PSF
2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
150000
100 145000
200 140000
300 135000
400 130000
500 125000
600 120000
700 115000
800 110000
900 105000
1000 100000
1100 90000
1200 85000
80000
75000
70000
65000
60000
55000
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
5000
C = 1.0
C = .9 TO 1.0
C = .8 TO .9
C = .7 TO .8
C = .8 TO .7
C = .9 TO .8
C = 1.0 - .9
C = 1.0



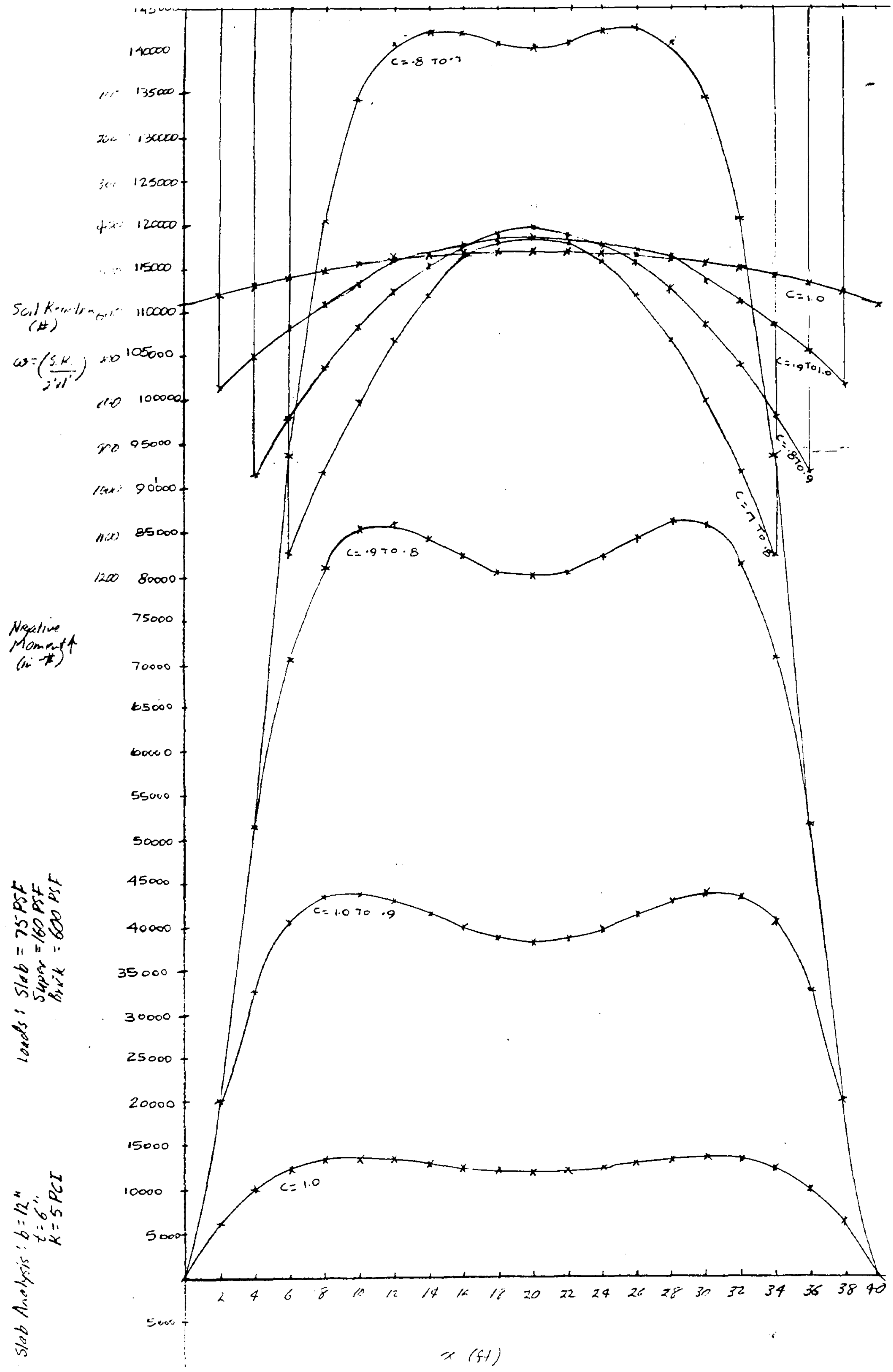
SLAB ANALYSIS : b = 12" t = 6" K = 5 #/in³
LOADS : SLAB = 75 PSF SUPER = 117 PSF Brick = 300 #/'
150000
145000
14000
135000
130000
126000
120000
115000
110000
105000
100000
95000
90000
85000
80000
75000
70000
65000
60000
55000
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
5000
100
200
300
400
500
600
700
800
900
1000
1100
1200
Reaction
C=1.0
C=.9 To 1.0
C=.8 To .9
C=.7 To .8
C=.8 To .7
C=.9 To .8
C=1.0 To .9
C=1.0



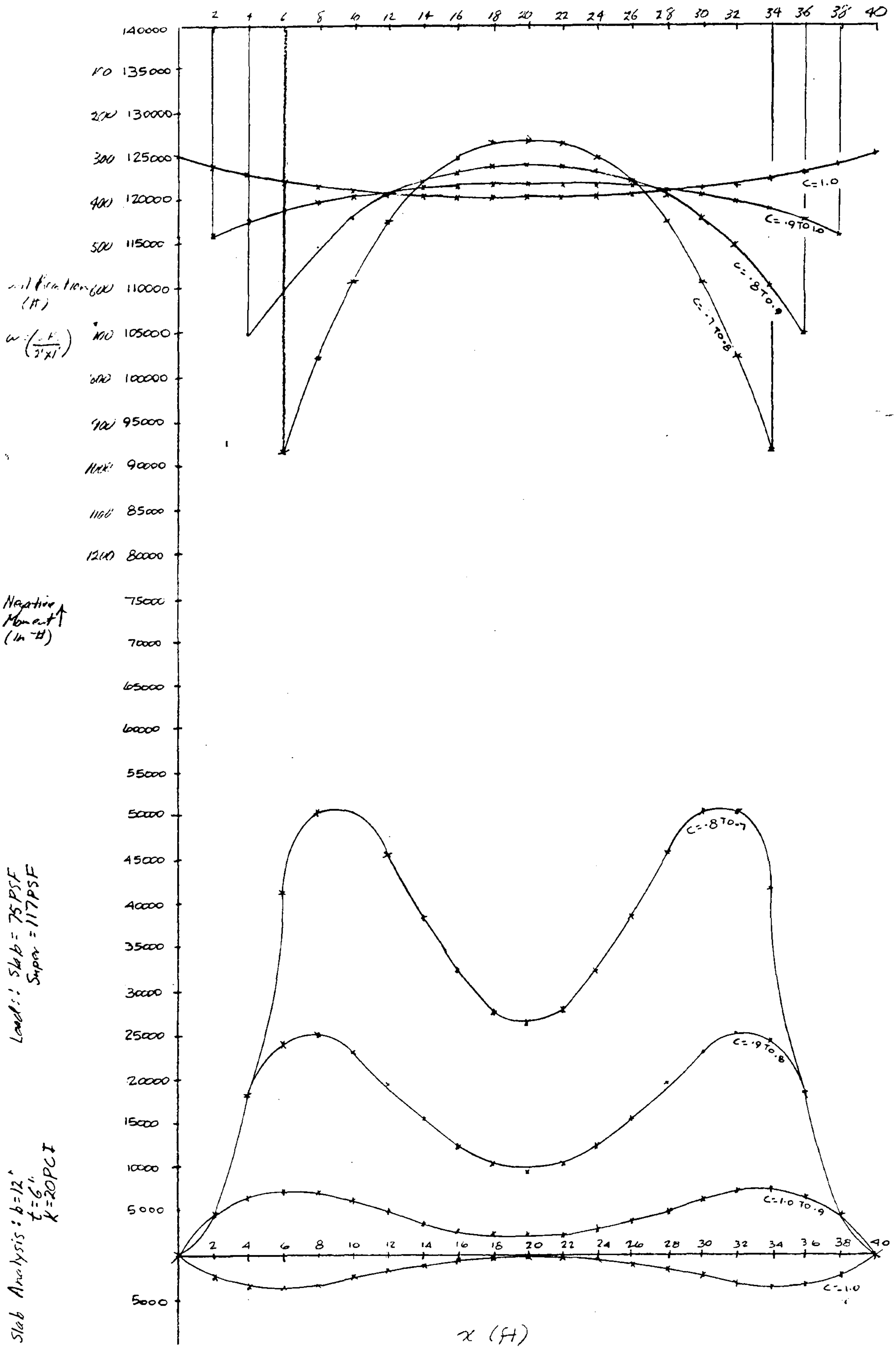
SLAB ANALYSIS: b = 12"
t = 6"
K = 5 #/in³
LOADS: SLAB = 75 PSF
SUPER = 160 PSF
C=1.0
C=.9 To 1.0
C=.8 To .9
C=.7 To .8
C=.8 To .7
C=.9 To .8
C=1.0 To .9
C=1.0
145000
140000
135000
130000
125000
120000
115000
110000
105000
100000
95000
90000
85000
80000
75000
70000
65000
60000
55000
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
100
200
300
400
500
600
700
800
900
1000
1100
2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40



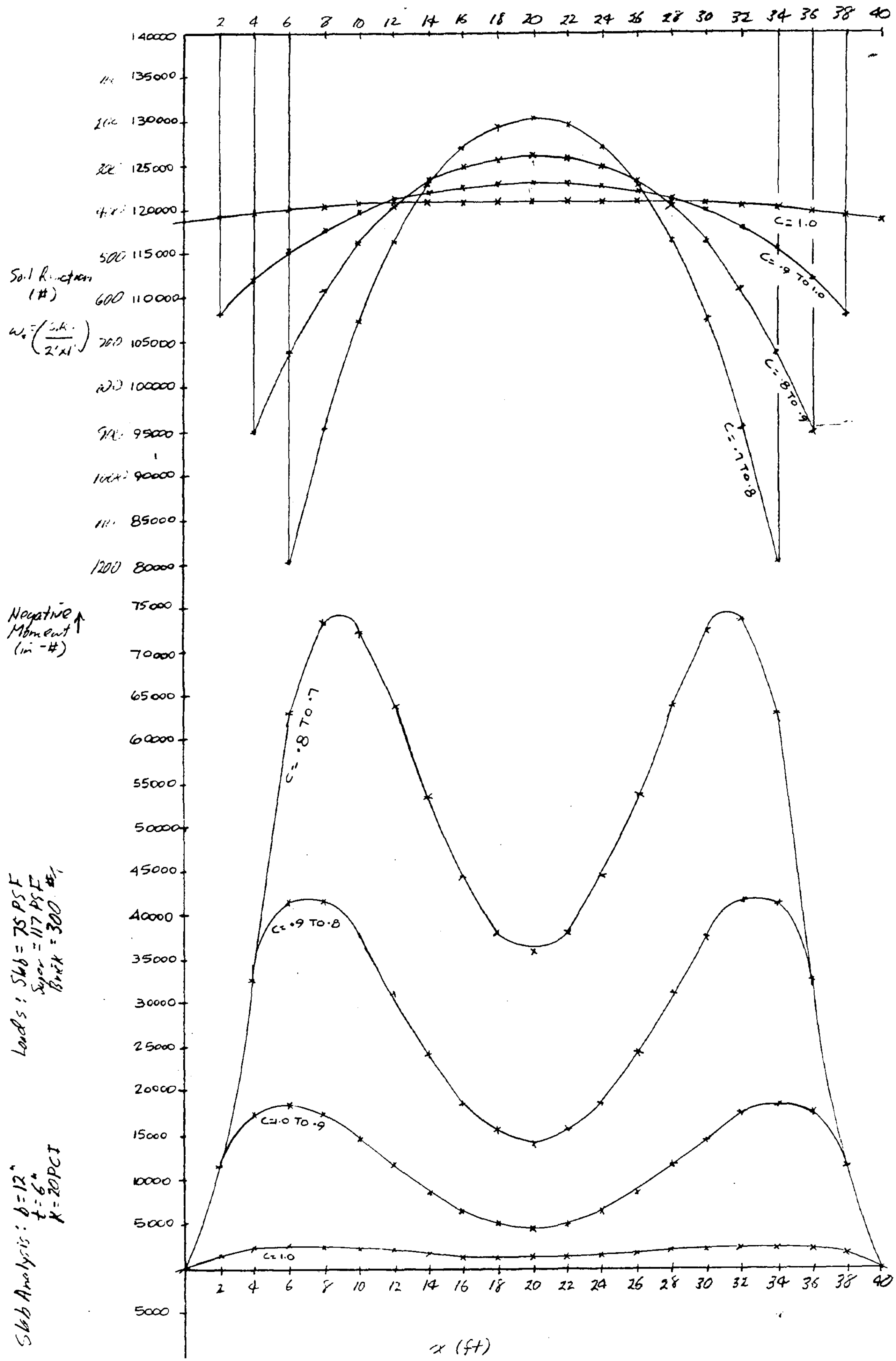
C=1.0
C=.9 To 1.0
C=.8 To .9
C=.7 To .8
C=.8 To .7
C=.9 To .8
C=1.0 To .9
C=1.0
LOADS: SLAB = 75 PSF
SUPER = 160 PSF
Brick = 300 #/'
SLAB ANALYSIS: b = 12"
t = 6"
K = 5 #/in³

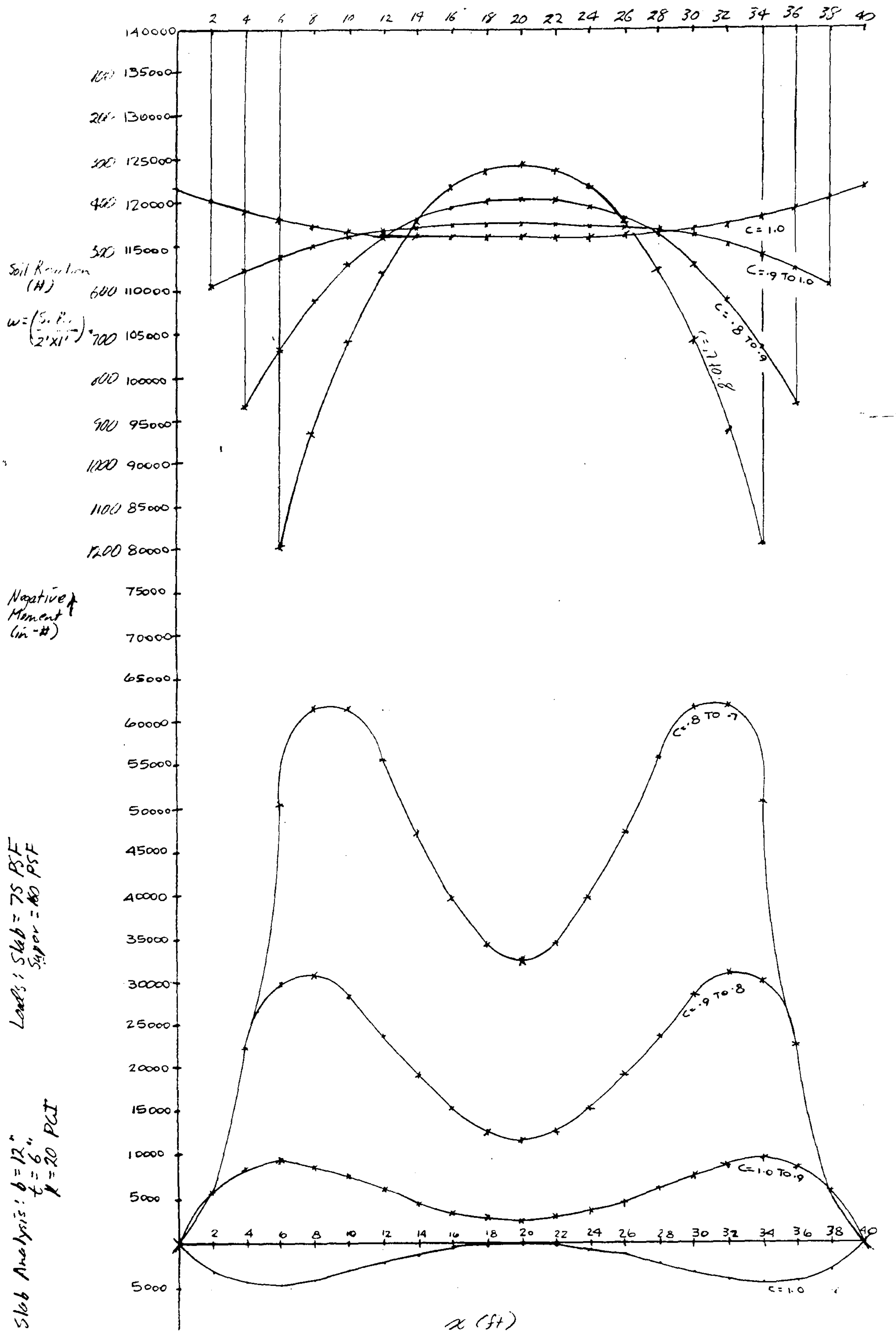


Soil Reaction (#)
$\omega = \left(\frac{S.R.}{2'(1')}\right)$
Negative Moment (in-#)
Loads: Slab = 75 PSF
Super = 160 PSF
Brick = 600 PSF
Slab Analysis: b = 12"
t = 6"
K = 5 PCI
C = .8 TO .7
C = 1.0
C = .9 TO 1.0
C = .8 TO .9
C = .7 TO .8
C = .9 TO .8
C = 1.0 TO .9
C = 1.0
x (ft)

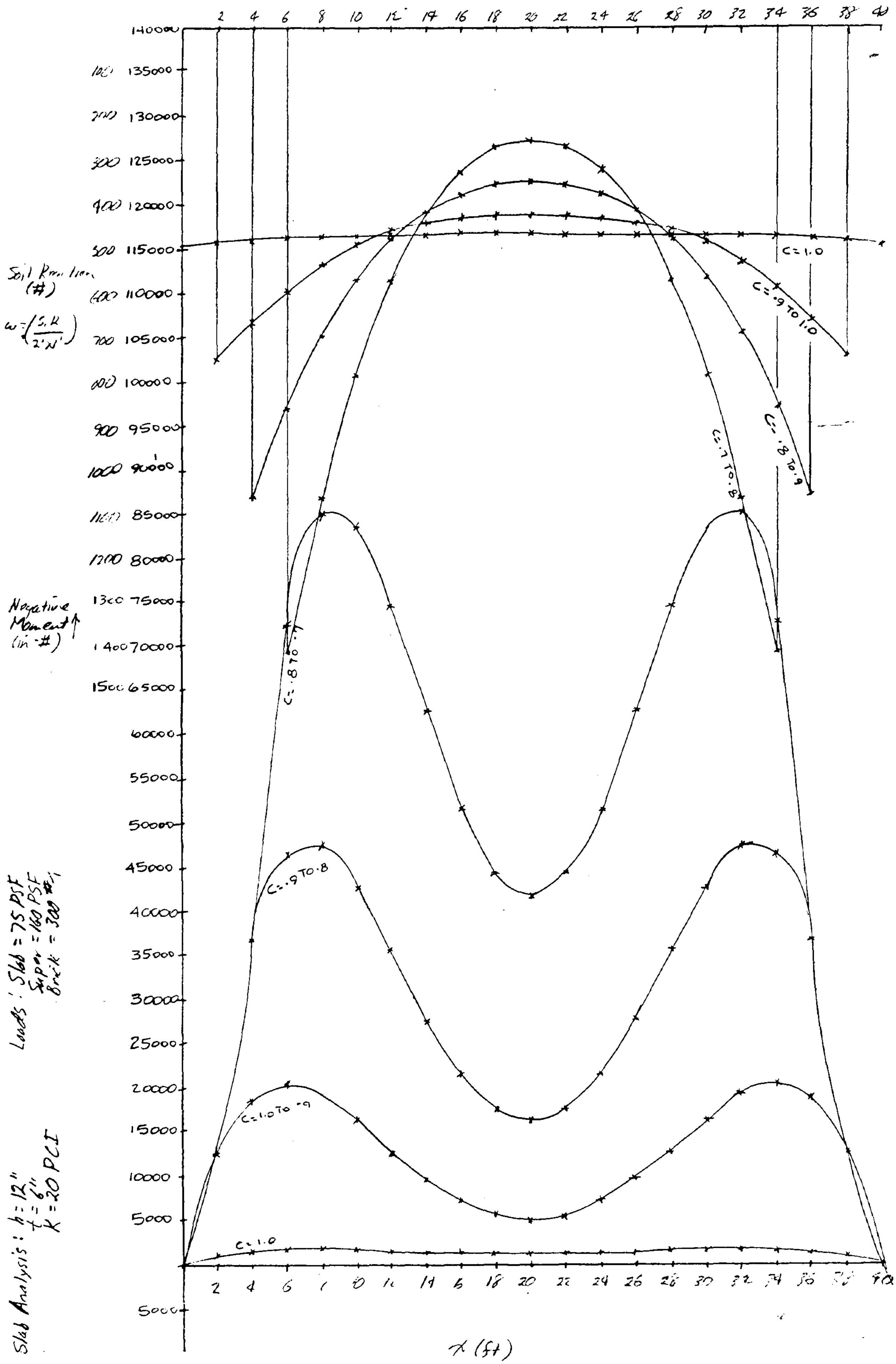


Negative Moment (in-lb)
Loads: Slab = 75PSF
Super = 117PSF
Slab Analysis: b=12"
t=6"
K=20PCI
c=1.0
c=.9 TO 1.0
c=.8 TO .9
c=.7 TO .8
c=.8 TO .7
c=.9 TO .8
c=1.0 TO .9
x (ft)

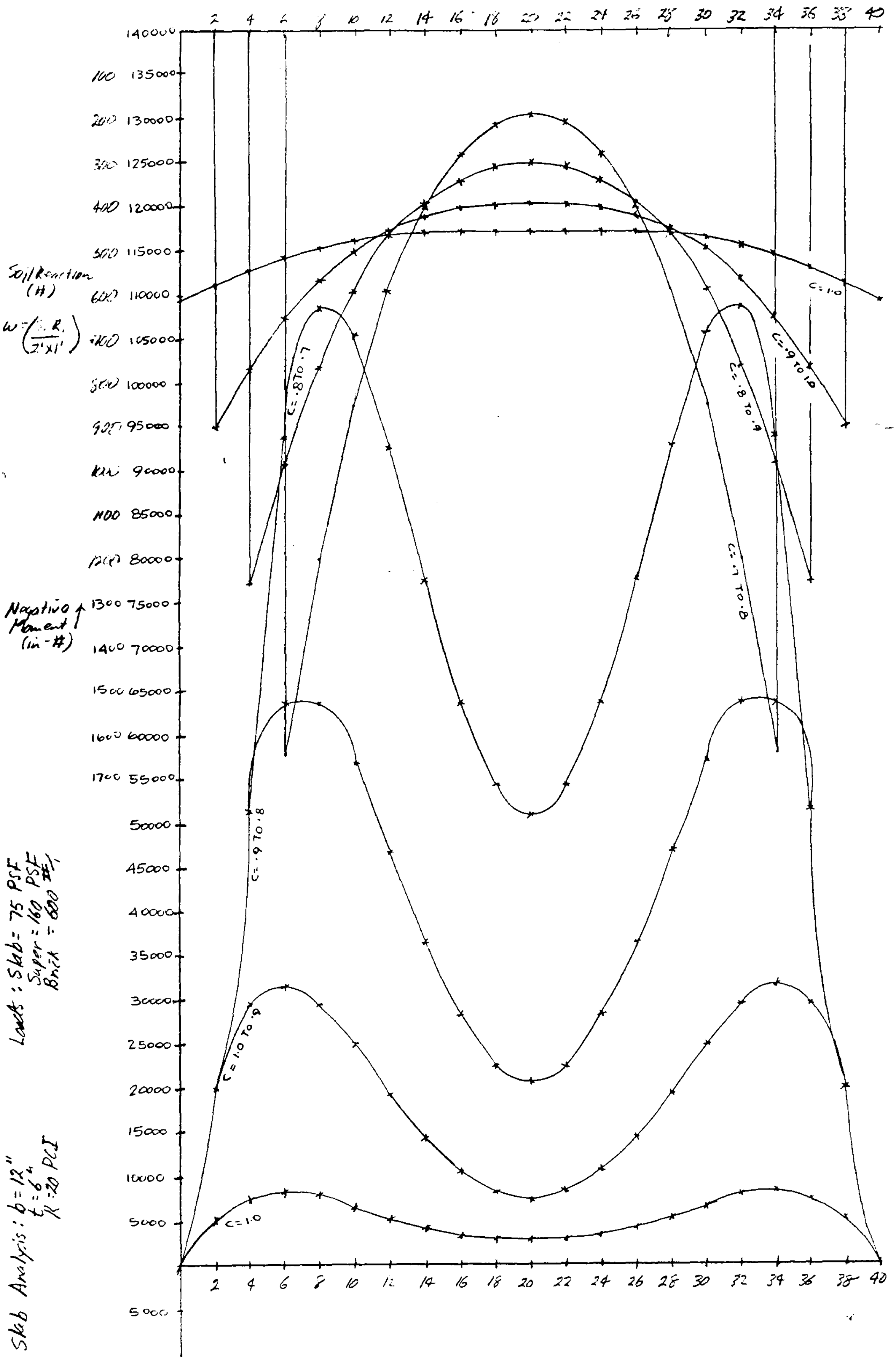

2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
140000
135000
130000
125000
120000
115000
110000
105000
100000
95000
90000
85000
80000
600
1200
Soil Reaction (#)
C=1.0
C=.9 TO 1.0
C=.8 TO .9
C=.7 TO .8
Negative Moment (in-#)
75000
70000
65000
60000
55000
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
C=.8 TO .7
C=.9 TO .8
C=1.0 TO .9
C=1.0
Loads: Slab = 75 PSF
Super = 117 PSF
Brick = 300 #/'
Slab Analysis: b=12"
t=6"
K=20PCI
x (ft)



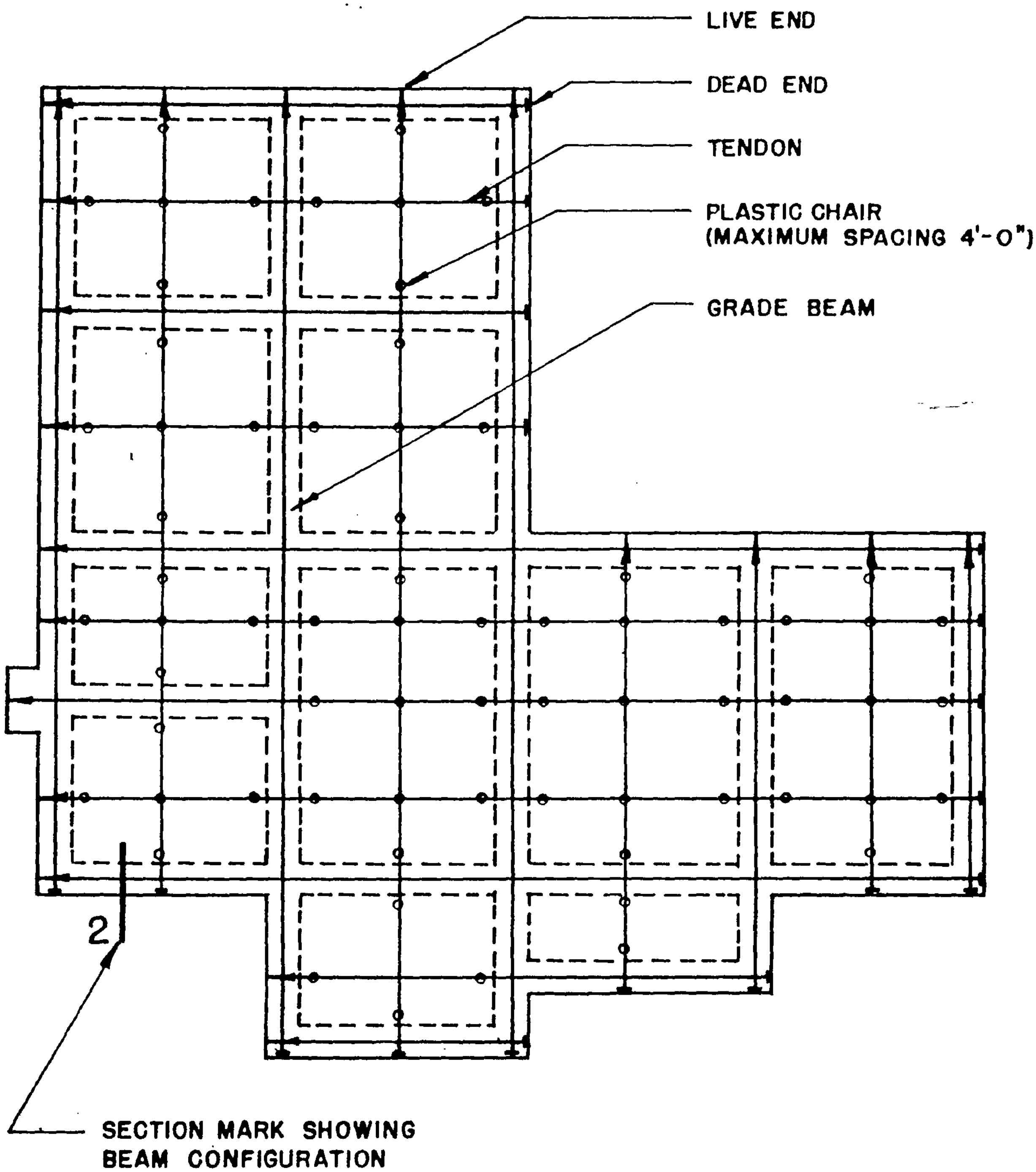
Soil Reaction (#)
$w = \left(\frac{S.R.}{2' \times 1'}\right)$
Negative Moment (in-#)
C = 1.0
C = .9 TO 1.0
C = .8 TO .9
C = .7 TO .8
C = .8 TO .7
C = .9 TO .8
C = 1.0 TO .9
C = 1.0
Loads: Slab = 75 PSF
Super = 100 PSF
Slab Analysis: b = 12", t = 6", k = 20 PCI
x (ft)



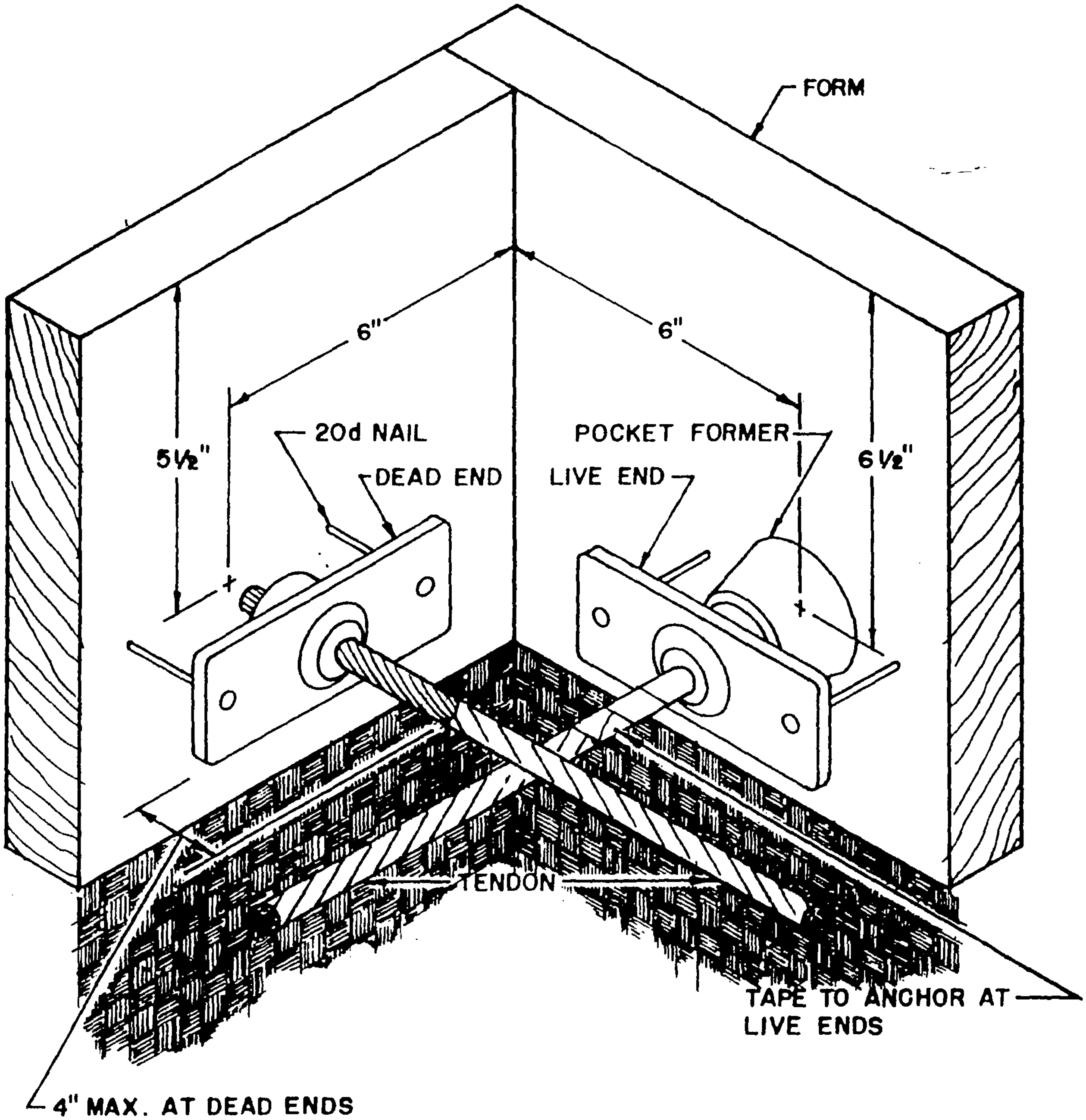
Soil Reaction (#)
w = (S.R / 2'N')
Negative Moment ↑ (in-#)
Loads: Slab = 75 PSF
Super = 160 PSF
Brick = 300 #/'
Slab Analysis: h = 12"
t = 6"
K = 20 PCI
x (ft)
C=1.0
C=.9 TO 1.0
C=.8 TO .9
C=.7 TO .8
C=.8 TO .7
C=.9 TO .8
C=1.0 TO .9
C=1.0


Soil Reaction (#)
$w = \left(\frac{S.R.}{2' \times 1'}\right)$
Negative Moment (in-#)
Loads: Slab = 75 PSF
Super = 160 PSF
Brick = 600 #/
Slab Analysis: b = 12"
t = 6"
K = 20 PCI
C = 1.0
C = 1.0 TO .9
C = .9 TO .8
C = .8 TO .7
C = .7 TO .8
C = .8 TO .9
C = .9 TO 1.0

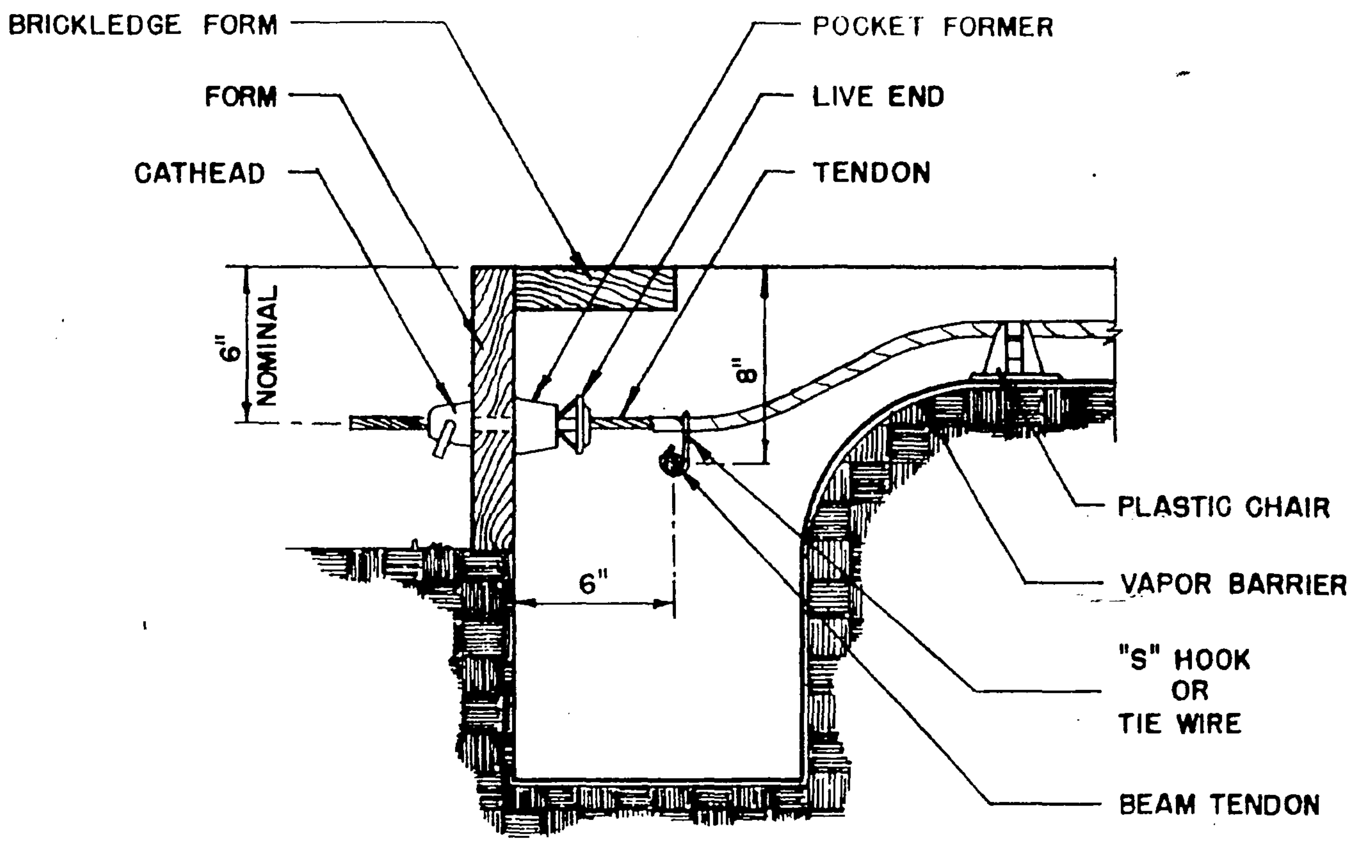
# TYPICAL DETAILS AND GENERAL NOTES



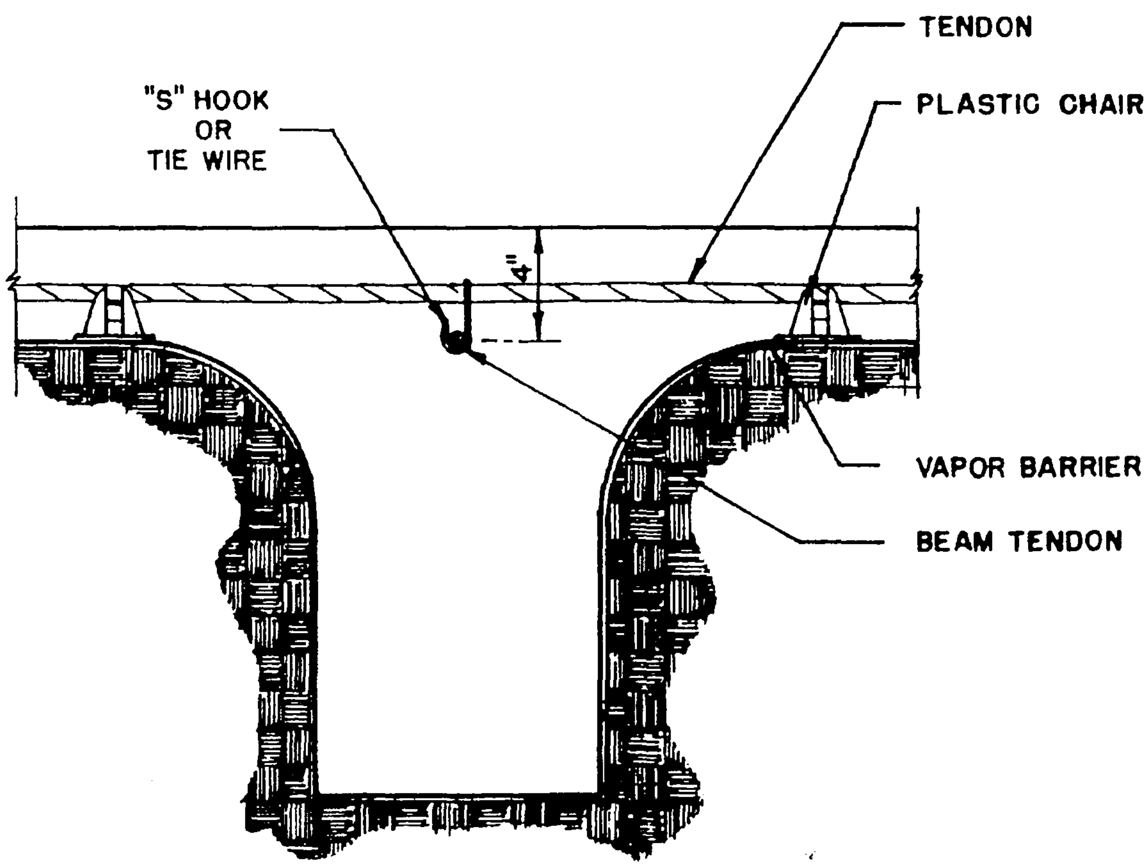

## TYPICAL FOUNDATION PLAN



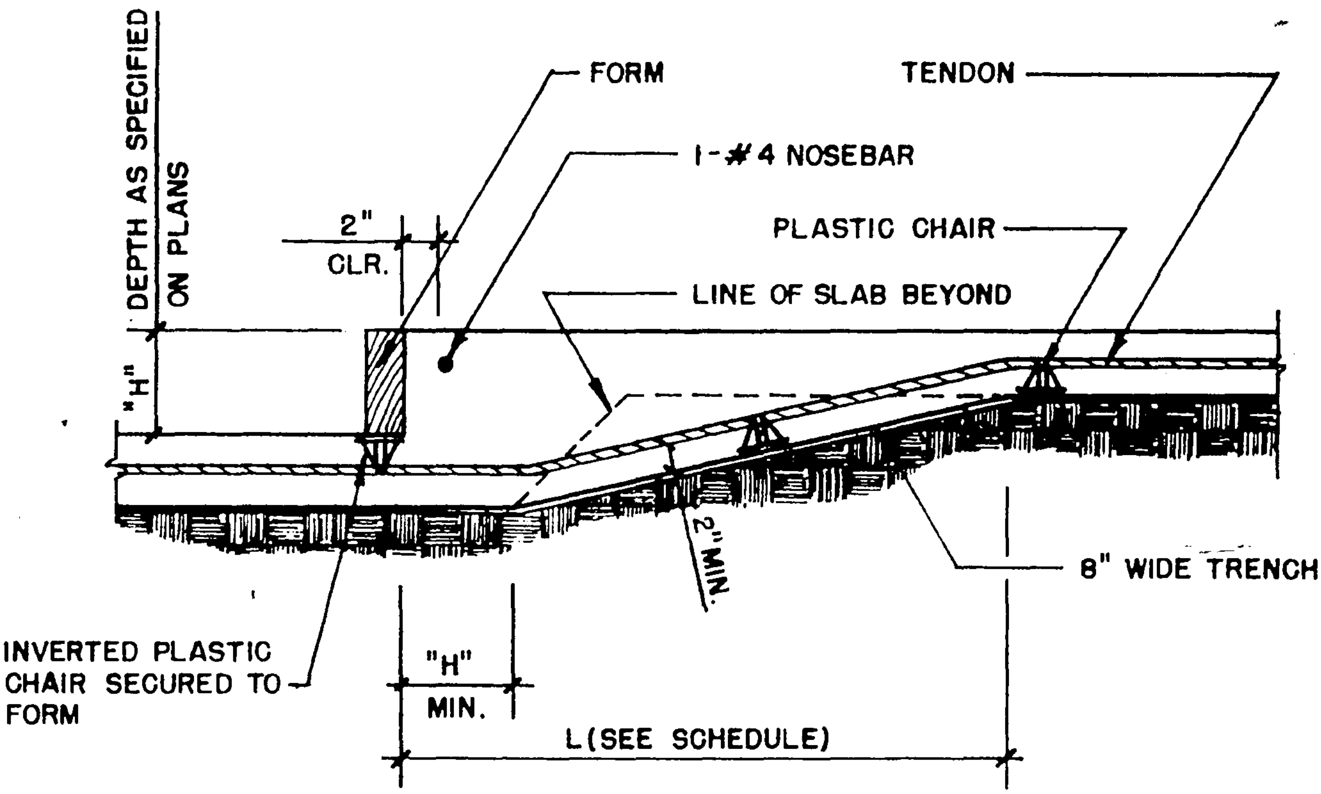

# Typical Outside Corner




TYPICAL PERIMETER BEAM




TYPICAL INTERIOR BEAM




## SCHEDULE FOR TYPICAL
# TENDON TRENCH AT DROPS

| NOMINAL DROP (H) | TRENCH LENGTH (L) |
|---|---|
| 2" | 2' - 0" |
| 4" | 3' - 0" |
| 6" | 4' - 0" |
| 8" | 5' - 0" |
| 10" | 6' - 0" |
| 12" | 7' - 0" |

## TYPICAL DETAIL AT DROP

## GENERAL NOTES - DESIGN

1. This foundation is designed in accordance with current acceptable engineering practices for the site shown on the plans and may not be used in any other location.

2. As with all ground supported slabs, this foundation is designed to move with the underlying soils while sustaining a calculated amount of flexure. It may also sustain normal temperature and shrinkage cracks as a result of the concrete curing process.

3. The design is based on the following assumptions:

   A. Final grading is completed as outlined in the General Notes - Sitework.

   B. Final grade and a fairly uniform moisture level is maintained for the life of the foundation.

   C. The foundation is not installed during a dry or wet period which is considered extreme or abnormal for the area. If such is the case, builder shall notify the engineer prior to trenching for a possible re-design.

4. This foundation is designed in accordance with the following geotechnical investigation:

   Soil Report #: ______________________________

   By: ______________________________

   Dated: ______________________________

## GENERAL NOTES - SITEWORK

1. Site preparation beneath the slab shall be in accordance with the soil report and shall meet the following minimum requirements:

   A. Strip all vegetation down to natural soil. Remove all trees within a close proximity of the foundation.

   B. Proof-roll exposed sub-grade. Backfill and compact tree-holes or soft pockets with material similar to the site materials.

   C. Bring subgrade to required elevation with select fill material. Select fill shall be sandy clay or clayey sand, free of organic material, having a plasticity index greater than 7, but less than 20.

   D. Fill shall be placed in maximum 8" lifts and compacted to 95% of its maximum dry density as determined by ASTM D698 (Standard Proctor). Where large depths of fill occur, field density tests are required for each lift located at or below the bottom of grade beams.

2. The four-inch sand fill shall be well-compacted bank sand or other clean granular material.

3. Initial site grading shall be completed prior to setting forms. Final grade shall slope away from the foundation one-inch/foot for the first five feet such that positive drainage away from the slab is assured.

## GENERAL NOTES - CONCRETE

1. Concrete shall be supplied and constructed in accordance with ACI-318 latest edition and shall have a minimum 28-day compressive strength of <u>2500 PSI</u>.

2. Water shall not be added to concrete at the jobsite unless approved by the engineer. If more workability is needed, contractor shall specify required slump on job order. Concrete plant to increase workability by adding up to 5% air entrainment, additional cement, or other approved admixtures.

3. Calcium chloride or admixtures containing calcium chloride shall not be used as additives. Where fly ash is used, only type C fly ash shall be accepted and a maximum of 15% may be substituted for cement.

4. Concrete shall not be placed at temperatures below 40 degrees Fahrenheit, in rainy weather or in other adverse weather conditions.

5. Concrete shall be well consolidated, especially in the vicinity of tendon anchorages.

6. A 6 mil polyethylene vapor barrier shall be placed under all slabs. All laps shall be taped.

7. Forms to be stripped no less than 24 hours and no more than six days after placement of concrete.

8. Builder shall verify all dimensions, drops, offsets, brickledges, inserts and openings with architectural drawings.

## GENERAL NOTES - REINFORCING STEEL

1. Reinforcing steel shall be per ASTM grade 60 with deformations per ASTM A305 and shall be detailed and installed per ACI-318 latest edition.

2. Welded wire fabric shall be 6 x 6 x W2.9 x W2.9 WWF (6 gage) per ASTM A185. Where shown on the plans, WWF shall be supplied in sheets and shall be placed two inches below the top of concrete.

3. Where field splices in the continuous reinforcing occur, bars shall be lapped a distance of 30 times the bar diameter.

4. Where reinforcing steel is shown in the exterior grade beams, provide corner bars in the outside face to match the horizontal steel from the intersecting interior and exterior grade beams.

5. At all re-entrant corners provide 2 #4 x 5'-0" in the slab.

## GENERAL NOTES - TENDONS

1. Prestressing steel tendons shall consist of seven-wire stress-relieved strand conforming to ASTM A416 with a minimum ultimate tensile strength of 270,000 PSI.

2. Tendons shall be coated with a permanent rust preventative lubricant within a plastic sheath. Tape all breaks in sheathing and tape sheathing ends up to live end anchors and to within four inches of dead end anchorages.

3. Tendons shall be initially prestressed to hand-tightness against the forms and shall be supported on chairs at 38 inches each way. All chairs shall be tied and all S-hooks shall be crimped.

4. Acceptable tolerances for the tendon placement shall be as follows:

| | |
|---|---|
| Beam tendons | $\pm$ 1 in. vert., $\pm$ 1/2 in. horiz. |
| Slab tendons | $\pm$ 1/2 in. vert., $\pm$ 12 in. horiz. |

Note that slab tendon horizontal deviation shall be limited to one-inch/foot of cable in order to miss obstructions.

## GENERAL NOTES - STRESSING

1. Tendons shall be stressed to 33.0 KIPS per strand and shall have a minimum set load of 28.9 KIPS.

2. Actual tendon elongations shall measure within 10% of theoretical elongations and corresponding pressure gauge readings.

3. Tendons shall be stressed no earlier than three days and no later than ten days after concrete placement. During cold weather conditions, stressing shall take place between seven and fourteen days after concrete placement.

4. Concrete shall have attained a minimum compressive strength of 70% of its 28-day strength at the time of stressing.

5. Brickwork shall not begin before stressing is completed.

6. Tendons shall be cut or burned at one inch from the wedges. Pockets shall be filled with non-shrink grout.

# FOUNDATION DESIGN ISSUES

## A State of the Art Review of Custom Home Foundations

Michael Skoller, PE

# FOUNDATION DESIGN FOR CUSTOM RESIDENCES AND LIGHT COMMERCIAL BUILDINGS ON EXPANSIVE SOILS

By

Michael A. Skoller

National Structural Engineering, Inc.

November 5, 1998

# MICHAEL SKOLLER

730 North Post Oak Road, Suite 300
Houston, TX 77024
Telephone: (713) 956-2094


**President**
National Structural Engineering, Inc. November 1992 - Present

Structural Engineering design and forensic consulting for the residential, commercial and light industrial industries. This includes framing and foundation designs, drawings and follow-up field construction reviews. Typical projects done include:

- Over 1000 residences in Houston and throughout the country
- Miscellaneous structures including churches, temples and schools
- Foundations for cranes, tanks and miscellaneous equipment
- Restaurants, office buildings and other miscellaneous commercial buildings
- Expert witness work for the above

**Part Owner** June 1989 - November 1992
Structuneering Inc.

Structural Engineering consulting for the residential and low rise commercial industries. See above.

**Chief Engineer & General Manager** October 1985- May 1989
Commercial Carports, Incorporated

Structural design of steel and combination wood/steel covered parking facilities and open sided buildings.

**Project Engineer** January 1985- September 1985
IMI Engineering Company

Contract designed and managed turnkey onshore and offshore workover and drilling rig packages.

**Project Engineer/Sales (part-time)** January 1984-January 1985
Vulcan Foundation Repair

Inspection and repair recommendations of residences and commercial buildings with foundation problems.

**Staff Engineer** December 1981-December 1984
Welltech, Incorporated

Designed and managed the fabrication of projects involving the structural, mechanical and hydraulic drilling and workover systems.

**Project Engineer** November 1979-November 1981
National Steel Products Company/ Stran Steel

Structural design of metal buildings and grain storage facilities.

**Chief Engineer** December 1976-October 1979
Parkerburg Tank division of Maloney-Crawford Corporation, Tulsa, OK

Structural design of oil and grain storage facilities.

**Project Engineer** June 1976-November 1976
Lee C. Moore Corporation, Tulsa, OK

Structural design and field inspections of oilfield drilling derricks and substructures.

## EDUCATION

| | | |
|---|---|---|
| University of Houston- | M.B.A. | December 1984 |
| Carnegie-Mellon University- | B.S. Civil Engineering | May 1976 |

## PROFESSIONAL AFFILIATIONS

Registered Professional Engineer in the State of Texas and approximately forty additional states including the National Council of Engineering Examiners.

Foundation Performance Committee (President)
American Society of Civil Engineers
Post Tensioning Institute
American Concrete Institute
Brick Institute of America
National Concrete Masonry Association
Concrete Reinforcing Steel Institute
American Plywood Association
Light Gage Steel Engineers Association
American Welding Society
Structural Engineers Association Of Texas

*I. Introduction*

*II. Foundation Options On Expansive Soils*

*A. Isolated Floor Systems*

*1. Structural Slabs*

*2. Crawl spaces*

*B. Stiffened Slabs on Fill with Drilled Piers*

*C. Stiffened Slabs with Drilled Piers*

*D. Stiffened Slabs on grade*

*1. Post-Tensioned slabs*

*2. Stiffened slabs with steel*

*III. Additional Comments and Suggestions*

## I. Introduction

A geotechnical report should be acquired for every foundation design. The geotechnical report will allow the structural engineer to become aware of the site conditions that include the expansiveness of the soils, vegetation (such as trees), sloped sites, existing on-site fill and perched water tables. Perched water table conditions exist when highly permeable soils (such as sands) lie over low permeable soils (such as clays). During the wet seasons these sites tend to hold water. This condition is very common in the north and northwest areas of Houston. In addition, the structural engineer should visit the site if adverse conditions are known to exist (for example, sloped sites). When designing a house on a sloped site the structural engineer should have a slope stability analysis performed by the geotechnical engineer determining pier depth and size, retaining wall requirements etc. If there is a known active fault on the site, all involved parties should be aware of the building risk.

Foundation designs on expansive soils are a challenge. Most of the soils in this area are expansive. Expansive soils can be defined as those with plasticity indexes (PI's) above 20. The greater the plasticity index the more expansive the soils. The more expansive the soils are, the greater the potential is for movement. In some areas the PI can exceed 90. Expansive soils act like a sponge. During the rainy seasons the soils expand, and during the summer droughts the soils shrink. The soils that are closest to the surface are affected the most by the seasonal moisture changes. If the soils that a foundation is supported on swell and shrink, then the foundation may move up and down, respectively. The deeper that the foundation is supported below the surface, the less the foundation is affected by the seasonal moisture changes.

## *II. Foundations Options On Expansive Soils*

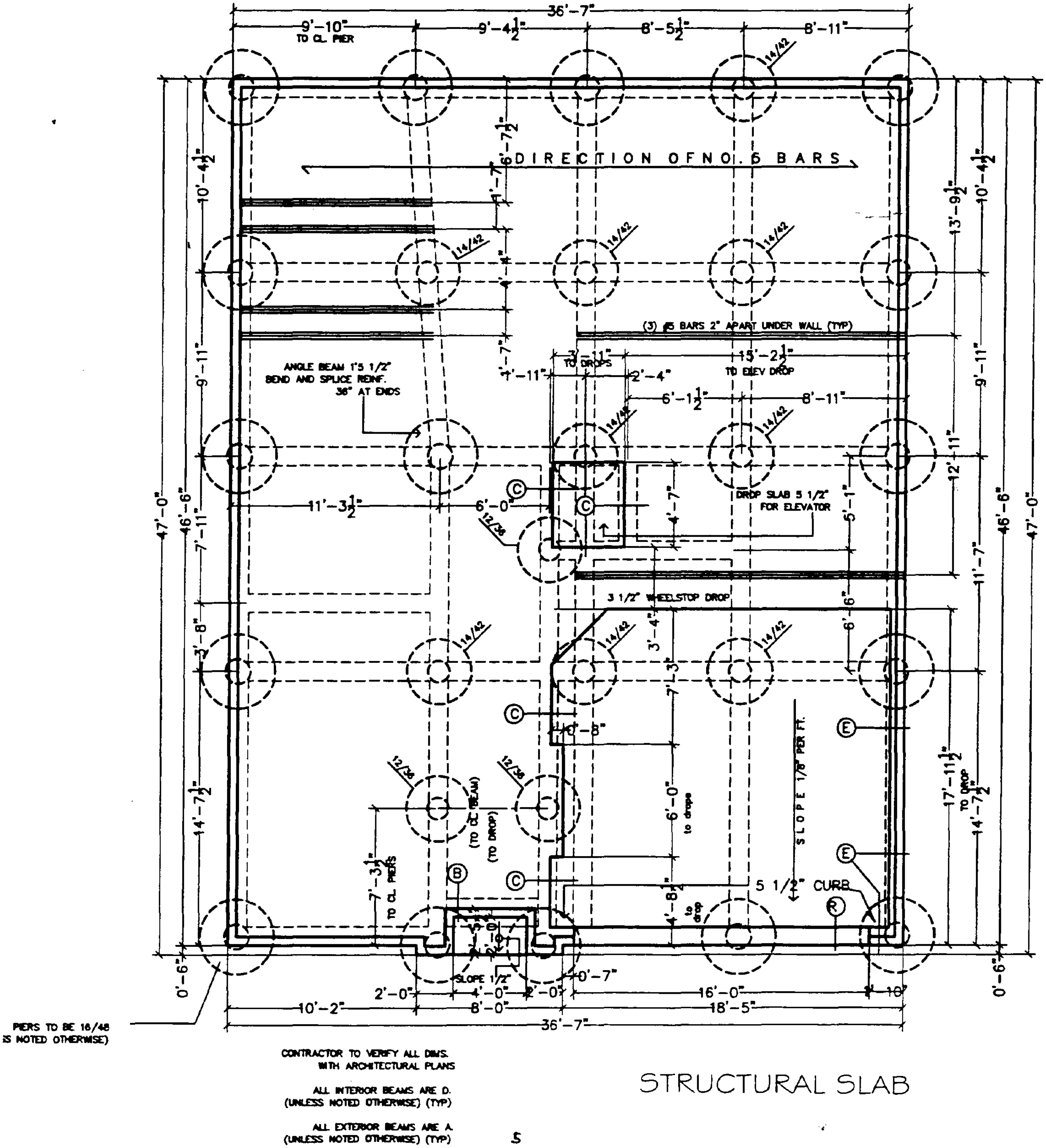
The following sections include various types of foundation designs that can be used to resist the movement of expansive soils. They are listed in order from least potential movement to most potential movement. The design engineer should be aware of the various foundation types and the inherent risks of each. The risks versus costs should be discussed with the client and all other parties involved in the decision making process. For example, during a drought a post-tensioned or stiffened slab on-grade has the potential to move more than a stiffened slab with piers; however, it costs substantially less.

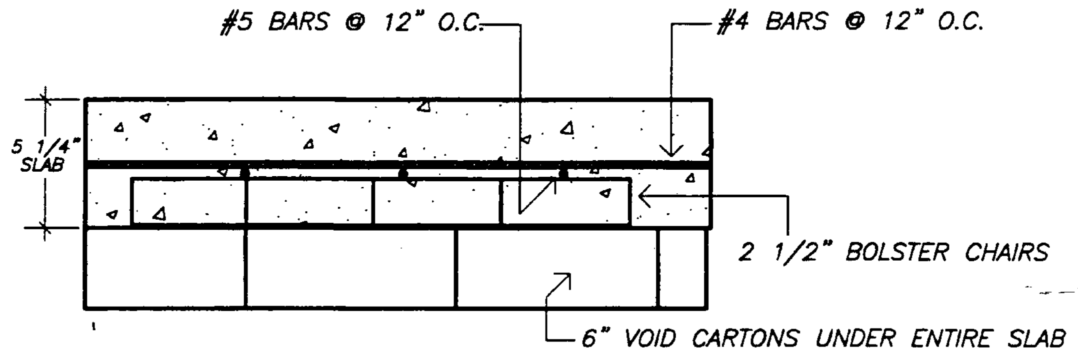
### *A. Isolated Floor Systems*

Isolated floor systems are generally considered to be the best foundation type for minimizing movement. The foundations are designed such that the majority of the foundation does not come in contact with the expansive soils that change moisture content and volume due to seasonal rains and droughts. The foundation system is supported by drilled under-reamed piers at a depth of 8'-0" to 20'-0" below natural grade. The pier depth is dependent on how expansive the soils are, the strength of the soils and the location of trees. The expansive soils at this depth maintain a constant moisture content and are not affected by the seasonal moisture changes and, thereby, don't move up and down like the soils do at the surface. Watering of the foundation is not necessary for buildings incorporating isolated floor systems. These foundation designs are, of course, the most expensive.

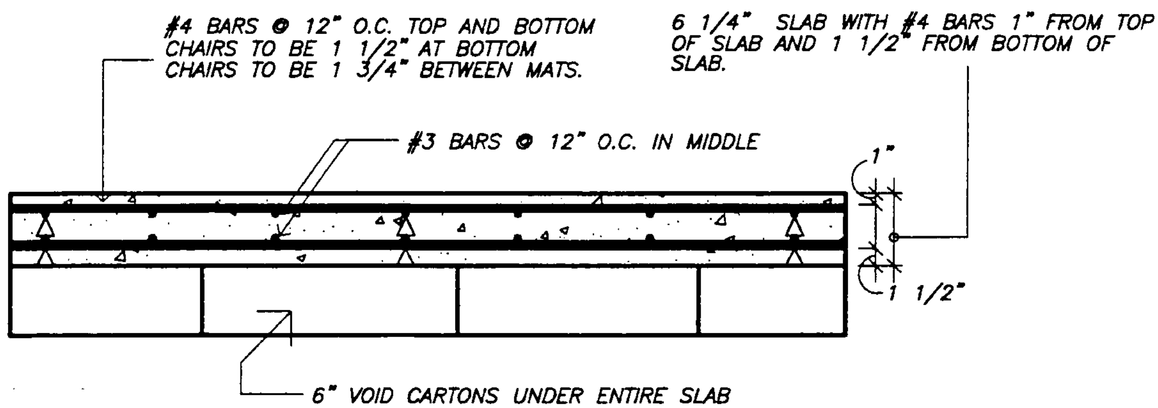
#### *1. Structural Slabs*

The structural slab has cardboard carton forms, which form a void separating it from the surface soils. These forms range in depth from 4" to 8" and depend on the expansiveness of the soils. The more expansive the soil, the higher the plasticity index, the deeper the cardboard carton form. The slab is called a "structural slab" because it spans between the grade beams that are supported by the drilled piers, similar to an elevated parking garage. The slabs can range in depth from five to seven inches. The reinforcement can consist of a single or double mat. The structural slab shall be designed in accordance with the American Concrete Institute 318 code. Following are some typical details and a design for structural slab foundations:





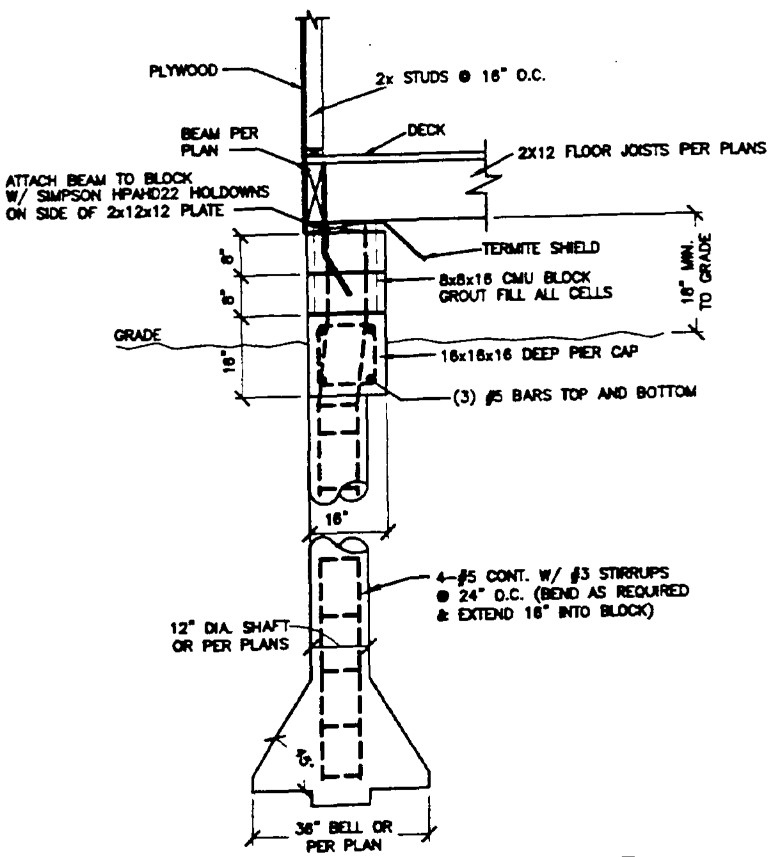

TYPICAL STRUCTURAL SLAB WITH SINGLE MAT



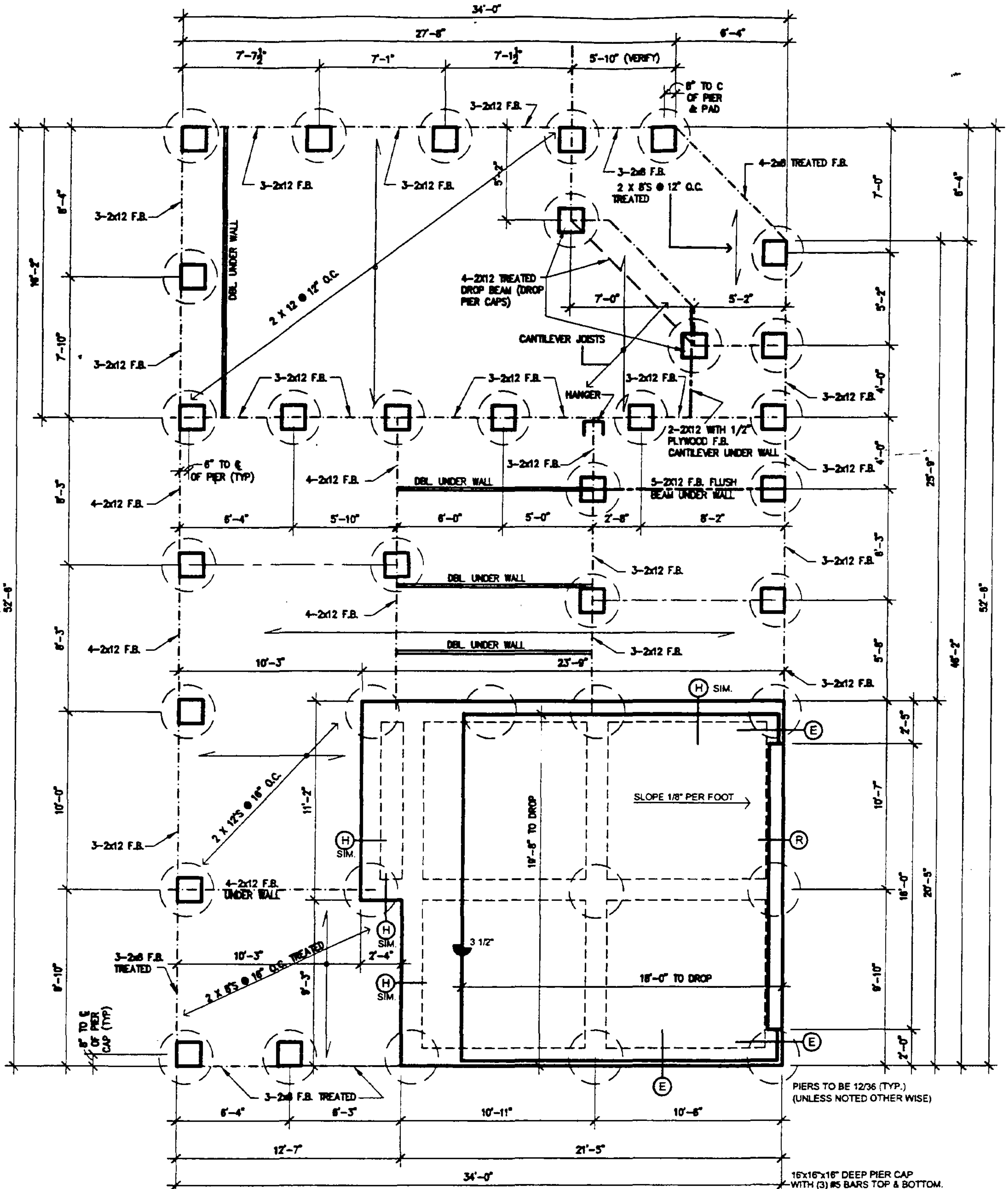

TYPICAL STRUCTURAL SLAB WITH DOUBLE MAT

## 2. *Crawl Spaces*

A crawl space can be erected utilizing any of the following methods: (a) Wood floor joists supported by wood, steel, or concrete beams or a combination of the three; (b) Concrete floor joists or poured in place concrete floor supported by concrete beams; or (c) Steel bar joists or cold formed cee sections supported by steel or concrete beams. In all of the preceding cases the support for the foundation is deep drilled under-reamed piers. The following is a typical detail and drawing for a crawl space supported by wood joists:




FOOTING & FOUNDATION DETAIL (G)




3 1/2" x 11 1/4" CAN BE SUBSTITUTED FOR 4-2x12

2X12 @ 16" O.C. FLOOR JOISTS (UNLESS NOTED OTHERWISE)

16"x16"x16" DEEP PIER CAP WITH (3) #5 BARS TOP & BOTTOM. PROVIDE (2) COURSES OF 8" BLOCK (HEIGHT AS REQUIRED) PROVIDE TERMITE SHIELD & 2"x12"x12" LONG PRESSURE TREATED PLATE AT TOP. BEAM IS ATTACHED TOP OF COLUMN WITH (2) SIMPSON HPAH D22 HOLDOWNS (TYP. AT ISOLATED FOOTINGS)

## FOUNDATION PLAN

1/4" = 1'-0"

CONTRACTOR TO VERIFY ALL DIMENSIONS WITH ARCHITECTURAL PLANS

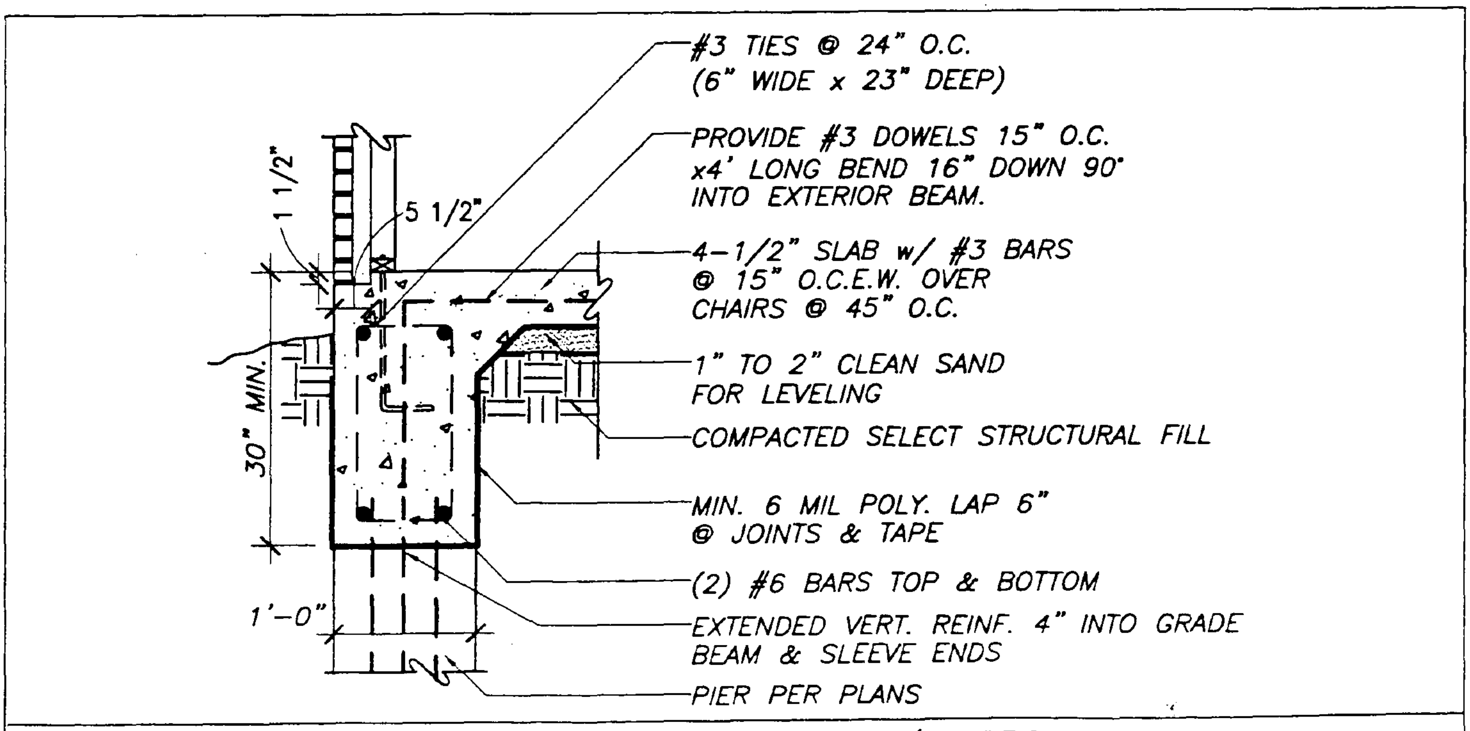
ALL INTERIOR BEAMS ARE D. (UNLESS NOTED OTHERWISE) (TYP)

ALL EXTERIOR BEAMS ARE A. (UNLESS NOTED OTHERWISE) (TYP)

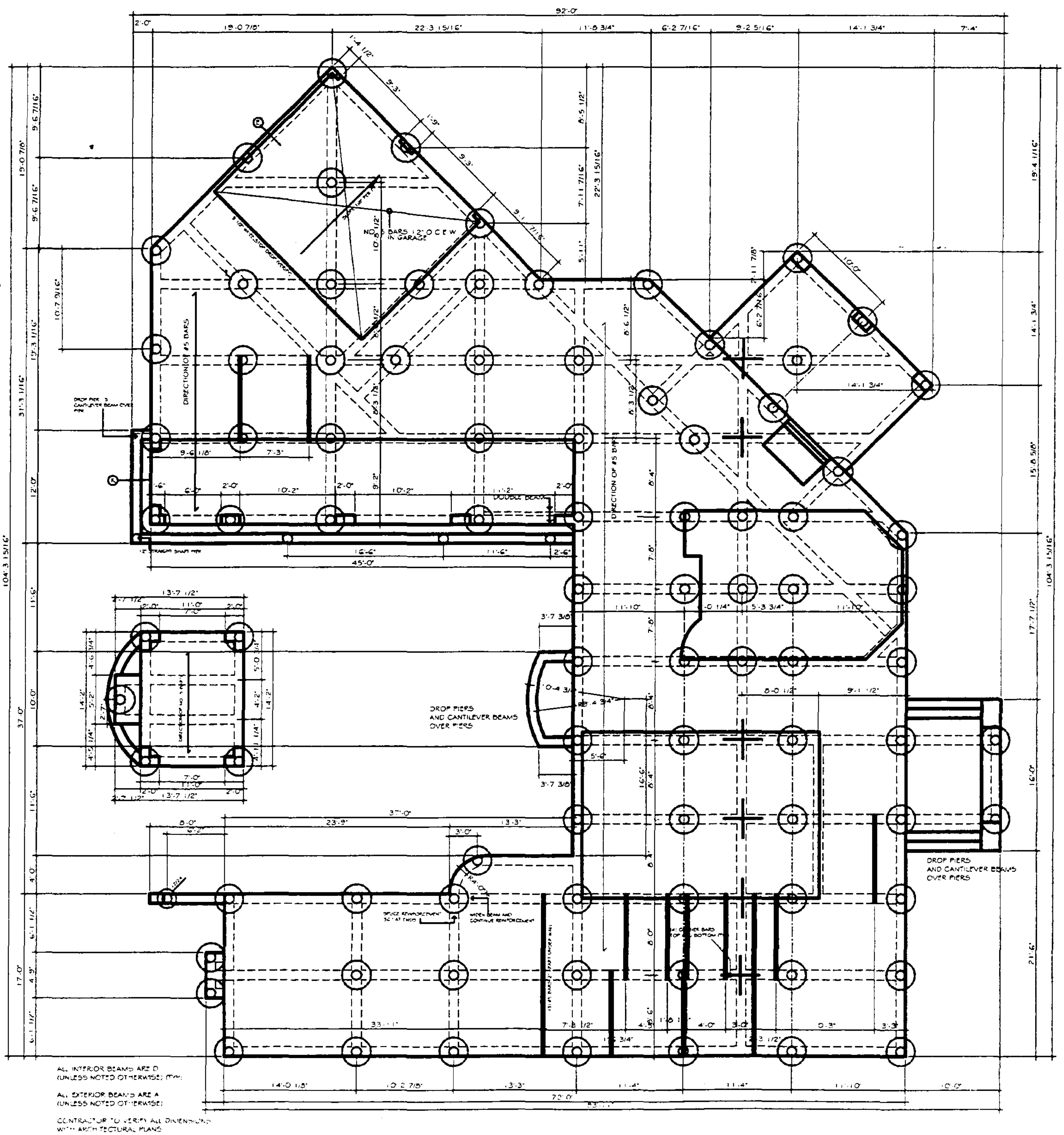
CRAWL SPACE SUPPORTED BY WOOD JOISTS

### B. Stiffened Slabs on Fill with Drilled Piers

This type of foundation is a concrete slab that sits on non-expansive select structural fill. Select structural fill can be defined as sandy clays with a plasticity index between 10 and 20 and a liquid limit less than forty. The fill acts as a buffer zone between the expansive soils and the slab reducing the potential movement of the foundation. The foundation should be designed as a "stiffened" slab. The grade beams will form a grid-like or "waffle" pattern in order to reduce the potential upward movement, or upheaval, caused by swelling soils. Continuity of grade beams should bear special consideration in soils with expansive soils, even in foundations with fill and piers. Using continuous footings in a grid-like fashion will help to reduce differential movement. Even slabs with angled sections or bay windows can be designed with continuous grade beams to supply some extra rigidity when movement occurs. The piers, which are usually not tied to the grade beams, are used to minimize downward movement, or settlement, caused by shrinking soils. I believe that the fill should extend three to five feet outside of the building pad in order to move the transition area away from the building. The stiffened slab on fill with drilled piers should be designed in accordance with the Welded Wire Mesh Institute manual "Design of Slab-on-Ground Foundations" and the American Concrete Institute's 318 and 302.1R codes. Following is a typical detail and drawing of a stiffened slab on fill with drilled piers:




EXTERIOR GRADE BEAM FOR SLAB-ON-FILL W/ PIERS




STIFFENED SLAB ON FILL WITH DRILLED PIERS

## *C. Stiffened Slabs with Drilled Piers*

This type of foundation is very similar to stiffened slabs on fill with drilled piers. The only difference is that there is no select structural fill required. Select structural fill is used only to raise the slab to its desired elevation. The foundation grade beams may be deeper and closer together than an identical slab with select structural fill. Potential movement is greater with this foundation system then that with select structural fill. The stiffened slab with drilled piers should be designed in accordance with the Welded Wire Mesh Institute manual "Design of Slab-on-Ground Foundations" and the American Concrete Institute's 318 and 302.1R codes.

## *D. Stiffened Slabs on grade*

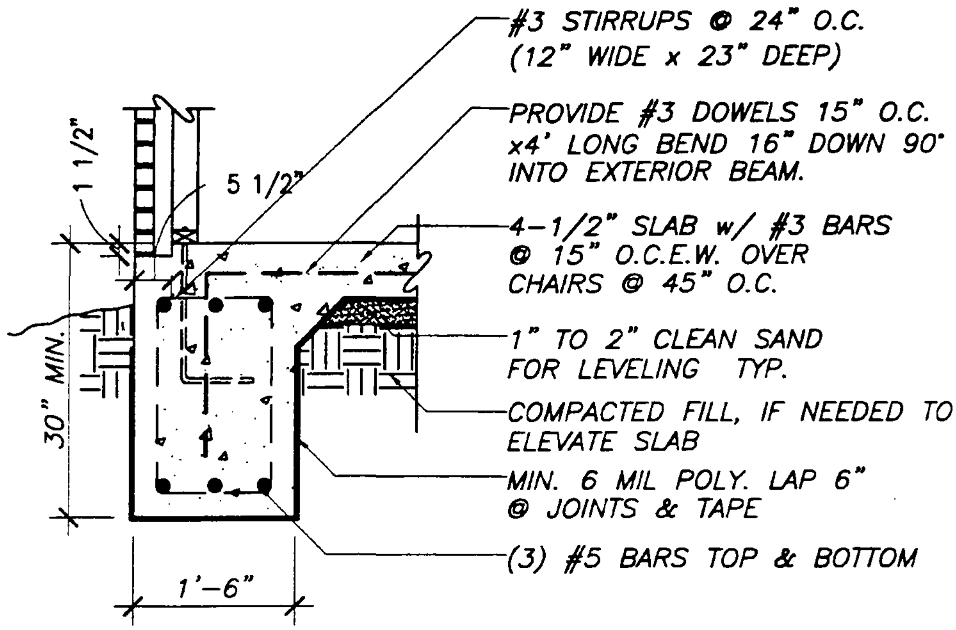
These are the most common, and least expensive, types of foundations used in the Houston area. The site is typically scraped six to eight inches, the sub-grade is compacted to 95% and any fill used to elevate the slab is also compacted to 95%.

### *1. Post-tensioned slabs*

Post-tensioned slab designs were covered by Lowell Brumley. Post-tensioned slabs should be designed in accordance with the Post-Tensioning Institute's "Design and Construction of Post-Tensioned Slabs-on-Ground" design manual.

### *2. Stiffened slabs with steel*

Stiffened slabs with steel are sometimes called conventional or "waffle" foundations. The entire slab is supported by the surface soils that are susceptible to the seasonal moisture fluctuations and movement. The foundation is designed utilizing beams that form a grid like pattern. Less differential movement will occur using a stiffer slab. The exterior grade beams are often wider to accommodate the heavier perimeter loads. Stiffened slabs with steel should be designed in accordance with the Welded Wire Mesh Institute's manual "Design of Slab-on-Ground Foundations" and the American Concrete Institute's 318 and 302.1R codes. The following is a typical detail of a stiffened slab on grade without drilled piers:

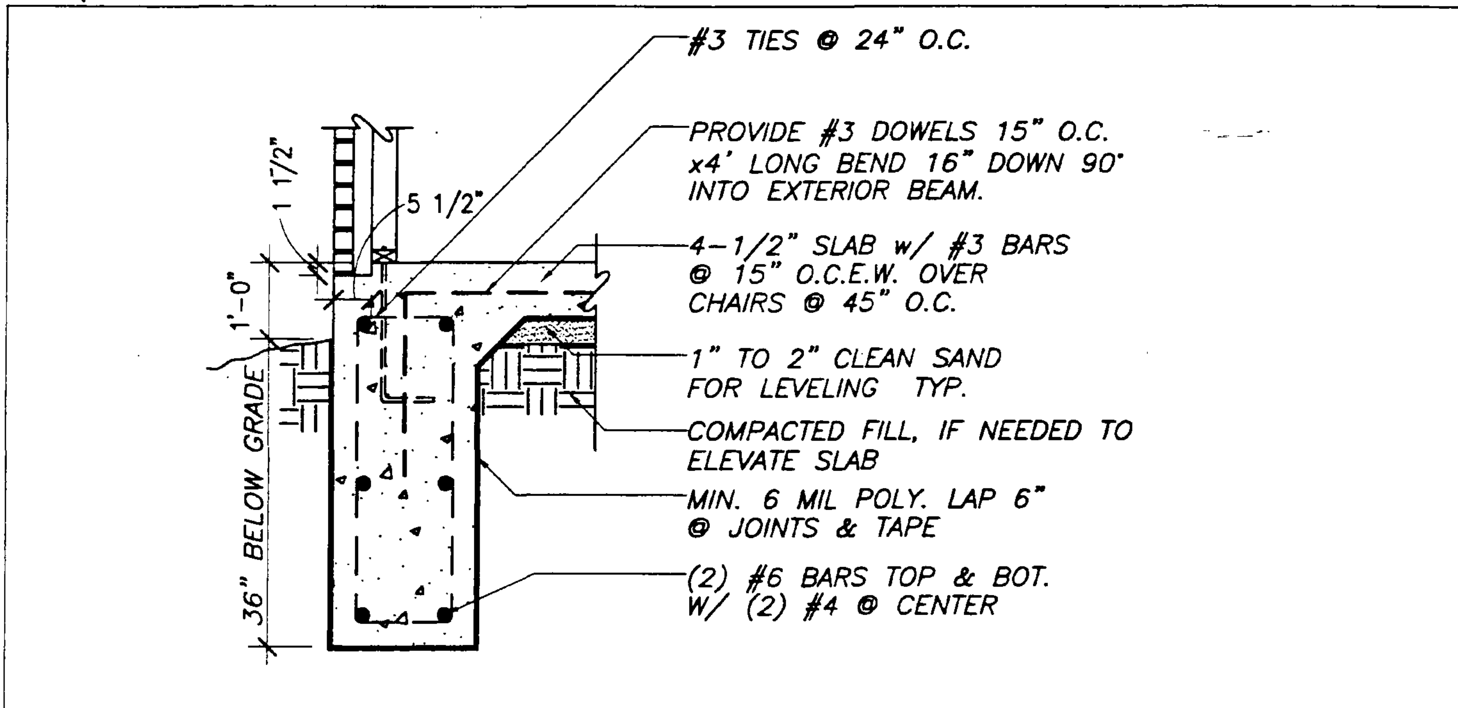



EXTERIOR GRADE BEAM FOR STIFFENED SLABS-ON-GRADE

## IV. Additional Comments and Suggestions

- Please note that all foundations will move somewhat. The following is a partial list of items that can adversely affect the performance of a foundation:
  - Plumbing leaks from sewer lines, pool drain lines and sprinkler lines.
  - Over-watering by sprinkler systems or soaker hoses. When the foundation was installed the soils were typically dry of, or near, optimum moisture content. Over watering will cause the soils to take on additional moisture and expand.
  - Bad or no drainage. Flower bed liners and planter areas near a foundation often won't allow water to drain properly.
  - Trees being planted too close to the foundation.
  - Trees dying or being removed in close proximity of the foundation.

- Deeper exterior grade beams below final grade will help to reduce moisture changes under the foundation, and therefore reduce the volume changes in the soils. Grade beams on all foundations, except isolated floor systems, should extend a minimum of twelve inches below final grade. A deep exterior grade beam is shown below:




DEEP EXTERIOR GRADE BEAM

- In cases where the floor system is not isolated from the soils, a stiffer slab will provide less differential movement. The following items will help achieve a stiffer slab:
  - Provide grade beams that are closer together or have more steel
  - Provide deeper grade beams
  - Provide a thicker slab with more reinforcement

- Occasionally the client would like to have exposed stained concrete floors. As everyone should know, concrete shrinks and cracks as it cures. To minimize the size and quantity of the shrinkage cracks two things can be done:
  - Use a double mat of number three bars for slab reinforcement. When the steel reinforcement is close to the surface, the cracks should be smaller and less frequent.
  - Add fiber reinforcing to the mix.

- Do not use void cartons under the grade beams. It is a channel for water and the use of void cartons may exacerbate upheaval rather than minimize it. Water that collects in the void under the grade beams can travel down the sides of the piers and/or migrate into the soils under the slab. Use void cartons under the slab portion of the foundation rather the beam portion of the foundation since the slab comprises about 90% of the foundation in contact with the expansive soils.

- In 12" wide grade beams use (2) # 6 bars top and bottom instead of (3) # 5 bars top and bottom. When (3) # 5 bars top and bottom are used in a 12" wide beam, and are spliced, there is no room for 1 1/2" concrete aggregate to fit between the bars. Also, since grade beams are typically under load bearing walls and walls sometimes have a plumbing pipe extending from the wall into the grade beam, it would be impossible to have the center reinforcing bar be continuous when using (3) # 5 bars top and bottom. Please note that the total cost (labor plus material) for using (2) # 6 bars top and bottom in the grade beams will be less than that of using (3) # 5 bars top and bottom.

- Tree removal on a site with expansive soils needs special consideration because of the potential for upheaval. Everyone involved in the project, especially the end client, should be aware of the risks.

# FOUNDATION CONDITION SURVEY

## State Subcommittee Report

**Bill Lawson**

## CONSTRUCTION ISSUES

### Construction Testing

Jack Spivey

# FOUNDATION

# CONSTRUCTION / REPAIR

# INSPECTIONS

by

**JACK SPIVEY**

**J. SPIVEY & ASSOCIATES, INC.**

# INTRODUCTION

The following documents are the results of two years of work by the Inspections Sub-committee of the Foundation Performance Committee. I have served as chairman of this committee and my fellow members have been:

**MR. MICHEAL SKOLLER P.E.**
**MR. JOE EDWARDS**
**MR. LOWELL BRUMLEY P.E.**
**MR. DEAN EICHELBERGER**

Our meetings have taken place on a monthly basis and have been attended by many interested parties. Special recognition should be given to Mr. Jim Dutton of Duwest Foundations and Mr. Dan Jaggers of Olshan Foundation. Their assistance with the foundation repair sections was invaluable.

The topics for discussion have followed a general outline established at the onset of the meetings. It was determined that our basic intent would be to establish a set of standards and procedures for the inspection of foundation construction and foundation repairs. These standards were to be incorporated into an inspection document which would be thorough in its scope, but also easy to use. It was established early on in our discussions that the best form for our purposes would be a simple checklist, which would fully cover the subject of the inspection. It was also determined that keeping the checklist to one page would afford the most user friendly instrument for our purposes. Once these parameters were established the subjects of the inspections were taken in the following order:

FOUNDATION MAKE-UP POST TENSION

CONCRETE PLACEMENT

STRESSING POST TENSION

FOUNDATION MAKE-UP CONVENTIONAL/REBAR

CONSTRUCTION PIERS

REPAIR PIERS

REPAIR PILES

These topics were judged to represent the major types of foundation construction and foundation repairs found in the Houston area. They are certainly not inclusive of every inspection situation or construction method in use, but they do offer a basic set of standards for the majority of inspections which one would encounter typical residential construction. They are also designed to be used by anyone who has some knowledge of foundation construction. It was our intention that they would serve field inspectors, builders, builders supertintendents, municipal inspectors, or anyone with an interest in quality foundations.

The first order of business worked on by the committee ~~was~~ to establish a heading format for each inspection. This portion of the form is meant to establish a context for the inspection. The basics of the site such as, the builder, subdivision, address, lot and block, are all set out at the top of the form. The next section is meant to establish the parameters that will govern the rest of the inspection. The most important of these deals with the plans. No inspection should be undertaken without a set of plans. The context of the plans is established by the name of the engineer, and the date of the plans and the detail sheet. Finally the other pertinent details of the site that are covered in this section are, the date, the time, the weather, and, whether there is a detached garage.

These guidelines were followed on each consectutive form with some variations dictated by the context of the inspection. For the Concrete Placement form there is specific reference to the Foundation Make Up form and the items in need of repair. In the Stress form there is an added reference to the cable count, the pour date, and the Post Tension company. On the Construction Piers form there is a reference to the Soils Engineer, and on the Repair Piers and Repair Piles form there is reference to the design documentation and the municipal permit.

Once the context is established in the heading, the forms move on to sections relating to different aspects of each inspection. In general these sections all are documented by simply checking the item to show it has been correctly completed. In most cases the check serves to show that the item has been considered and it is confirmed. In some sections items must also be answered. Finally the lowerer sections of the forms generally have reference to a drawing of the slab, the piers or piles, or the foundation being repaired.

The drawings further document the conditions specific to the site and the foundation, and allows the inspector to orient the data being described in the conclusion of the inspection.

Each of these forms represents an attempt to document the events related to a specific foundation or repair. It should be remembered that all the answers and data reported are typically the only documentation of what actually happened during the construction of the foundation. For this reason we feel that every item is pertinent and should be given careful consideration during the inspection. Though many of the items listed are fairly common knowledge to the typical inspector or builder, it is the sequencing and nuances of certain questions and items listed, which are the greatest advantage of using the forms. It was felt by the committee, that all the major items: beam size, tendon counts, plan dates etc. were adequately covered in the forms and that if the user answered the questions asked, he would have the foundation that the designer intended. It was also noted that the in most cases the forms could be expanded or made more specific. One example is that the Concrete Placement form calls for an estimated slump on each truck load of concrete. This could be expanded to include actual slump test data taken under the guidelines established by ASTM C94-86b. The same degree of detail could be applied to the site reference regarding drainage and tree removal. Finally it should be noted that the Repair Piers and Repair Piles forms contain information which is not found in any established sources or specifications. Particularly the Repair Piles form. It was generally agreed that no one knows how to inspect these items and to this point they are in certain degree of limbo. Hopefully these forms will establish a precedent for these types of inspections.

In conclusion, it is our hope that the work performed has resulted in a workable set of documents. No doubt the forms could be made more specific and exacting but it is our contention that all the major areas of each type of construction have been covered in a manner sufficient to provide a quality foundation or foundation repair.

# FOUNDATION MAKE-UP
## POST TENSION

Builder ______________________________ Subdivision ___________________________ Date_____

Address ______________________________ Lot___ Blk___ Sec___ Plan site specific yes ___ no_____
Plan No._______ Cable Count ___ Supt. ____________ Engineer _________________ Time _______
Plan Provided at site __Yes __No Weather ____________ Plan Date _____ Detail Date_____
Detached garage Yes No **CHECK IF ITEMS ARE OK** **IF NOT EXPLAIN BELOW**

**SITE**
Subdivision lot _____Other ____________
Explain ______________________________
Fill on site ___Compaction verified by
soils Engineer ________________________
Will make-up drain yes ______no_____
Trees removed _________________________

**FORMS**
________ Are forms secure ?
________ Are floats installed ?
________ Proper clearance at floats
________ Is garage closed in ?

**SLAB**
______ 4" Thickness
Measured ____screeds______stringline_____
Fill material ______________________
______ Level and firm

**TENDONS**
Count: L to R ____ F to B ____Garage _________
Total _____ Variance_____Explain ____________
Number of tendons left on site _____Rebar ____
_____ 1/2" tendons ____ No tendons spaced over 6'-0" _____ 20d nails used ____ stressing ends stripped of plastic not over 1"___cathead in place ____ all intersections tied ____ Dead ends have 3/4" clearance ____ Beam tendons hand tight _____ "S" hook crimped ____ Tendons supported at intersections ____ Double beam tendons draped and secured by # 3 stakes or bricks ____Ample chairs all tied ____Sheathing taped at dead ends and tears _____
Tendon grid is rigid ____YES ____NO

**BEAMS**
_____ Design depth ________in.
Actual depth ________in.
_____ Design width ________in.
Actual width ________in.
_____ Average depth into undisturbed soil or fill per detail sheet
_____ Clean of soil & debris
_____ Water ____ Depth ____ in.
Will water drain __________
_____ Is plumbing obstructing a significant section of the beam __________

**MOISTURE BARRIER**
____ 6 mil. Lapped and seams taped
____ Seated in the bottom of beams
____ Mastic/tape applied at plumbing

**REINFORCING STEEL**
____ Mesh: Size _____Roll____Sheet___ **or** # 3 Rebar at ___ On center bothways Chairs___
____ All mesh seams lapped 6" ____ All edges 2" from the forms_______
____ Rebar:grade ____ 3" bottom & sides, 1½" top ____ splices lapped 36 dia.______
____ corner bars installed Extra rebar installed in slab ____________________
____ rebar cage at bay windows or offsets ________________ stirrups size ________
____ # 3 rebar corner bars installed ___ number of corners _____
____ Exterior beams __________________________ # 3 stirrups size" ___at __" centers
____ Interior beams __________________________ # 3 sturrups size" ___at __" centers
____ Diagonals / 2 #_______ x 5'-0" at inside corner, Number of corners _______
____ Nose Bars at ____________ Construction joints __________ Dowels ______________

**IS THE FOUNDATION READY FOR CONCRETE PLACEMENT** **YES** ____ **NO** _____
**NEEDED CHANGES :** ___________________________________
____________________________________________________
____________________________________________________
____________________________________________________

______________________________
Inspectors signature

**sketch**

______________________________
Superintendents signature

# CONCRETE PLACEMENT

Builder ______________________________ Subdivision___________________________Date_______

Address _______________________________________________Lot____Blk_____Sec_________
Plan No._______Cable Count ____ Supt. _____________Engineer____________________Time ______
Copy of Foundation Makeup Report provided ____yes ____no Date of copy _______________
Items repaired ___yes ___no Explain: ____________________________________________

**check if ok** **If not explain below**

**SITE**
Subdivision lot _____Other _____________
Explain ______________________________
**Are there obstructions at the site which would prevent access for concrete trucks**
______________________________________

**FORMS**
________ Are forms secure?
________ Are floats installed?
________ Proper clearance at floats?
________ Garage closed in.?

**WEATHER**
Start time ________ estimated finish ____________
Weather conditions _____________________________________________
Will temperature rise above 40 degrees for five hours ___________________________
Fourty eight hour forecast ___________________________________________

**CONCRETE**
Concrete company _____________Batch plant__________________tickets onsite__________
Delivered by truck over what distance________ Was a pump used ____yes _____no
Mix: _______________psi _______________psi "pump mix"
Sack Mix : 4½ __ 5 ___5½ ___ **or** Strength Mix ___yes ____no strength _____________
Additives: ___________________________________ **No Calcium Chloride**
Fly Ash : No more than 20% Type C Yes ____ no ______
Slump as ordered from plant ________ inches, As delivered _________ inches
Explain ____________________________________________________________
Was concrete consolidated by : vibrator ____yes _____no Other ______________
Sequence of pour ________________________________________Concrete temperature___
Number of test cylinders taken ____ Testing Company _____________________
Number of slump tests ____ Testing Company _____________________________

If water is added at the jobsite show the amounts over ten gallons and give a visual estimate of the final slump

| Truck # | Gallons added | Placement Location | Est. Slump |
|---|---|---|---|
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |

Draw a diagram of the slab showing the locations of each load by the truck number
**ADDITIONAL COMMENTS** ____________________________________________________
____________________________________________________________________
____________________________________________________________________

_________________________ Inspectors signature

_________________________ Superintendents signature

# STRESSING
# POST TENSION

Builder ______________________________ Subdivision ___________________________ Date_____

Address ______________________________ Lot__ Blk___ Sec___ Plan site specific yes ___ no___
Plan No.________ Cable Count ___ Supt. ____________ Engineer_________________ Time _________
Plan Provided at site __Yes __No Plan Date ______ Detail Date__________ Pour Date__________

Stress date ______ Partial stress date ______ Post tension company ______________________

**Check if ok** **If not explain below**

_______ Are there any cracks in the surface of the slab? Describe ____________________
Estimate size and locate on the drawing below

_______ Are the tendons painted at the edge of the slab? What is the pre-determined distance between the mark and edge ________

_______ Are the wedges placed in a vertical position ?

_______ Are the wedges seated ?

_______ Are there evidence of gripper marks on the tendon ends

_______ Are the tendons stressed from one end only ___yes ___no
Stressed from two ends ___yes ___no How many tendons ________

_______ Are the elongation measurements correct with regard to the length of the tendons ? ( reference attached chart )

**MULTIPLY THE TENDON LENGTH BY .08**

| | | |
|---|---|---|
| 12 ft = 1 " | 40 ft = 3 1/8" | 70 ft = 5 1/2" |
| 15 ft = 1 1/8" | 45 ft = 3 5/8" | 75 ft = 6 " |
| 20 ft = 1 1/2" | 50 ft = 4 " | 80 ft = 6 3/8" |
| 25 ft = 2 " | 55 ft = 4 3/8" | 85 ft = 6 3/4" |
| 30 ft = 2 3/8" | 60 ft = 4 3/4" | 90 ft = 7 1/8" |
| 35 ft = 2 3/4" | 65 ft = 5 1/4" | 95 ft = 7 5/8" |

**SKETCH** Draw a simple sketch of the foundation configuration noting all tendon locations and any problems which you have observed, particularly blowouts at corners or the garage entry.

_______ Are the tendon ends cut inside the pocket former and are the nails cut?

_______ Are the tendon ends grouted with a non-shrink grout?

______________________________
Inspectors signature

______________________________
Superintendents signature

# FOUNDATION MAKE-UP
## CONVENTIONAL / REBAR

Builder ______________________ Subdivision ______________________ Date______

Address ______________________ Lot___Blk___Sec___Plan site specific yes ___no_____
Plan No.________Supt. ______________ Engineer ______________ Time ________
Plan Provided at site __Yes __No Weather ____________ Plan Date ______Detail Date_____
Detached garage Yes No **CHECK IF ITEMS ARE OK** **IF NOT EXPLAIN BELOW**

**SITE**
Subdivision lot _____Other _____________
Explain ______________________
Fill on site ___Compaction verified ____
Will make-up drain yes ______no_____
Trees removed ______________________

**SLAB**
______ 4" Thickness ________other
Measured ____screeds______stringline_____
Fill material ______________________
______ Level and firm

**MOISTURE BARRIER**
______ 6.mil. Lapped and seams taped
______ Seated in the bottom of beams
______ Mastic/tape applied at plumbing

**FORMS**
________ Are forms secure ?
________ Are floats installed ?
________ Proper clearance at floats?
________ Is garage closed in ?

**BEAMS**
_____Design depth _________in.
_____Actual depth _________in.
_____Design width _________in.
_____Actual width _________in.
_____Average depth into undisturbed soil or fill per detail sheet
_____Clean of soil & debris
_____Water _____Depth _____in.
Will water drain __________
_____Is plumbing obstructing a significant section of the beam ______________

**CONSTRUCTION PIERS:** Number of piers ______ Are pier tops clean of debris ____yes ___no

**REINFORCING STEEL** Grade of steel __________

BEAM SECTIONS
Exterior Beams: Steel size ___ Number top___bottom___ Stirrup size ____spacing ____in.
Interior Beams: Steel size ___ Number top___bottom___ Stirrup size ____spacing ____in.
Extra beam depth: ___yes ___no Additional steel required ______________
Proper Clearance: bottom 3"____sides 1½"____top 1½"____ support system ____________
Continuity: Laps 36 x diameter____yes ___no Corner bars installed ___yes ___no
Rebar clean of mud and excessive rust ____yes ____no

SLAB REINFORCING
Mesh: size _____ Rolls_____ Sheets____ proper laps _____ 2" clearance at forms____
Rebar: size ____grade____spacing ____proper laps _____ 2" clearance at forms_______
Grade Transition Steel: size____ Is continuity maintained ___yes __no Where _______
Exterior Grade Beam Dowels: size_______spacing _______in.

ADDITIONAL REINFORCING
Diagonals: Size _____ Number in slab _____
Fireplace Pads: Size of steel _____ Placement ______________________
Baywindows: Size of steel ____ Placement ______________________
Other Projections: ______________Control Joints ______________
Construction Joints: ______________________
Anchor bolts on site ____yes ____no Size ___________

**IS THE FOUNDATION READY FOR CONCRETE PLACEMENT** **YES** ____ **NO** _____
**NEEDED CHANGES :** ______________________

______________________
Inspectors signature

**sketch**

______________________
Superintendents signature

# CONSTRUCTION PIERS

Builder ______________________________ Subdivision __________________________ Date______

Address _____________________________ Lot___ Blk___ Sec___ Plan site specific yes ___no_____
Plan No.__________ Supt. ____________ Engineer______________ Soils Engineer________________
Plan Provided at site __Yes __No Plan Date ______Detail Date_____ Detached Garage __yes __no
Time___________ Weather at site______________________________________________________

**CHECK IF ITEMS ARE OK** **IF NOT EXPLAIN BELOW**

**SITE**
Subdivision lot _____Other _____________ Explain ______________________________
Fill on site ___Compaction verified ____ Will make-up drain yes ______no_____
Trees removed __yes ___no describe _______________________________________

**PIERS**
Name of drilling company ______________________________
Can drill truck access site ___________________________
Total Number of piers ______
Pier Sizes Shaft ______ Bell Size _____ Total______

| Shaft | Bell Size | Total |
| --- | --- | --- |
| ______ | _____ | ______ |
| ______ | _____ | ______ |
| ______ | _____ | ______ |

Describe method of measuring bell sizes
______________________________________________________
( Bell checking tool is required )
Describe bearing strata ________________________________
______________________________________________________
Was water apparent in pier hole_________________________
How was it dealt with _________________________________
______________________________________________________

**REINFORCING**
Number of pieces of #______ Rebar per pier ________ with
# ______ Stirrups at _______" Rebar Grade ________

**CONCRETE**
Will concrete truck be able to access site ___Yes ___No
Was pump truck used ________________________________
Concrete company _______________ Truck numbers __________ **SKETCH TYPICAL PIER**
Strength of concrete ______________PSI __________ Sack mix
Was concrete placed on the same day as the pier drilling __Yes __No
If not explain ________________________________________

**DRAW A SKETCH OF THE STRUCTURE INDICATING THE PIER PLACEMENT**

**COMMENTS** __________________________________________________________________
______________________________________________________________________________

_____________________________
Inspectors signature

_____________________________
Superintendents signature

# REPAIR PIERS

Client ______________________________ City ______________________________ Date__________
Address ______________________________ Lot___ Blk___ Sec___ Plan ____________ Date_ ________
Reference ________Supt. ____________Design Documentation ______________________________
Municipal Permit No ____________ Date ______ Detached Garage __yes __no
Time__________ Weather at site ______________________________________________

**CHECK IF ITEMS ARE OK** **IF NOT EXPLAIN BELOW**

**SITE**

Subdivision lot _____Other ____________ Explain ______________________________
Fill on site ______________________ Will site drain yes_____no_____
Trees removed __yes ___no describe ______________________________________________

**UNDER PINING**

Name of Repair Contractor ______________________________
Method of repair ______________________________________
Total Number of piers ______
Pier Sizes Shaft ______ Bell Size _____ Depth______
______ _____ ______
______ _____ ______

Describe method of measuring bell sizes
________________________________________________________
( Bell checking tool is required )
Describe bearing strata________________________________
________________________________________________________
Was water in pier hole________________________
How was it dealt with _________________________________
________________________________________________________

**REINFORCING**

Number of pieces of #______ Rebar per pier ________ with
# ______ Stirrups at ______"

**CONCRETE**

Was pump truck used ______________________________
Concrete company ______________ Truck numbers __________ **SKETCH TYPICAL PIER**
Strength of concrete ____________PSI __________ Sack mix
Was concrete placed on the same day as the pier drilling __Yes __No
If not explain ______________________________________

**DRAW A SKETCH OF THE STRUCTURE INDICATING THE PIER PLACEMENT**

**COMMENTS** ______________________________________________________________
______________________________________________________________________

______________________________
Inspectors signature

______________________________
Superintendents signature

# REPAIR PILES

Client ______________________________ City ______________________________ Date__________
Address ______________________________ Lot___Blk___Sec___Plan ____________ Date_ ________
Reference ________ Supt. ____________ Design Documentation ______________________________
Municipal Permit No ____________ Date ______ Detached Garage __yes __no
Time__________ Weather at site ______________________________

**CHECK IF ITEMS ARE OK** **IF NOT EXPLAIN BELOW**

**SITE**

Subdivision lot _____Other _____________ Explain ______________________________
Fill on site ______________________ Will site drain yes______no_____
Trees removed __yes ___no describe ______________________________
Soils visable at the site ______________________________

**PILINGS**

Name of Repair Contractor ______________________________
Method of repair ______________________________
Total Number of pilings _____
Pile Sizes Shaft ______ Typical Depth______ Max Depth ______
______ ______ Min Depth ______
______ ______

Total number of piles observed driven to completion_____
Minimum of five is recommended

Was the pile log available at the site ____yes ____no explain ______________

Were the piles capped immediately upon completion of being driven
to refusal ____yes ____no If no explain ______________________________
______________________________

Is the pile cap horizontal ___ yes ___no If no explain ________________
______________________________

Were the piles driven to completion without interruption ___yes ___no
If no explain ______________________________

What is the method of interlock ______________________________

Were interior piles installed ____yes ____no If so were tunnels
used ? Describe ______________________________

Was jetting required to install piles ____yes ____no

**DRAW A SKETCH OF THE STRUCTURE INDICATING THE PILE PLACEMENT**

**COMMENTS** ______________________________
______________________________

______________________________
Inspectors signature

______________________________
Superintendents signature

# BIOGRAPHY

Jack Spivey is president of J. SPIVEY & ASSOCIATES, INC., a real estate inspection firm serving the greater Houston area. Mr. Spivey is a Licensed Real Estate Inspector and has been in business since 1987. Prior to that, he was employed for nine years, as Vice President of MFI Associates, an engineering firm which primarily provided foundation design and inspection services to tract and multi family builders, in Houston, Dallas-Ft Worth and San Antonio. He was previously employed as the Division Manager for Slab on Grade Foundations with the Prescon Corporation. Prescon was one of the earliest post tension companies in the United States and was extremely influential in the design and implementation of post tension slab on grade foundations. Mr. Spivey was involved in the introduction of these foundations in early days of "tract building" in the Houston area. His firm J. Spivey & Associates, Inc. currently provides field inspection services for a number of tract builders in the Houston area, as well as inspection and foundation repair design. His firm currently employs some twelve field inspectors as well as a Structural Engineer. Mr. Spivey is a 1970 graduate of the University of Texas at Austin.

**CONSTRUCTION ISSUES**

**Building Code Integration**

**Joe Edwards**

# BUILDING CODES

JOSEPH A. EDWARDS
EDWARDS CONSULTING COMPANY
P.O. BOX 741201
HOUSTON, TEXAS 77274-1201
TELEPHONE --- 713-783-2229

PRESENTED AT THE FOUNDATION PERFORMANCE COMMITTEE SYMPOSIUM ON NOVEMBER 5, 1998 BROWN CONVENTION CENTER HOUSTON, TEXAS

## OUTLINE

HISTORY OF BUILDING CODES

BUILDING CODES IN THE UNITED STATES

THE INTERNATIONAL BUILDING CODES

HISTORY OF BUILDING CODES

RECOGNITION OF THE NEED FOR SETTING STANDARDS FOR BUILDING CONSTRUCTION IS DOCUMENTED BACK TO THE CODES OF HAMMURABI 4,000 YEARS AGO AND HAS FREQUENTLY BEEN ACCELERATED BY GREAT FIRES.

2000 BC – CODES OF HAMMURABI – EARLIEST KNOWN CODE.

64 AD - BURNING OF ROME – " URBAN RENEWAL ."

1666 AD – THIRD GREAT FIRE OF LONDON – RESULTED IN GREATER BUILDING CONTROLS IN BRITAIN.

1871 AD - CHICAGO FIRE – INSURANCE COMPANIES INSISTED UPON STRICTER BUILDING LAWS.

LATER, THE NEED FOR SETTING STANDARDS FOR CONSTRUCTION MATERIALS AND TESTING FOR PERFORMANCE RESULTED IN THE FOUNDING OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS (ASCE) IN 1852, THE AMERICAN SOCIETY FOR TESTING MATERIALS (ASTM) IN 1902, THE UNDERWRITERS LABORATORIES (UL), THE NATIONAL BUREAU OF STANDARDS (NBS) AND OTHER SIMILAR AGENCIES.

IN THIS CENTURY, THE OBVIOUS BENEFITS OF UNIFORMITY OF REGULATIONS PROMPTED THE DEVELOPMENT OF MODEL CODE ORGANIZATIONS, SUCH AS :

1905 – NATIONAL BUILDING CODE – NATIONAL BOARD OF FIRE UNDERWRITERS. (NOW AMERICAN INSURANCE ASSOCIATION)

1927 – UNIFORM BUILDING CODE – PACIFIC COAST BUILDING OFFICIALS. (NOW INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS)

1945 – SOUTHERN STANDARD BUILDING CODE – SOUTHERN BUILDING CODES CONGRESS INTERNATIONAL, INC.

1950 – BASIC BUILDING CODE – BUILDING OFFICIALS AND CODE ADMINISTRATORS INTERNATIONAL, INC.

AS DESIGN TECHNOLOGY, MATERIALS AND CONSTRUCTION PRACTICES CHANGE, THE CODES AND STANDARDS ARE REVISED TO REFLECT THE CURRENT "STATE OF THE ART." AT THE SAME TIME, LIFE AND PROPERTY LOSSES, IN MANY CASES, PROMPT AN INCREASE IN PROTECTION THROUGH MORE RESTRICTIVE REGULATIONS UPON THE DEMAND OF THE PUBLIC.

THE LATEST REVISION TO THE BUILDING CODES IS THE JOINT EFFORT BY THE UNIFORM BUILDING CODE, THE SOUTHERN STANDARD BUILDING CODE AND THE BASIC BUILDING CODE IN DEVELOPING THE INTERNATIONAL BUILDING CODES TO BE IMPLEMENTED BY THE YEAR 2000.

**SEWER LEAK ISSUES**

**Subcommittee Report**

**Bob Newman**

# FOUNDATION REPAIR

## Subcommittee Report

Ann Nelson

FOUNDATION PERFORMANCE COMMITTEE

## FOUNDATION REPAIR SUBCOMMITTEE

**1998 STATUS REPORT**

**Ann Nelson**
**Nelson Construction Company**
**and**
**Richard W. Peverley**
**Peverley Engineering, Inc.**

## INTRODUCTION

As a part of the 1996 Foundation Performance Committee Symposium, a paper was presented by Mr. Jim Dutton, of DuWest Foundation Repair Company titled Foundation Repair Techniques. This paper contained a brief description of various methods of foundation repair as were commonly used in the greater Houston area. Shortly thereafter, a Subcommittee was then formed for the purpose of analyzing various methods of affecting foundation repair, quantifying the data acquired during such an analysis, and preparing a report on this subject for distribution to the general public. It was the desire of the Committee that such a document would be useful in dispelling some of the misinformation that is currently being promulgated to the public; both from without and within the industry.

Although the work of this Subcommittee is far from over, there has been sufficient work completed so that an interim report can be filed. Basically, the Subcommittee has prepared its first draft of the description and requirements for those repair systems currently in use in the greater Houston area today. This draft is contained as Attachment 1 herein.

In addition, there is an initial draft of our analysis of drilled concrete pier repair concept, pressed segmented pile repair concepts, and helical pile repair concepts. This work is contained as Attachment 2 herein.

# ATTACHMENT 1

# A SUMMARY OF FOUNDATION REPAIR PROCEDURES

# ATTACHMENT 1

# A SUMMARY OF FOUNDATION REPAIR PROCEDURES

# FOUNDATION PERFORMANCE COMMITTEE SYMPOSIUM

## November 5, 1998

# FOUNDATION REPAIR METHODS

## Sub-Committee Report

Committee Members: Ann Nelson, Jim Dutton,
Larry Modes, Dan Jaggers,
and Mike DeShazer

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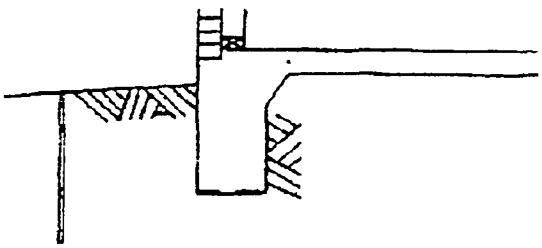


## ROOT SHIELD

An in-ground barrier used where trees or other plants adversely effect foundations.
In expansive soil, trees can cause settlement by reducing soil volume through reduced soil moisture. In nonexpansive soil, trees can cause heave by increasing soil volume through increased root mass.
One type of barrier uses biocides to prevent root hair growth. Another type blocks root hairs from sensing moisture in soil under structure.

### ADVANTAGES

Low cost.
One day to install.
Minor excavated material.
Minimum disturbance of landscaping.
Can increase effectiveness of watering systems in expansive soils.

### DISADVANTAGES

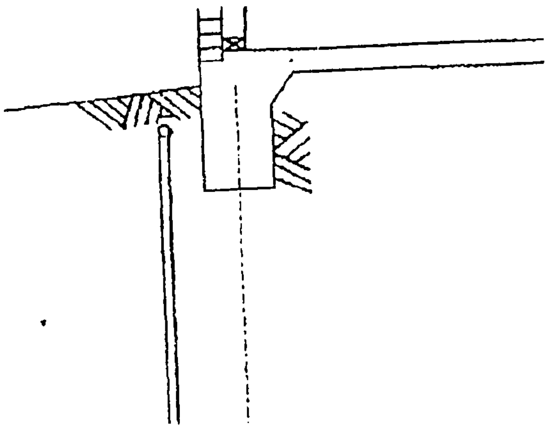
Expansive soil may take several years to recover lost volume.
Foundation rises and falls in expansive soils.
May compromise tree stability.
May compromise tree health.
If roots of tree existed under foundation when it was poured on expansive soils, installation of root shield or removal of tree may cause heave. If roots were severed shortly before foundation was poured, the heave may have occurred anyway.
Material only warranty, generally.
Not a permanent solution.

### BASIC INSTALLATION INSTRUCTIONS

1. Layout line of root shield.
2. Locate all utility lines across root shield.
3. Dig trench.
4. Install shield. Seal around utility lines if moisture type.
5. Backfill trench. Take care to keep top edge above soil if moisture type.
6. Clean up site.

### HOMEOWNERS' INSPECTION

Check that shield is installed in outer half of leaf canopy.
Check that trench is of specified depth.
Check seal around utilities in moisture barrier type.
Check that top of moisture barrier type is above soil.



## PERIMETER WATERING

Injection of water into soil around slab. In expansive soil increased soil moisture expands soil to raise slab. Principally used to increase soil moisture and thereafter to moderate changes in soil moisture under foundations.

### ADVANTAGES

Low to medium cost.
Soil test not needed.
No load calculations needed.
One day or more to install.
Minor excavation needed, generally less than 1 ft.
Moderate changes in soil moisture.

### DISADVANTAGES

Structure rises and falls in expansive soils. In sandy soils may cause settlement by washing out of line binders.
Owner must operate and maintain.
If ever turned off, structure can settle dramatically.
Material only warranty.
Not a permanent solution.

### BASIC INSTALLATION INSTRUCTIONS

1. Install root barrier as needed.
2. Attach control panel to water, electrical systems, and to well.
3. Trench completely around foundation keeping 12" to 18" from foundation.
4. Install pipes and hoses.
5. Test system for leaks.
6. Backfill trench.
7. Clean up site.
8. Postpone repair of cosmetic damage or until foundation becomes stable and structure has time to adjust to new shape of foundation, perhaps several years.

### HOMEOWNERS' INSPECTION

Check that soaker pipe is level.
Check for adequate feeder points.
Check for water-tightness of connections.
Check for back-flow preventer.

## HYDROSTATIC TESTING

A non-pressure test of drain lines under slab. Inflatable bladders can be used to isolate segments of drain to more closely locate any leak.

### ADVANTAGES

Low cost.
One day to perform.
Minor excavated material.
Establishes whether plumbing leak exists prior to leveling.
It provides evidence in support of insurance claim by owner.

### DISADVANTAGES

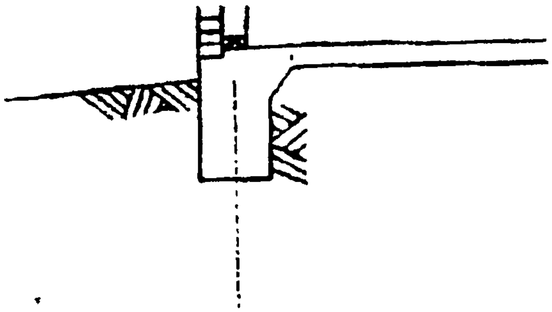
Takes time and may delay start of job.

### BASIC INSTALLATION INSTRUCTIONS

1. Ask occupants to not use water.
2. Locate clean out, or locate line and install clean out.
3. Install standpipe. Insert bladder below clean out.
4. Fill drain line up to a level just below floor. The higher it is the better it tests the system.
5. Mark water line.
6. If water stays at that level for at least 30 minutes, then system has no leak. End of test.
7. If level drops then system has leak. Insert bladders upstream to test segment between bladders.
8. Note rate of water level drop and location of bladders.
9. Remove equipment. Tell occupants test is over.

### HOMEOWNERS' INSPECTION

Watch whether test takes more or less than 30 minutes. If it takes less, then plumber found a leak.
Check that any excavation has been properly backfilled.



## STABILIZERS

Materials that reduce swell/shrink factor or that improve mechanical properties of soil when injected into soil. Methods that require working of soil are not applicable to repair situation.

***LIME***

**ADVANTAGES**

Semi-permanent loss of plasticity in soil.
Needs special handling and equipment.

**DISADVANTAGES**

Soil test needed.
Little migration from point of injection.

***AMMONIUM CHLORIDE***

**ADVANTAGES**

Permanent loss of plasticity in soil.
Needs special handling and equipment.

**DISADVANTAGES**

Soil test needed.
Little migration from point of injection.
Settlement of structure as expansive soils dry.

***SULFONATED OIL***

**ADVANTAGES**

Permanent loss of plasticity in soil.
Good migration from point of injection.

**DISADVANTAGES**

Soil test needed.
Corrosive. Needs special handling and equipment.
Settlement of structure as expansive soils dry.

***ENZYMES***

**ADVANTAGES**

Permanent loss of plasticity in soil.
Good migration from point of injection.
Environmentally harmless.

**DISADVANTAGES**

Soil test needed.
Settlement of structure as expansive soils dry.

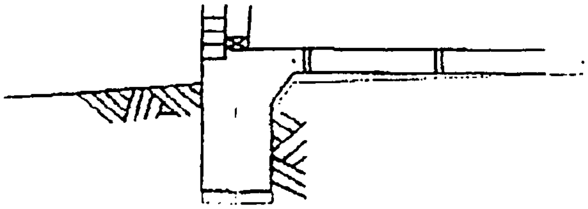
### BASIC INSTALLATION INSTRUCTIONS

The instructions for specific stabilizers may vary greatly from the following.

1. Get soil report to determine injection pattern and depth concentration, and volume.
2. Drill access holes in foundation.
3. Set up any hazardous materials precautions needed.
4. Dilute liquids or mix solids as needed.
5. Pump material into soil.
6. Clean up site.
7. Get soil report to determine effectiveness of treatment.
8. Postpone repair of cosmetic damage or until foundation becomes stable and structure has time to adjust to new shape of foundation, perhaps several years.

### HOMEOWNERS' INSPECTION

Check that holes in floor or foundation are filled.



## FOAM INJECTION

Injection of a urethane-based material into under slab area.

Foam expands with enough pressure to raise slab. The material remains under slab to keep slab from settling to former level.

On expansive soil, principally used for roads, driveways, sidewalks, and foundations that need to stay with surface through the seasons.

On nonexpansive soil, used for all types of foundations.

### ADVANTAGES

Low cost.
Soil test not needed.
Load calculations not needed.
One day or more to install.
Minor excavation needed: generally, less than 1 ft.
Best when used on nonexpansive soil.

### DISADVANTAGES

Holes must be drilled through flooring.
Under slab voids may remain.
Foundation rises and falls in expansive soil.

A. If repair done in dry season, foundation may heave each wet season and return to level each dry season.
B. If repair done in wet season, foundation may settle each dry season, and return to level each wet season.
C. If repair done between dry and wet seasons, foundation may settle a little each dry season and heave a little each wet season, but over all, stays closer to level than A or B.

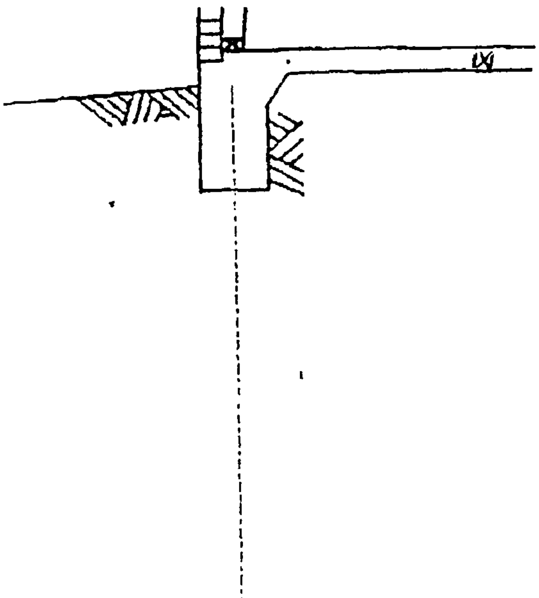
Not uplift resistant.

### BASIC INSTALLATION INSTRUCTIONS

1. Set up injection location grid in area to be raised.
2. Drill injection holes on grid.
3. Inject foam under slab to fill void. Let it set.
4. Inject in short shots, additional foam under slab to raise slab to desired level. Let each set. Do not raise a grid point to level all at once. Raise each grid point a small amount and rotate location of injection (more frequently to low areas, less to near-level areas) until whole area is leveled. Do not over raise as it can cause excessive cosmetic cracking.
5. Patch holes in slab and flooring.
6. Clean up site.
7. Postpone repair of cosmetic damage until foundation becomes stable and structure has time to adjust to new shape of foundation.

### HOMEOWNERS' INSPECTION

Check that holes in floor or foundation are filled.
Check that sewer still drains and is not blocked.



## MUD JACKING

Injection of a cement-based material into under slab area with enough pressure to raise slab. The material remains under slab to keep slab from settling to former level.
On expansive soil, principally used for roads, driveways, sidewalks, and foundations that need to stay with surface or ground through the seasons.
On non-expansive soil, used for all types of foundations.

### ADVANTAGES

Low cost.
Soil test not needed.
Load calculations not needed.
One day or more to install.
Minor excavation needed: generally, less than 1 ft.
Best when used on non-expansive soil.

### DISADVANTAGES

Spills can damage landscaping.
Under-slab voids may remain.
Foundation rises and falls in expansive soil.

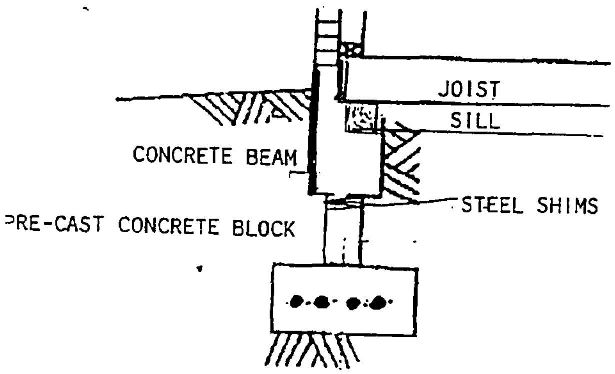
A. If repair done in dry season, foundation may heave each wet season, and return to level each dry season.
B. If repair done in wet season, foundation may settle each dry season, and return to level each wet season
C. If repair done between dry and wet seasons, foundation may settle a little each dry season, and heave a little each wet season, but over all, stays closer to level than A or B.

### BASIC INSTALLATION INSTRUCTIONS

1. Determine injection locations.
2. Drill 1-1/2" to 2" holes through grade beam of slab at injection locations.
3. Mix mud to desired viscosity.
4. Pump mud under slab to fill void or to lift to desired level. Do not over raise as it can cause excessive cosmetic cracking.
5. Patch holes in grade beam, or slab and flooring.
6. Clean up site.
7. Postpone repair of cosmetic damage until foundation becomes stable and structure has time to adjust to new shape of foundation.

### HOMEOWNERS' INSPECTION

Check that holes in floor or foundation are filled.
Check that sewer still drains and is not blocked.




## SPREAD FOOTING

A wide cast-in-place concrete pad placed under footing of structure to reduce pressure on soil. On expansive soils, bottom of spread footing may be several feet below the surface of the active soil, but may still be above a considerable depth of active soil.

### ADVANTAGES

Traditional method.
Low cost.
Interior footings can be installed without going through slab and flooring.
Moderate excavation needed; generally, less than 4 ft.
Depth of footing can be verified.
Placement of steel can be verified.
Best when used on low strength, non-expansive soil.

### DISADVANTAGES

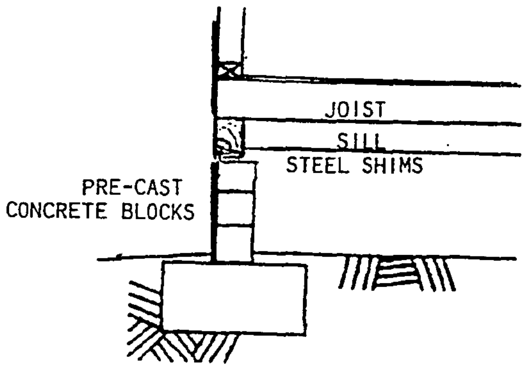
Seven days or more to install.
Settlement needed to develop load bearing capacity.
Footing rises and falls in expansive soils.
Short no-repair-cost warranty (6 mo. to 1 yr.).
On expansive soils reshimming needed frequently (1 to 5 yrs.)

### BASIC INSTALLATION INSTRUCTIONS

1. Dig box with flat bottom.
2. Install steel rebar.
3. Use concrete of specified strength. If none specified, use 3,000 psi.
4. Place jacks and level foundation. Do not over raise as it can cause excessive cosmetic cracking.
5. Backfill with the soil dug out or a similar soil.
6. Patch any excess breakouts, and clean up site.
7. Postpone repair of cosmetic damage at least 90 days or until foundation becomes stable and structure has time to adjust to new shape of foundation.

### HOMEOWNERS' INSPECTION

Check that all loose soil is removed from excavation.
Check that there is rebar on site.
Check thickness of concrete by measuring from bottom of footing to bottom of hole before concrete is poured and later measuring from bottom of footing to top of concrete.
Check to see if wood debris is removed from under house.




## BLOCK & BASE

Floor of structure is held above by wood or concrete blocks placed on a wider base of precast concrete.
In expansive soils, the air space allows the wood and soil under the structure to dry. The soil under the structure will change. It will expand and contract with season changes.

### ADVANTAGES

Traditional method.
Low cost.
One day or more to install.
Interior block & base can be installed without going through flooring.
Minor excavated material: generally, less than 1'.
Best when used on non-expansive soil.

### DISADVANTAGES

Base rises and falls in expansive soils.
Short service agreement, usually 1 - 5 years.
On expansive soils reshimming needed frequently, sometimes seasonally.
Not uplift resistant.

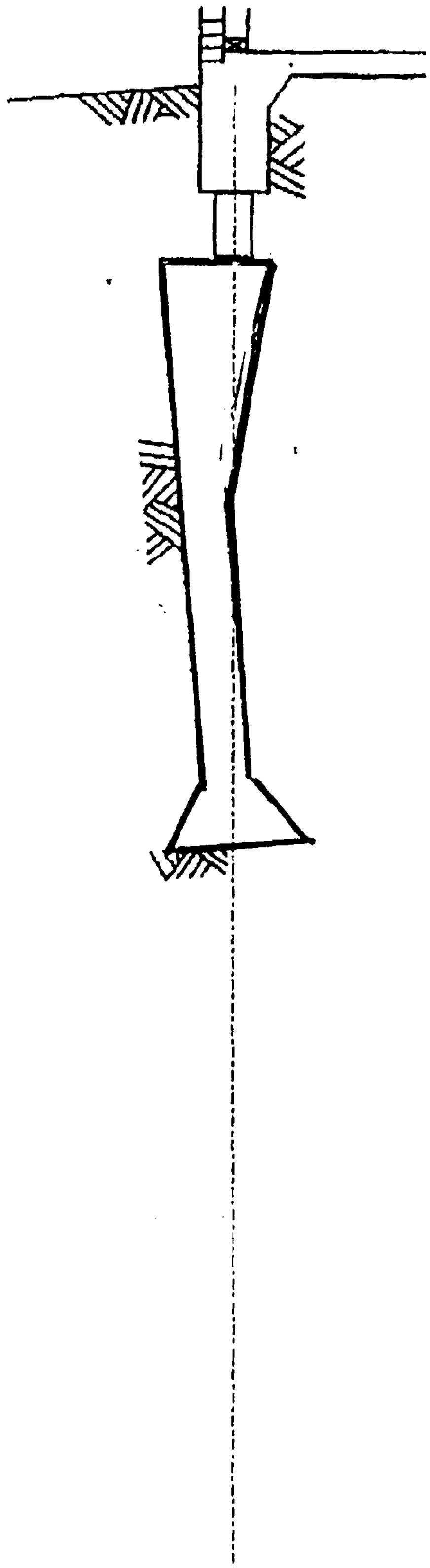
### BASIC INSTALLATION INSTRUCTIONS

1. Set jacks and lift structure.
2. Straighten and level bases. Replace any broken bases. Always remove dirt to level bases.
3. Do not add dirt beneath base to level it. At edge of house dig base a little deeper so part outside house can be covered with soil.
4. Put blocks in center of base.
5. Level structure. Do not over raise as it can cause excessive cosmetic cracking.
6. Cedar shakes or steel plates, as shims or 2" concrete shim blocks.
7. Remove jacks, clean up under structure and around site.
8. Postpone repair of cosmetic damage at least 30 days allowing structure to adjust to new shape of foundation.

### HOMEOWNERS' INSPECTION

Check that there is no loose soil under base pads or left over wood debris under house.

## BELL- BOTTOM PIER



A cast-in-place, deep-seated foundation support in expansive soil. Bottom of pier must be below active layer.

### ADVANTAGES

Traditional method can be adapted/customized to lift and hold unusual problem slabs.
Moderate cost.
Depth and diameter of bell can be verified.
Placement of steel can be verified.
Moderate service agreement (5 - 20 yrs).
Best when depth below the active zone.

### DISADVANTAGES

Soil test to help determine soil characteristic.
Major excavation needed, 12 ft. recommended.
Interior piers need to be installed through slab.
Cannot be in water bearing sand.
Difficult to drill below 15 ft.
Several days cure time.

### BASIC INSTALLATION INSTRUCTIONS

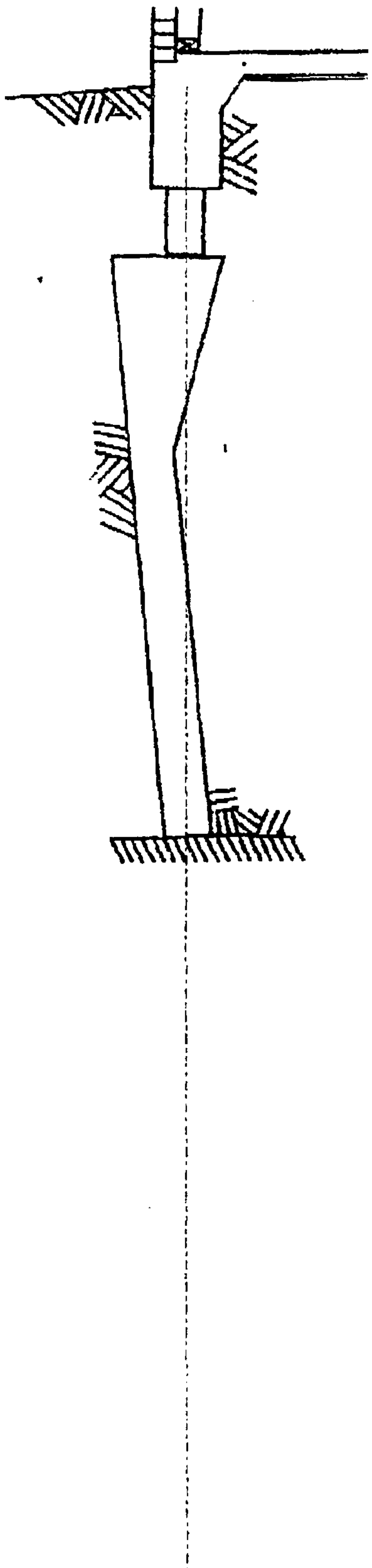
1. Determine spacing for each pier and pier depth.
2. Dig pier cap box with a taper as shown.
3. Drill shaft on 3 degree angle.
4. Drill shaft to proper depth.
5. Under-ream to proper diameter, usually 2.5 to shaft diameter.
6. Install steel rebar and stirrups.
7. Pour concrete same day of drilling to minimize soil sloughing into bell.
8. Observe redi-mix for consistency, ask for batch ticket to verify mix.
9. Use a vibrator to minimize air pockets.
10. Cover pier-cap holes while concrete sets.
11. Wait appropriate time for concrete cure.
12. Place jacks and level foundation. Do no over raise to compensate for initial settlement, as it can cause excessive cosmetic cracking.
13. Place concrete blocks and steel shims.
14. Backfill with soil dug out or similar soil. Don't use sand, it creates a reservoir for water.
15. Patch any breakouts and clean up site.
16. Wait more than 90 days after leveling to repair cosmetic damage.

### HOMEOWNERS' INSPECTION

Observe for a change in soil color, texture or other observable difference at bottom of pier, then check spoils pile for that difference.
Check that pier spacing, layout and depth match plan.
Check that there is rebar on site and in the pier holes.
Obtain copy of batch ticket for redi-mix.

Note: The contractors that have installed 12"/30" piers to a minimum 12' depth have no knowledge of any warranty calls.

## STRAIGHT-SHAFT PIER



A cast-in-place, deep-seated foundation support in expansive soil. Bottom of pier must be below active layer.

### ADVANTAGES

Traditional method can be adapted/customized to lift and hold unusual problem slabs.
Moderate cost.
Placement of steel can be verified.
Moderate service agreement (5 - 20 yrs).
Best when depth below the active zone.

### DISADVANTAGES

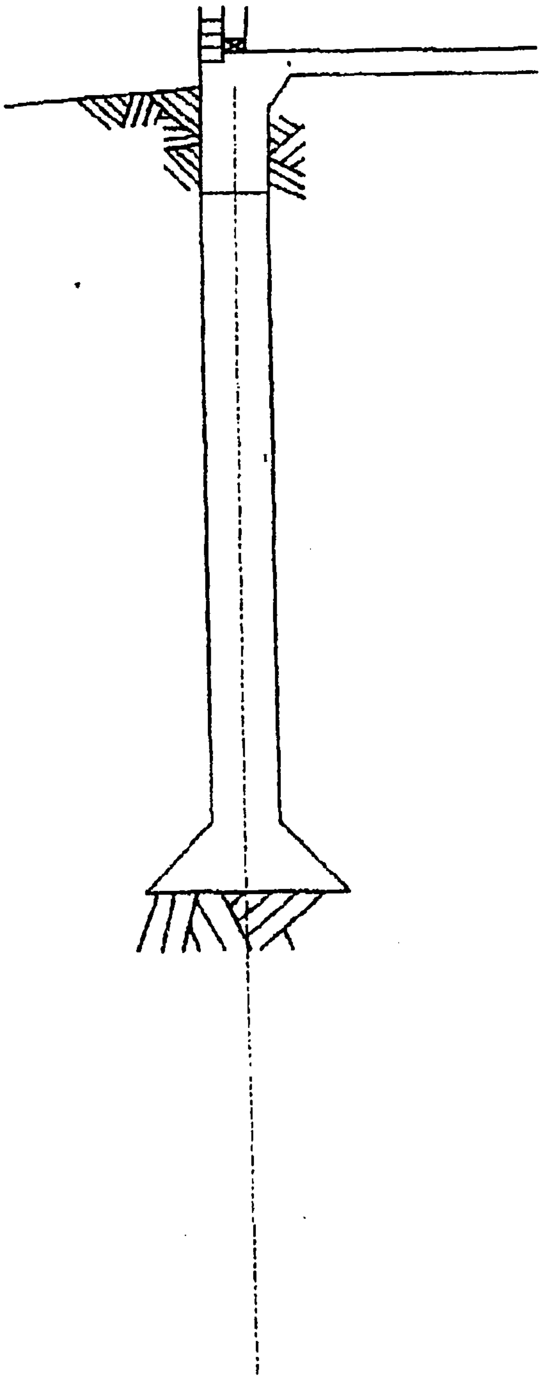
Soil test to help determine soil characteristic.
Major excavation needed, 12 ft. recommended.
Interior piers need to be installed through slab.
Cannot be in water bearing sand.
Difficult to drill below 15 ft.
Several days cure time.

### BASIC INSTALLATION INSTRUCTIONS

1. Determine spacing for each pier and pier depth.
2. Dig pier cap box with a taper as shown.
3. Drill shaft on 3 degree angle.
4. Drill shaft to proper depth.
5. Install steel rebar and stirrups.
6. Pour concrete same day of drilling.
7. Observe redi-mix for consistency, ask for batch ticket to verify mix.
8. Use a vibrator to minimize air pockets.
9. Cover pier-cap holes while concrete sets.
10. Wait appropriate time for concrete cure.
11. Place jacks and level foundation. Do no over raise to compensate for initial settlement, as it can cause excessive cracking.
12. Place concrete blocks and steel shims.
13. Backfill with soil dug out or similar soil. Don't use sand, it creates a reservoir for water.
14. Patch any breakouts and clean up site.
15. Wait more than 90 days after leveling to repair cosmetic damage.

### HOMEOWNERS' INSPECTION

Observe for a change in soil color, texture or other observable difference at bottom of pier, then check spoils pile for that difference.
Check that pier spacing, layout and depth match plan.
Check that there is rebar on site and in the pier holes.
Obtain copy of batch ticket for redi-mix.



## BUILDER'S PIER

A cost-in-place bell-bottom pier installed as part of original construction.
The concern here is repair of a foundation where builder's piers are rigidly attached to grade beam by rebar. Some of these concerns apply to builder's piers that abut grade beam without attachment.
The alternative methods used in repair of a slab with builder's piers are:

1) To cut pier from slab.
2) To leave it attached.

*TO CUT LOOSE?*

ADVANTAGES

In areas of settlement, builder's pier has shown itself inadequate to support slab.
Reduced stress in slab if builder's pier cut loose in areas where slab is lifted.
When soil is expanded, weight and uplift resistance of builder's pier may increase stress in slab.

DISADVANTAGES

Additional labor costs. Builder's pier usually located where it must be excavated, shortened, and backfilled on its own, and not able to share a hole with a remedial pier or pile.
Builder's pier must be shortened by max PVR at surface.

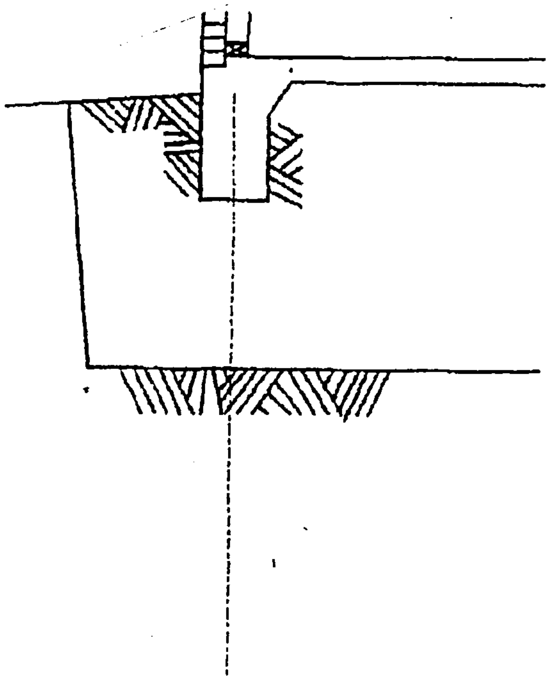
*OR NOT TO CUT LOOSE?*

ADVANTAGES

Less tress in slab if builder's pier not cut loose in areas where slab is pinned or not raised.
Less time and effort.

DISADVANTAGES

May increase stress in areas where slab is raised.
May increase loading of remedial pier or pile during periods of soil expansion.



## TUNNELING

Principally used where interior piles are required. This is an alternative method to breaking a 2 - 3 sq. ft. hole in slab inside home at each interior location of a pier or pile to be installed.

### ADVANTAGES

Possible less costly to homeowner considering there is no unstated additional cost of repair or replacement of flooring no interior cleaning after contractor leaves.
Homeowner can remain in home in relatively normal comfort and routine during job.
No worker or equipment inside home.
No dirt or wet concrete hauled through home.
No need to install protection on steps or floors nor at doorjambs or walls along work routes.
No need to install dust protection in rooms where a pier is installed.
No damage to computer hard drives, or audio or video equipment from abrasive dust.
No weakening of post tension slab by damage to or cutting of post-tension tendons.
No weakening of reinforced slab by cutting deformed reinforcing bars or inadequate overlap within repair patch.

### DISADVANTAGES

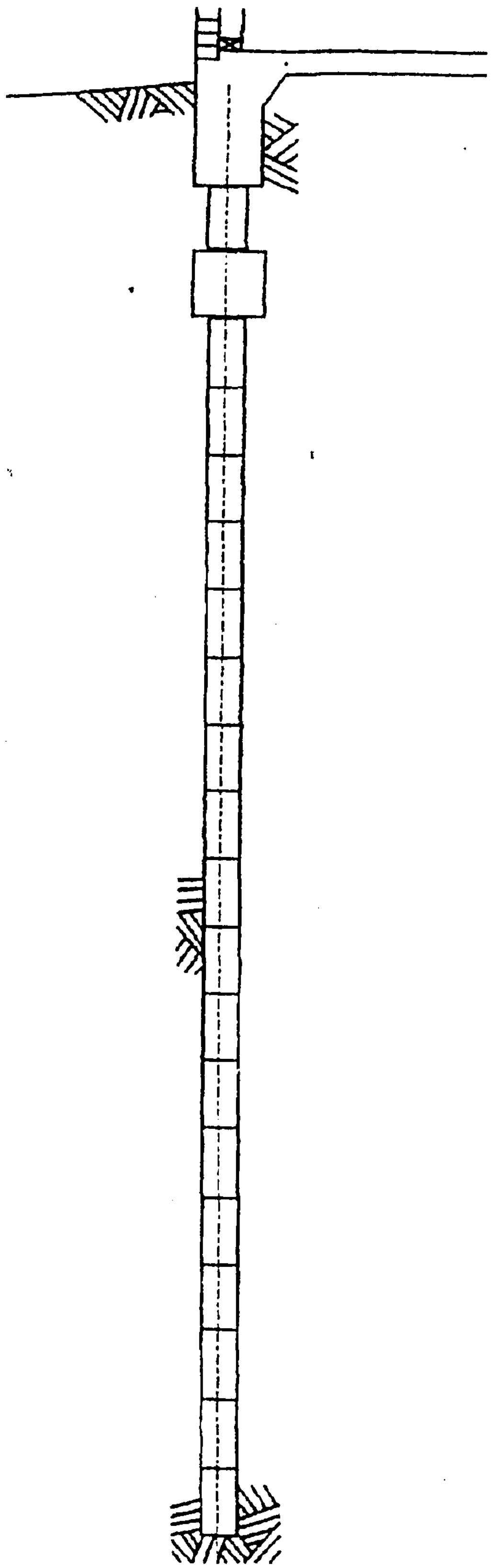
More costly to contractor than going through slab.
Cannot be used for bell-bottom pier, cast-in-place pier, steel pipe pile, nor steel screw pier.
Need for positive ventilation.
Extra effort needed to get material and equipment through tunnel to pile site.
Cramped work areas.
Difficult communication to pile site.
Must properly backfill tunnel to prevent existing soil settlement that may cause plumbing problems and foundation settlement in areas beyond repairs.
Must retunnel to perform reshim work.

### BASIC INSTALLATION INSTRUCTIONS

1. Lay out proposed tunnels to minimize distance from adit to head. Use multiple tunnels and keep them short.
2. Follow OSHA and other applicable law.
3. Don't let spoil accumulate in tunnel.
4. If hole structure on piles, backfill with removed material. Otherwise, backfill with a nonexpansive material to prevent settlement of undisturbed material adjacent to tunnel.
5. Clean up site.

### HOMEOWNERS' INSPECTION

Check for fresh air fans on tunnels more than 10' deep.
Check that nothing projects from tunnel and that backfill is adequate to prevent a tunnel entrance.



## SOLID PRECAST PILE

A solid, unreinforced, segmented pile driven to depth using weight of structure as resistance.

### ADVANTAGES

Moderate cost.
One day or more to install.
Soil test not needed.
Load calculations not needed.
Moderate excavation needed (Generally, less than 4 ft.).
Can be installed below water-table.
Not eccentrically loaded. No steel required.
Interior piles can be installed without going through slab via tunnel.
Greater depths under heavier parts of structure.
(Record depth of 63 ft. in Houston area.)
Pile driven to refusal at lower sliding friction coefficient, but performs at higher static friction coefficient.
Load bearing capacity develops without settlement.
Soil moisture does not adversely affect performance.
Long no-repair-cost warranty. (10 yrs to life of structure.)
Best when used on expansive soil.
Reshimming needed infrequently (sometimes never).

### DISADVANTAGES

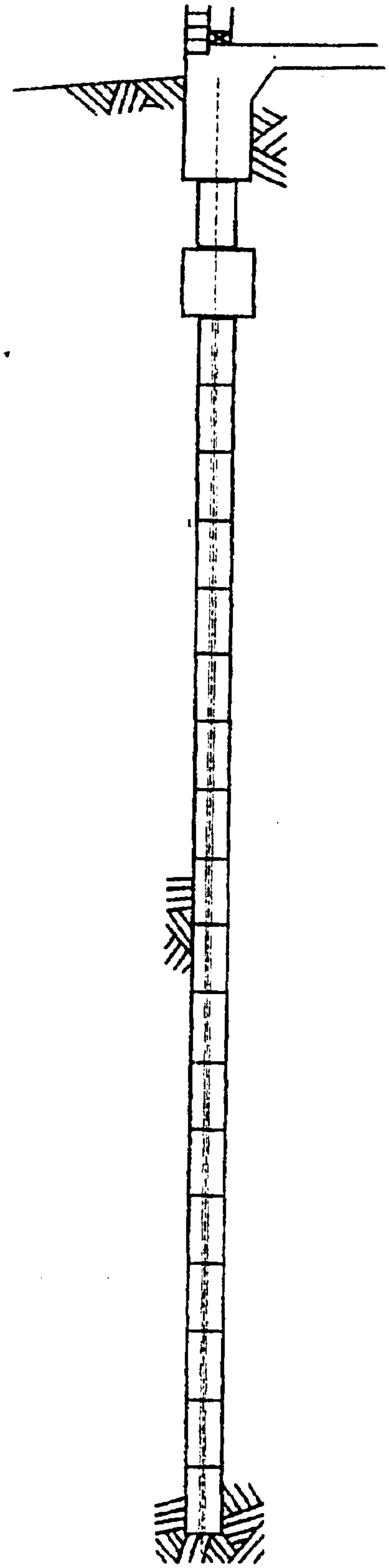
Depth of pile cannot be verified.
May lose strength if springback occurs.
If rectangular section used, there is additional driving resistance due to rotational effects.
Difficult to install in firm sand.
Difficult to install in rocky soil.
Not uplift resistant.

### BASIC INSTALLATION INSTRUCTIONS

1. Order pile segments of specified strength before job starts, if none specified use 3,000 psi.
2. Test at least one cylinder from each pallet. If of specified strength, use them. If not, have manufacturer replace cylinders.
3. Dig drive pit with flat bottom under beam.
4. Press one pile at a time to avoid load sharing.
5. Keep pile slipping through soil until refusal. Do not let it seize in static friction.
6. Temporarily cap and shim pile to prevent springback.
7. When piles are capped, place jacks and level foundation. Do not over lift as it can cause excessive cosmetic cracking.
8. Back fill with soil dug out or a similar soil.
9. Patch any excess breakouts, and clean up site.
10. Wait more than 90 days after leveling to give structure time to adjust to new shape of foundation before repairing cosmetic damage.

### HOMEOWNERS' INSPECTION

Check that pile spacing matches plan.
Ask for concrete test reports and do not let contractor proceed unless test psi exceeds spec. psi.



## REINFORCED PRECAST PILE

A Hollow center, steel reinforced, segmented pile driven to depth using weight of structure as resistance.

### ADVANTAGES

Moderate cost.
One day or more to install.
Soil test not needed.
Load calculations not needed.
Can be installed below water-table.
Not eccentrically loaded. No steel required.
Interior piles can be installed without going through slab.
Greater depths under heavier parts of structure.
Moderate excavation needed, generally less than 4 ft.
Depth of pile can be verified.
Placement of steel can be verified.
Pile driven to refusal at lower sliding friction coefficient, but performs at higher static friction coefficient.
Load bearing capacity develops without settlement.
Soil moisture doesn't adversely affect performance.
Long no-repair-cost warranty. (10 yrs to life-of-structure)
Best when used on expansive soil.
Reshimming needed infrequently, sometimes never.

### DISADVANTAGES

May loose strength if springback allowed.
Difficult to install in firm sand without water-jetting.
Difficult to install in rocky soil.

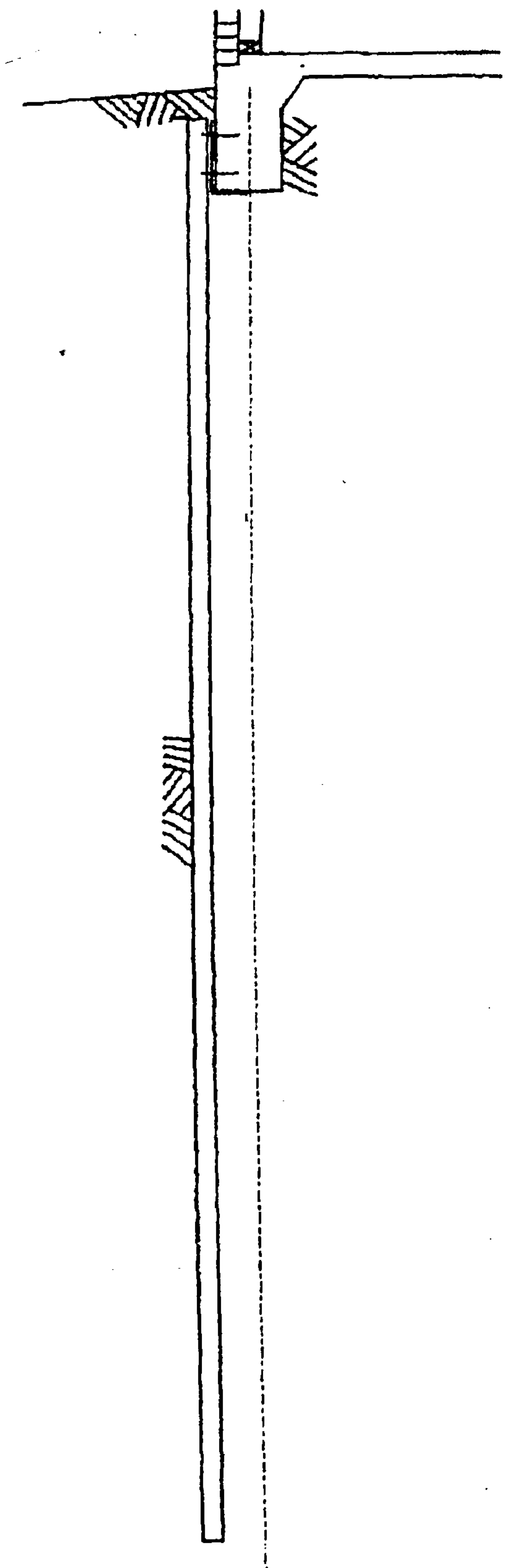
### BASIC INSTALLATION INSTRUCTIONS

1. Order pile segments of specified strength before job starts, if none specified use 3,000 psi.
2. Test at least one cylinder from each pallet. If of specified strength, use cylinders. If not, have manufacturer replace cylinders.
3. Dig drive pit with flat bottom under beam.
4. Press one pile at a time to avoid load sharing.
   A. For Cable Lock®: place starter cylinder then thread cable through central hole of subsequent cylinders.
   B. For Ultra Pile® : water jetting can be used to get additional depth.
5. Keep pile slipping through soil until refusal. Don't let it seize in static friction.
6. Fro Cable Lock® epoxy cable end, or for Ultra Pile® grout in rebar.
7. Temporarily cap and shim pile to prevent springback.
8. When all piles are capped, place jacks and level foundation. Don't over raise as it can cause excessive cosmetic cracking.
9. Backfill with soil dug out or a similar soil.
10. Patch any excess breakouts and clean up site.
11. Wait more than 90 days after leveling, to give structure time to adjust to new shape of foundation before repairing cosmetic damage.

## HOMEOWNERS' SPECIFICATIONS

Check that pile spacing matches plan.
Ask for concrete test reports and do not let contractor proceed unless test psi exceeds spec. psi.
Verify depth of pile. (Only two brands can be verified for depth.)

A. For Cable Lock®: Mark cables with your own mark before driving and return to job before contractor cuts off cable.
B. For Ultra Pile®: Have all piles driven, but no rebar installed until you return to job to verify length of rebar going into pile.



## STEEL PIPE PILE

A steel pipe pile driven to depth using weight of structure as resistance.

### ADVANTAGES

One day or more to install.
Soil test not needed.
Load calculations not needed.
Pile is driven to bedrock or to refusal at lower sliding friction coefficient, but performs at higher static friction coefficient.
Minor excavation needed: generally, less than 4 ft.
Depth of pile can be verified.
Load bearing capacity develops without settlement.
Can be installed below water-table.
Soil moisture does not adversely effect performance.
Greater depth under heavier parts of structure.
Uplift resistance capable.
Moderate duration no-repair-cost warranty.
Best when used where bedrock can be reached.
Adjustment needed infrequently. ( Sometimes never where driven to bedrock.)

### DISADVANTAGES

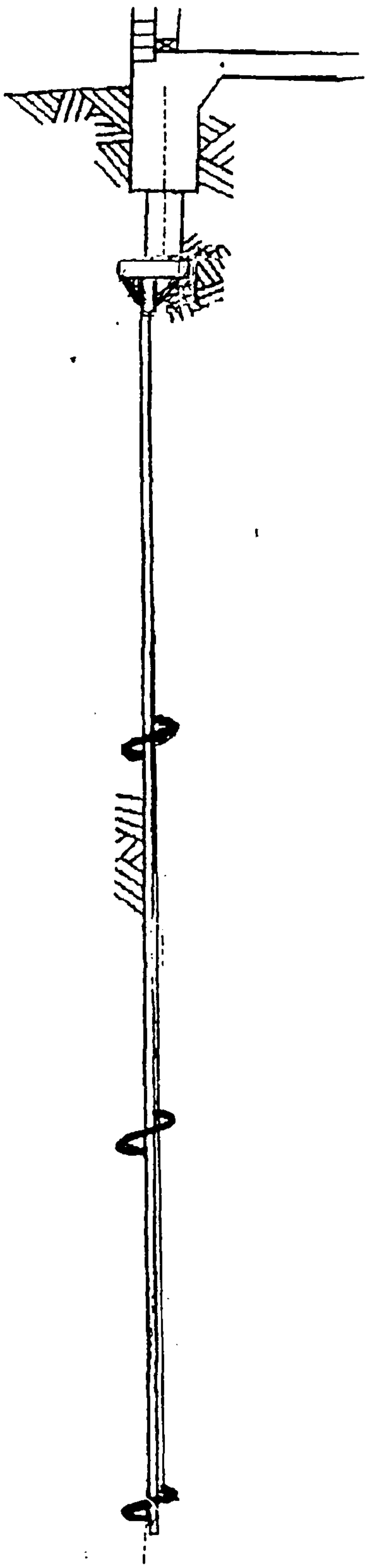
High cost.
May lose strength if springback allowed.
Eccentrically loaded.
Interior piers need to be installed through slab and flooring.

### BASIC INSTALLATION INSTRUCTIONS

1. Dig drive pit with flat bottom under beam.
2. Drive pipe shaft at a slight angle, keep it as straight and vertical as possible.
3. Bolt bracket to side of grade beam.
4. Cut off excess pipe.
5. Press one pile at a time to avoid load sharing.
6. Keep pile slipping through soil until refusal. Do not let it seize in static friction.
7. Clamp bracket to prevent springback. Where specified, attach bracket to foundation for uplift resistance.
8. When all piles are bracketed, place jacks and level foundation. Do not over raise as it can cause excessive cosmetic cracking.
9. Backfill with soil dug out or similar soil.
10. Patch any access breakouts, and clean up site.
11. Wait more than 90 days after leveling, to give structure time to adjust to new shape of foundation, before repairing cosmetic damage.

### HOMEOWNERS' INSPECTION

Check spacing.
Check undriven pipe for signs of excess corrosion, particularly on interior, reject all with significant pitting.



## HELICAL PIER®

A solid steel square shaft with helix plates attached at intervals. When the shaft is rotated, it screws into the soil.

### ADVANTAGES

One day or more to install.
Moderate excavation needed: generally, less than 3 ft.
Can be installed below watertable.
Soil moisture doesn't adversely affect performance.
Seismic and uplift resistance capable.
Uplift load bearing capacity develops without initial movement.
Can be used in most soil conditions.
Depth of pier can be verified.
Can be used on slopes and tie-down situations.
Pre-engineered product with 80 yr. reputation.
No-repair-cost warranty. 5 - 20 yrs.
Ultimate load capacity of pier is pre-determined by ft. lbs. of torque developed during installation.

### DISADVANTAGES

Live and dead loads must be calculated.
Installation of each pier requires monitoring as it goes into the ground.
Each foot of shaft is measured for applied foot pounds of torque.
Interior piers need to be installed through slab.

### BASIC INSTALLATION INSTRUCTION

1. Dig drive pit beside foundation.
2. Chip out footing to accommodate bracket if 2 bolt bracket is needed.
3. Position installer equipment and torque bar.
4. Drive pier into ground to desired torque.
5. Record gauge readings below 5'.
6. Keep shaft on 3 degree angle.
7. Trim shaft to proper height.
8. Place grout if specified, and install bracket.
9. Place jacks and lift foundation.
10. Backfill with soil dug out or a similar soil.
11. Patch any breakouts, and clean up site.
12. Wait more than 90 days after leveling, to give structure time to adjust to new shape of foundation before repairing cosmetic damage.

### HOMEOWNERS' INSPECTION

Check pier spacing.
Check torque records.

# ATTACHMENT 1

# A SUMMARY OF FOUNDATION REPAIR PROCEDURES

# ATTACHMENT 2

## AN ANALYSIS OF FOUNDATION REPAIRS USING
## DRILLED PIERS
## PRESSED SEGMENTED PILES
## AND
## HELICAL PIERS

## ATTACHMENT 2

## AN ANALYSIS OF FOUNDATION REPAIRS USING
## DRILLED PIERS
## PRESSED SEGMENTED PILES
## AND
## HELICAL PIERS

# PRELIMINARY DRAFT REPORT

**AN ANALYSIS OF FOUNDATION REPAIRS USING**
**DRILLED PIERS**
**PRESSED SEGMENTED PILES**
**AND**
**HELICAL PIERS**
**BY**

**Richard W. Peverley, PE**
**Peverley Engineering, Inc.**
**and**
**Ann Nelson**
**Nelson Construction Company**

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## HISTORICAL INFORMATION

Prior to the beginning of World War II, a majority of the residential buildings were founded on what was often referred to as pier-and-beam type of foundations. Today, they are often referred to as block-and-base foundations. Concrete slab-on-ground foundations came into general use at the beginning of the building boom following World War II. A spin-off of the military construction effort during the War was the development of concrete slab construction technology. Unfortunately, there was little or no effort made to understand the techniques required to produce satisfactory foundation performance in the unique geological and meteorological environments of the greater Houston area. During the 1950/1960 time period, the Federal Housing Authority sponsored a slab-on-ground design study which was conducted by the Building Research Advisory Board, Division of Engineering, National Research Council, National Academy of Sciences. Their final report was issued in 1968 under the title Criteria for Selection and Design of Residential Slabs-on-Ground.[Ref. 1] The report was frequently referred to as BRAB No. 33. The building industry was noticeably less than enthusiastic about the recommendations contained in this report. As a result, little was done to improve the design of concrete slabs-on-ground and the failure rate of this type of foundation, particularly in the greater Houston area, steadily increased. The next FHA sponsored study was conducted by the University of Texas at Arlington in the 1996/1970 area.[Ref. 2] An entire subdivision in the Dallas area had been purchased for the construction of a highway, whose construction was delayed. Studies were conducted using these abandoned homes. Although this study did produce some interesting data, the results had little effect on the home building business.

A majority of the Houston homes that are currently in the thirty to forty year age group, that are founded on expansive clay soils, were constructed in farm land areas where there was a sparsity of growing trees.[Ref. 3] A typical homeowner would then plant trees in the yard, often within ten to twenty feet of the edge of the foundation.[Ref 4] A significant number of these trees were extremely hearty varieties such as Oaks, Tallows, and Pecans, whose water demands could be significant. During the first ten years of growth, the water demands of the trees were somewhat limited and the trees and the buildings generally lived on the same site in harmony. As the trees continued to grow, however, their water demands increased steadily as their leaf areas increased, resulting in the suction of soil moisture from under the foundation. A small void area then appeared between the bottom of the foundation slab and the top of the soil which, in turn, provided an oxygen source which then allowed tree root fibers to grow under the foundation slab. Experience has shown that tree roots can be found under the entire area of a foundation slab.[Ref. 5] It has been stated that the roots of a tree may extend as far beyond their trunk as 2 to 3 times the radius of their crown (or drip line).[Ref. 6 & 7] As a result, many such foundations failed and were repaired. Until the early 1980's, such repairs were made using drilled piers which were sunk to depths as shallow as 8 to 10 feet and whose bells (if indeed any bells existed) were as narrow as 21 inches. During the years 1988 through 1990, the greater Houston area suffered the worst drought that had been seen in 50 years and many foundations supported on such shallow piers failed.[Ref. 8]

In the early 1980's, the concept of segmented pressed pilings appeared. Despite their many detractors, the performance of the pilings far exceeded their pier counterparts.[Ref. 9] The primary reason was that these pilings had been driven to depths below the surface in excess of what could be achieved using drilled piers. Data were later published, which showed that trees can lower the active zone of the soil to depths as deep as 12 to 15 feet.[Ref 10] Thus, the importance of depth has perhaps become one of the most essential factors in foundation repair. This importance can perhaps be best demonstrated by two repair jobs in the First colony subdivision where on one home, helical piers were place at depth in excess of 24 feet[Ref.11] and pressed piling, on another home, to depths of as much as 34 feet.[Ref. 12]

## UNDERPINNING REQUIREMENTS

The following is a summary of the engineering requirements which may be recommended for the restoration of slabs-on-ground foundations (some of which may be unrealistic at this time):

HOME OWNER EXPECTATIONS
- o Identify the owners major complaints and their expectations?
- o Thoroughly brief the home owners on what the realistic expectations may be?
- o The homeowner will be informed on the need of their presence during the foundation raising.

FOUNDATION CONDITIONS
- o The relative heights of the interior floors shall be measured.
- o Measure the relative levelness of door tops, window sills, counter tops, mantels, etc parallel the surface of the floors?
- o If they are parallel to the floors, how much lifting is required to achieve a reasonable state of levelness to bring the interior floors into a level position?
- o If they are not, how much lifting is required to bring these surfaces into a level position?
- o Advise the homeowner of the foregoing conditions.
- o Determine the bending moment resistance in the been calculated and the allowable span been determined. Note that the depth and width of the beam shall be measured. If the reinforcement configuration is not known, the bending calculations shall be done assuming the beam to be unreinforced).

  NOTE: Such calculations have been made by members of this firm using, as a model, a beam with a 20" depth, 12" width, and 2 #4 bars top & bottom which show that 7 pier/pile spacing is safe. Copies of this calculation will be included in the next issuance of this document.
- o Provide the homeowner with an estimation of the cost of cosmetic repairs.
- o Have the repair plans Sealed by a Licensed Engineer.

SOILS DETERMINATION

- o Have soil tests conducted by a soils testing firm?
- o From the soils tests, identify the following:
  - What is the soil type in the area in which this residence is located?
  - What is the minimum and maximum depth of the active zone?
  - What is the strength of the soils at the bottom of the active zone?
  - Are there any perched water table conditions on this site.
  - Have water table condition been encountered.

UNDERPINNING PLACEMENT

- o Are all plans and specifications in place?
- o Have the required permits been obtained?
- o Have all installations been inspected by an independent inspector?
- o Have any problem areas been identified and corrective measures taken?
- o Have the flbor heights been measured after the lifting has been completed?
- o Has all lifting damage been identified?

POST-REPAIR REQUIREMENTS

- o Measure the relative heights of the interior floors after the lifting is done.
- o Identify all lifting damage which has occurred.
- o Conduct a post-repair conference with the homeowners and identify any responsibilities they may have.

## FOUNDATION UNDERPINNING OPTIONS

### DRILLED PIERS

Drilled, bell bottom piers (which will hereinafter be referred to simply as piers) have been used for the repair of residential slab-on-ground foundation in the greater Houston area since shortly after World War II.[Ref. 13] The use of such piers for the support of a variety of foundations has a long history. Drilled piers were used early in this century and were generally hand-excavated. Horse driven rotary machines were used in Texas as early as 1920[Ref. 14] in 1920 in the San Antonio Area. After World War 2, more powerful machines became available and the use of drilled piers became more wide-spread. Extensive research was carried out in the 1960 and 1970 time period [Ref. 15] and the use of drilled piers became more prevalent. With regard to their use in the repair of residential foundations, the local office of the Federal Housing Authority developed a specification for what they called Type 2 drilled piers. A modified copy of this specification is contained in Attachment A. No one seems to know what Type 1 piers were. For years, a typical drilled pier configuration was one with a 16 x 24 inch box at the top, an 8 inch shaft, ans a 21 inch maximum diameter bell. Better equipment has become available and 30 inch diameter bells on 12 inch shafts are currently available. Some examples of hybrid piers are also attached.

Bell bottom piers can be utilized in cohesive soils where the water table conditions are such that the concrete will not be place in the bottom of piers through water where cement/aggregate segregation can occur. A classic pier configuration is shown in Figure 1.[Ref. 16] The capacity of the pier is designated by the term $Q_u$. The ultimate capacity of the pier is the result of the tip resistance $Q_t$ and the shaft resistance $Q_s$. Thus,

$$Q_u = Q_t + Q_s$$

Reference is made to Figure 1

in a cohesive soil, the following applies:

$$Q_u = (c_t N_c A_t) + (c_s A_s)$$

where:

$c_t$ = undrained soil strength under the bell

$N_c$ = Bearing capacity of the soil

$A_t$ = The area of the bell

$c_s$ = Undrained adhesion between the pier shaft & the soil

$A_s$ = Periphery area of the shaft

In a cohesive soil, the term $N_c = 9 \times w$, where w = a dimensionless coefficient.

In terms of s foundation repair pier, the term $c_s A_s$ can generally be ignored since the bottom of the piers is seldom below 10 feet and even under ideal conditions, much of the shaft is within the active zone where soil shrinkage can result in the movement of the soil away from the shaft. Thus, it is essential that the strength of the soil be verifies to assure that the pier will perform effectively.

In the years between 1950 and 1970, it was common to place the bottom of repair piers at a depth of 8 to 10 feet below the surface of the soil. This criteria was satisfactory until the occurance of the drought of 1988 to 1990, particularly in areas of Southwest Houston where the presence of large trees was prevalent. Soil data have shown that in the presence of trees, the active zone in the soil can be as low as 10 to 12 feet.[Ref. 17] In the opinion of geotechnical engineers, the depth of any repair pier or pile must be at a depth of somewhere between 16 to 20 feet below the surface.[Ref. 18]

Drilled pier can be effectively used for foundation repair provided the following criteria can be met:

- The depth of the bottom of the pier must be below the bottom of the active zone during the driest time of the year.
- The bottom of the pier must be of a proper diameter to distribute the weight such that the weight of the pier under the most severe loading conditions is substantially below the allowable bearing strength of the soil.
- The soil strength must be known to assure that the piers will be supported without settlement.

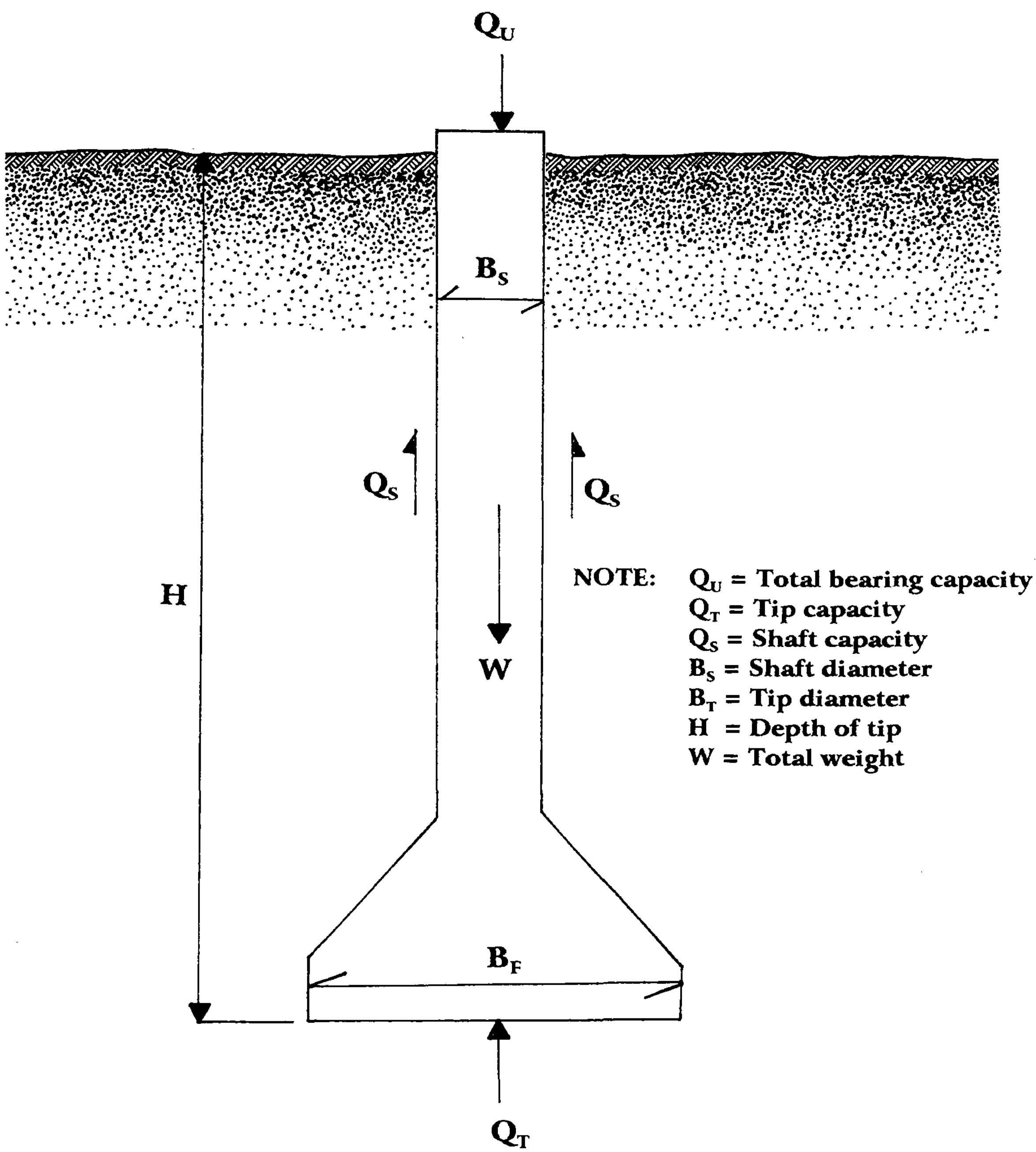
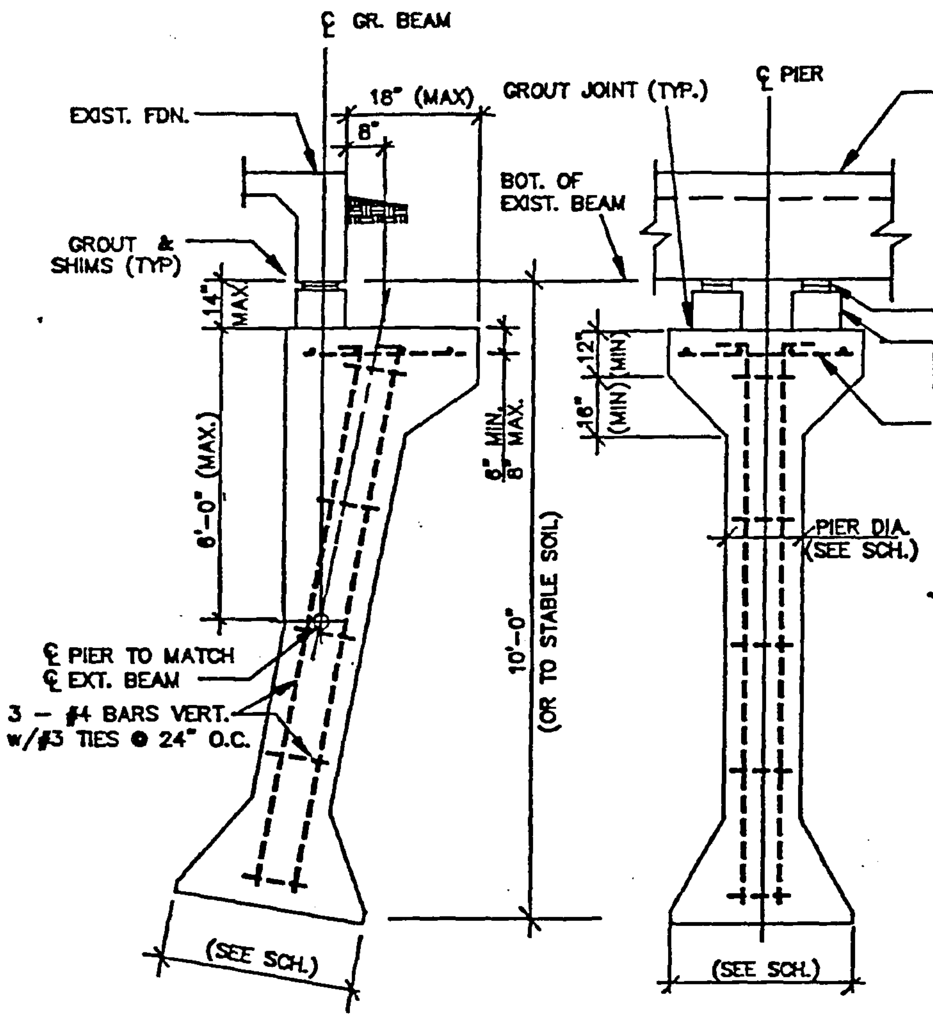
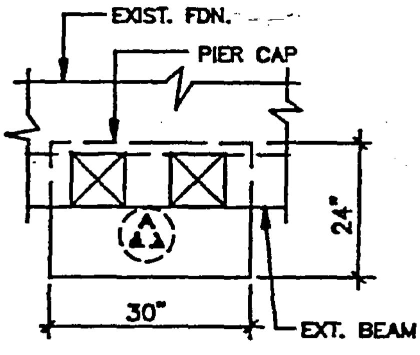



**FIGURE 1. ULTIMATE RESISTING FORCES ACTING ON A DRILLED PIER WITH AN ENLARGED BASE IN COHESIVE SOIL.**




SECTION

ELEVATION




PARTIAL PLAN VIEW

N.T.S.

| PIER SCHEDULE | |
|---|---|
| PIER DIA. | BELL DIA. |
| 12" | 24" |

# LEVELING PIER

N.T.S.

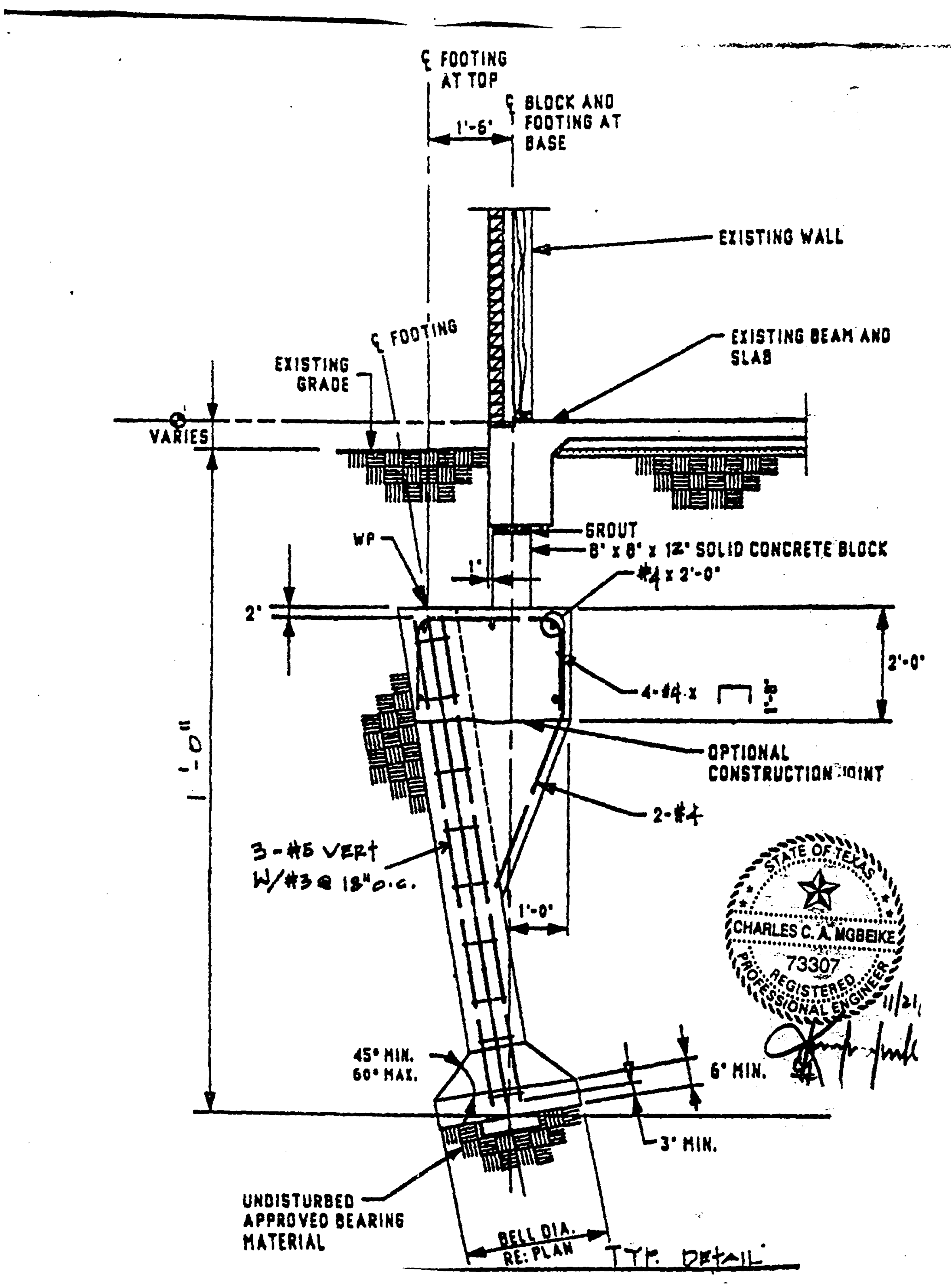
GENERAL NOTES:

1. CONCRETE: 3000 PSI AT 28 DAYS AND IN ACCORDANCE WITH ACI.
2. REINFORCEMENT: A–615 GRADE 40 UNLESS OTHERWISE NOTED AND INSTALLED IN ACCORDANCE WITH ACI 315.
3. GROUT: HIGH STRENGTH, EPOXY GROUT WITH MINIMUM 5000 PSI STRENGTH.
4. SEE PIER LAYOUT PLAN FOR LOCATION OF PIERS.
5. REMOVE ANY STANDING WATER AND DEBRIS FROM THE FOUNDATION PIER EXCAVATION PRIOR TO PLACEMENT OF THE CONCRETE.
6. THE PLACEMENT OF CONCRETE TO BE IN ONE CONTINUOUS POUR TO AVOID POSSIBILITIES OF COLD JOINTS AND HONEYCOMBS.
7. PIER HOLES SHOULD BE FILLED WITH CONCRETE IMMEDIATELY UPON COMPLETION OF DRILLING AND SHALL NEVER BE LEFT OPEN OVER NIGHT.

EXHIBIT A


MAVERICK ENGINEERING
9730 TOWN PARK, SUITE 111
HOUSTON, TEXAS 77036
(713) 271-1941
CORPUS CHRISTI – HOUSTON


| | |
|---|---|
| DRAWN BY: RS | SHEET 1 OF 1 |
| DATE: | JOB NO. |

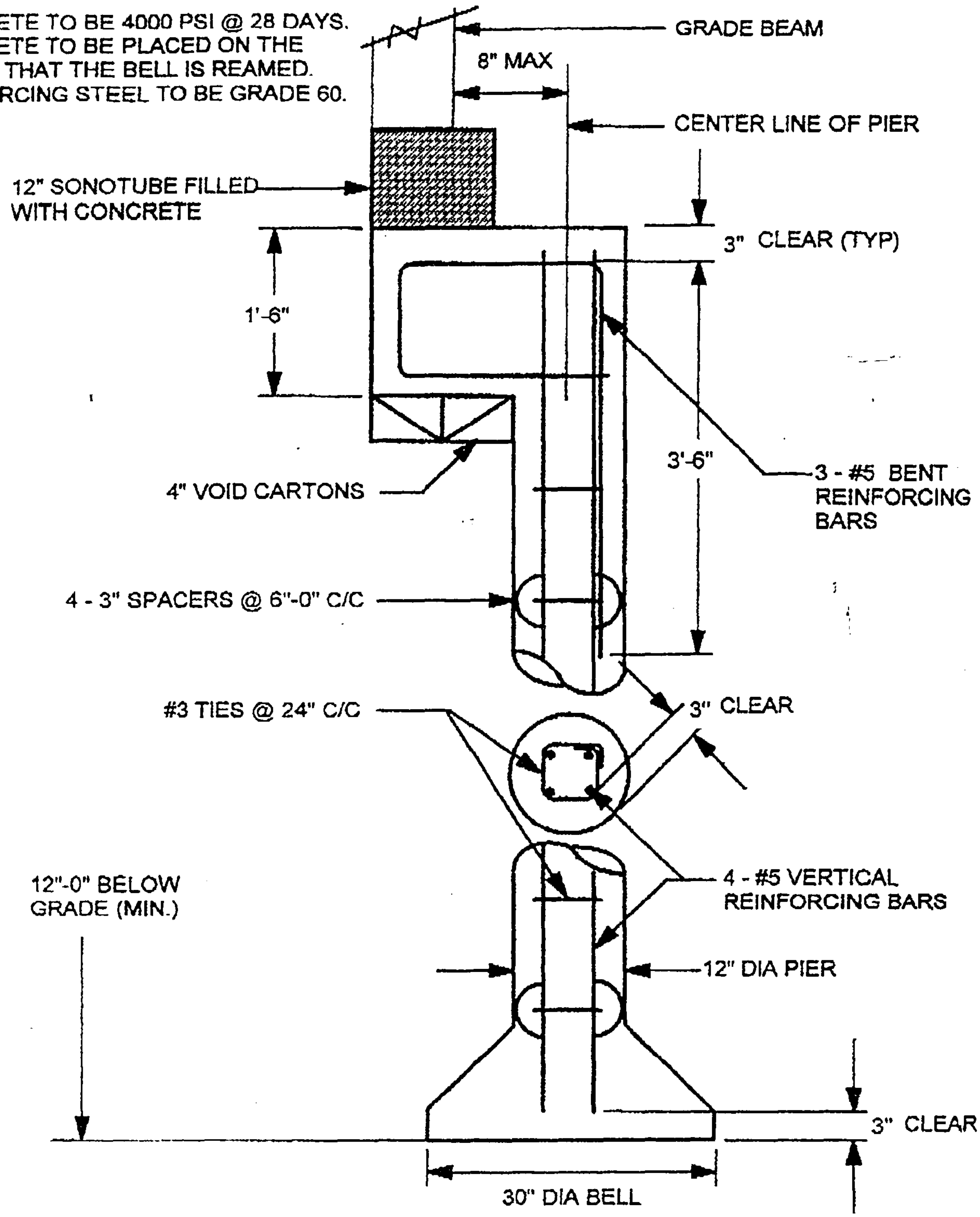


℄ FOOTING AT TOP
℄ BLOCK AND FOOTING AT BASE
1'-6"
EXISTING WALL
℄ FOOTING
EXISTING GRADE
EXISTING BEAM AND SLAB
VARIES
GROUT
8" x 8" x 12" SOLID CONCRETE BLOCK
WP
1"
#4 x 2'-0"
2"
2'-0"
4-#4 x
OPTIONAL CONSTRUCTION JOINT
2-#4
1'-0"
3-#5 VERT
W/#3 @ 18" o.c.
1'-0"
45° MIN.
60° MAX.
6" MIN.
3" MIN.
UNDISTURBED APPROVED BEARING MATERIAL
BELL DIA.
RE: PLAN
STATE OF TEXAS
CHARLES C. A. MGBEIKE
73307
REGISTERED PROFESSIONAL ENGINEER


TYP. DETAIL

# DJS ENGINEERING

NOTES:
1. CONCRETE TO BE 4000 PSI @ 28 DAYS.
2. CONCRETE TO BE PLACED ON THE SAME DAY THAT THE BELL IS REAMED.
3. REINFORCING STEEL TO BE GRADE 60.




| | | |
|---|---|---|
| PIER DETAIL | 3/4" = 1'-0 | |
| | | |
| | DJS | |

To meet these objective, soil borings are recommended. The borings should be done by an agency governed by a Licensed Engineer with expertise in geotechnical engineering. A sufficient number of borings should be taken to assure that the worse case condition on the site have been identified. An independent inspection is recommended to measure the diameter of each bell to assure it meets requirements to limit settlement after the repair project has been completed.

## PRESSED SEGMENTED PILINGS

In the late 1970s time period, Mr. Gene Wilcox, PE, having had similar experiences with drilled piers in the Beaumont area, invented a system for driving concrete cylinders into the soil using hydraulic rams.[Ref. 19] In the early 1980s, Mr. Wilcox moved his business into the Houston, Texas area. The early experience with the segmented piles was something less than exemplary because the refusal depth of the piles tended to be sufficiently shallow to still be in the active zone. Mr. Elmer Dutton, who was an associate of Wilcox, then devised a system for lubricating the sides of the piers with water which enabled them to be driven well beyond the depth of the active zone. Subsequently, Wilcox formed a corporation titled Perma Pile, Inc. with which there were somewhere between 10 and 15 franchisee firms.[Ref. 20] Some other firms then converted their modus-operandi from drilled piers to pilings and several enhancements were made in the employment of the pile systems, one of which was to cast the pile segments with a hole in the center through which cables and/or reinforcing steel bars could be placed. As a result, the success of the pressed segmented pile repair procedures has been exemplary. In fact, the firm of Peverley Engineering, Inc. has inspected somewhere between 15,000 and 16,000 residential buildings without having one documented failure where a foundation, or a part of the foundation, supported on pressed pilings, has failed.[Ref 21]

The following is a discussion of the generic method for the installation of segmented piles. This discussion will be limited to what has been termed the "simple" pressed piling system; i.e., pile systems which are not interconnected. After a foundation has been inspected and a decision made for its underpinning, the locations for the piles are chosen. In the greater Houston area, the distance between piles is often limited to 6 to 7 feet, not because of the capability of the piles (or piers) to be able to lift and support the weight of that portion of the building, but instead because of inherent limitations that may exist in the construction of the concrete grade beams in those older buildings in the greater Houston area which are often in the most need of repair. The installation begins by digging a pit whose dimensions are approximately 3' x 3' x 3' directly under the grade beam. A hydraulic ram is then placed between the bottom of the grade beam and the top of the cylinder, and the cylinder is then pressed into the soil. Care must be taken to start the first pile in as much of a vertical plane as can be achieved. The cylinders are then forced into the soil, one after another, until the desired depth has been determined. The cylinders are driven one at a time so that the weight of a large segment of the foundation and the building it supports is available to offset the force supplied by the hydraulic cylinder. The depth of the pile system should be somewhere between 5 and 10 feet below the active zone in clay soils. In some parts of the greater Houston area, particularly in the Brazos River Basin, it is sometimes necessary to drive the cylinders through sand layers to depths as much as 35 to 40 feet.[Ref. 22] Thus, each pile is, in reality, an ASTM pile load test to failure. Calculations have shown that using a 6 foot spacing, a safety factor greater than 6 can be achieved, not including the benefit of thixotropy. A further discussion on this condition will be provided below.

When pressed piling systems first came into use, there were many detractors who published documents pointing out the drawbacks of this system(23, for example). Some of their complaints included such things as that piles could be deflected by roots, rocks or calcarious nodules; there was no factor of safety (piles were at incipient failure); there was no allowance for live load capacity, etc. Experience has shown, however, that none of these complaints were valid. For example, the concerns that the pile penetrations could be deflected and the piles stem would not be perpendicular did not happen in the greater Houston area. One of the approaches used in these piling systems was to have holes longitudinally through the center of each pile segment through which reinforcing steel bars could be placed after the piles stem had been driven to its desired depth. Problems did not occur with the rebar insertions showing that the pile alignment integrity was maintained.[Ref 24] Safety margins are provided by driving only one pile at a time. Thus the weight of a large section of the building is used to drive a pile which, in service, supports a very limited weight of the building. During one or more excavations beside existing pile installations, it was found that the pile segments tended to be encased in an extremely firm layer of soil somewhere between 2 and 3 inches thick. This condition appears to be the result of thixotropy. Mitchell[Ref. 25] has described thixotropy as an isothermal, reversible, time-dependent process occurring under conditions of constant composition and volume whereby material stiffens while it rests and softens or liquefies upon remolding. Terzaghi and Peck[Ref. 26] have further stated that after soil is thoroughly been molded, the portions of the particles with respect to each other can rotate and assume a more stable configuration, at an unaltered volume. The shearing strength then may correspondingly increase. An example of this phenomenon has repeatedly occurred in off-shoring rig installations where the placement of pilings had to be halted for one to two hours for welding operations. When the installing operations continued, thixotropy had begun and extra efforts were required to even begin the installation operations.[Ref . 27] The existence of thixotropy phenomena, therefore, appears to provide additional stability to the pile system after it has been installed. Thus, most of the detraction promulgated regarding the shortcoming of this approach appear to be unfounded.

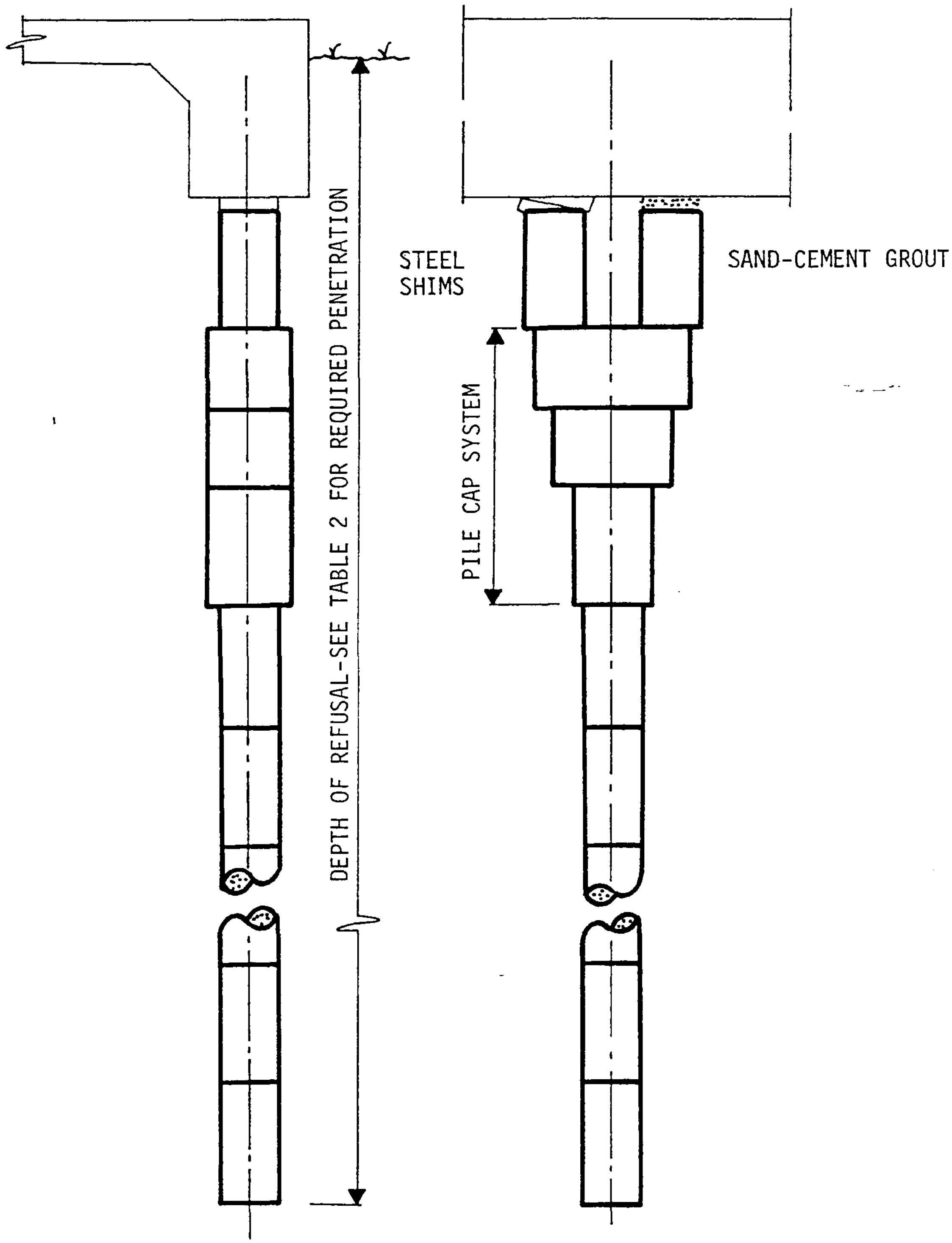
The installation of pressed segmented piles does not require the same degree of quality control as does the installation if drilled piers, some degree of care is still required. The pits must be excavated to a depth to be under the presence of large tree roots. The installation of the initial pile segments must be done with care so as to be in as vertical plane as can be achieved. During the driving process, the operator must use care to not where obstacles are encountered. The bottom of the piles must be at least 6 feet under the bottom of the active zone.

## HELICAL PIERS

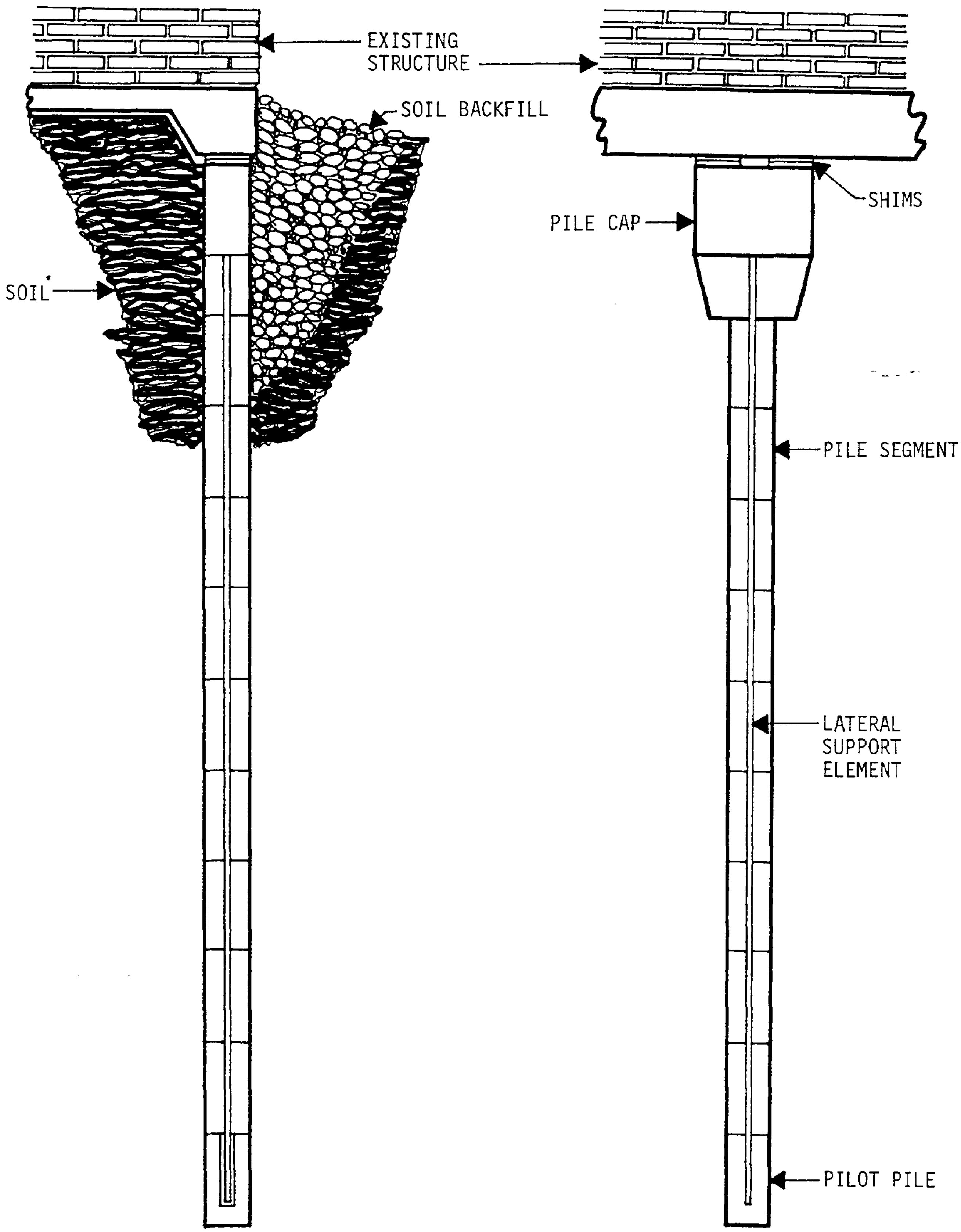
At the present time, the Foundation Repair Subcommittee has not had sufficient time to perform any type of analysis in helical piers. For information purposes, therefore, we are providing, in Attachment B, a technical paper on this subject was presented in 1990.[Ref. 28] This paper is provided for information purposes and is not intended to be the official, or latest, word from the manufacturer.

## CONCLUSIONS

The foregoing information is, as previously stated, very preliminary in nature. It is submitted herein as the initial draft from which comments from anyone will be accepted and considered. Such comments should be submitted to Ms. Ann Nelson.




COMPLETED PERMA-PILE INSTALLATION




GENERIC DRIVEN PILE SYSTEM

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21. Ibid, Ref. 5.
22. Ibid, Ref 18.
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## ATTACHMENT A

## ORIGINAL FHA DRILLED PIER SPECIFICATION

# INSTRUCTIONS TO BIDDERS

1. Access to the property may be arranged through FHA Property Management Section.

2. The bidder should acquaint himself with the condition of the building and of the property upon which the work is to be performed.

3. During the operating of the contract, the builder will be responsible for his tools and equipment and for protecting all work performed or to be performed by or for him. He shall protect against damage and vandalism. Any damage to the property caused directly or indirectly by him, his employees, subcontractors or material suppliers shall be repaired or replaced to its original condition. This applies equally to curbs, gutters, walks, approach, and to grass area which is offsite and adjoining this property.

4. The specifications and drawings have been prepared by the Houston Office of the Federal Housing Administration and supervision will be under the direction of an FHA representative.

5. No work will be commenced until a properly signed and dated contract has been received from FHA by the builder.

6. The successful bidder will be required to carry builders' risk and compensation insurance to the extent required by law of the State and Federal Government. He shall secure all permits, licenses, and pay all fees which are required in connection with the execution of this contract.

7. The FHA may reject any or all bids.

8. Payment for work performed will be issued from Washington after approval by the local FHA office.

9. The occupied property shall have the entrances kept clear for ingress and egress at all times. If and when access is necessary into the house, it shall be by prearranged permission of the occupant.

10. Toilet facilities of the house will not be available to the workmen. Builder shall make his own arrangements for the use of any utilities which he may require.

11. Builder shall provide covers of the necessary size and strength and shall cover all holes which have been created in the operation of this contract, as a protection against injury or damage. The covers shall be in place except when work is being performed. Covers may be of rough lumber or equivalent materials.

10/16/67

(INSTRUCTIONS TO BIDDERS, CONT'D)

Subcontracts:

12. The successful bidder shall be designated as the Contractor.

13. The contract for the execution of this work may not be transferred in whole or in part to a third party nor sublet, without the written approval of FHA.

AFTER THE CONTRACT HAS BEEN SIGNED, THE FOLLOWING PROCEDURE SHALL BE FOLLOWED:

1. Notify owners when work will be commenced.
2. Notify FHA when work will be commenced.
3. Secure any required permits.
4. Remove shrubs which would obstruct operations. Box other shrubs in the manner described in these specifications.
5. Provide covers for holes which are to be dug in execution of the contract.
6. Furnish soil samples to FHA where herein called for.
7. Advise FHA the day before starting work on:
   (a) Start excavation
   (b) Start concrete footing
   (c) Start jacking beam & mud jacking
8. Set permanent bench marks if called for. Continue balance of work as herein specified.

## SCOPE OF WORK

**SHRUBS:**

1. Shrubs around the building which would impede progress of the work shall be balled and heeled in at rear of lot or some other designated area. They shall be properly cared for and returned to their original position at completion of this contract. Shrubs requiring special attention shall be moved under the direction of experienced nurserymen. Properly protect shrubs which are not moved.

**GRASS & TREES:**

2. No heavy equipment is to be permitted on lawns. Trees shall be fully protected. Grass shall be protected to the extent possible.

**INTERIOR FLOORS, WALLS, AND CEILINGS:**

3. Unless specifically called for, no work will be done in the nature of refinishing to the interior surfaces of the house, except that builder will be responsible for correcting any damage due to negligence.

**FOUNDATION WORK:**

4. The foundation grade beam is to be brought to a predetermined level by means of jacks. After the beam has been levelled and stabilized, the house slab shall be brought to a compensating level by means of "mud jacking" or "mud pumping."

5. There will be sections of the beam which will not require jacking up. Footings under these areas will be for the purpose of stabilizing the beam. Except for jacking, these piers will be of the same design and depth as other piers. See the floor plan for possible variations of depth of stabilizing piers.

6. Plumbing lines, water and sewer, shall be protected and repaired if damaged.

7. After the above work has been completed, this contractor shall clean away all debris, remove excess dirt, or fill in, if need be, any voids or low places caused by this operation, with top soil. If any mud or trash has been brought into the building, it shall be completely cleaned out.

   Level the ground around the house by means of rake or hoe to a finished grade, sloping gradually away from the house.

10/16/67

## MATERIALS AND EQUIPMENT

1. Cement: Portland cement - ASTM C 150-64

2. Admixture: Water, reducing and accelerating admixture - ASTM C 494, Type (E)

3. Concrete: 2500 psi in 28 day test, using not less than $4\frac{1}{2}$ sacks of cement per cu. yd. Aggregate shall be clean, washed and properly graded sand and gravel or acceptable crushed rock. Concrete shall be transit-mixed.

4. Pumping Mud shall be a mixture of top soil, free of partially decayed matter and roots, to which is added $2\frac{1}{2}$ sacks of portland cement per cu. yd. of soil, mixed with water, in a motor driven mortar mixer to a highly plastic consistency.

5. Concrete Blocks: Solid 8" x 8" x 16" precast blocks, 3000 psi strength.

6. Equipment: All equipment shall be in proper condition, working order, and capacity for the work involved. Jacks may be hydraulic or ratchet type.

## FLOOR PLANS AND DETAILS

7. Floor plans and a drawing showing sectional details is attached as a part of this specification. The floor plan is only detailed to the extent necessary to show the required work. The sectional drawings show the size, depth, design and reinforcing of the piers and are to be followed exactly. Any variation will be subject to rejection by FHA.

8. Upon completion of all work included within the contract, this Contractor shall, on one of the two attached floor plans, show the location of each pier (to the nearest six inches) by actual job measurements.

9. If any changes have been made from the drawings, a description of the change shall accompany the floor plan.

10. This plan shall be brought or mailed to F.H.A. Attention: W. A. McElroy

## DESCRIPTION OF THE WORK

1. The purpose of this contract is to properly and securely underpin and raise, where necessary, the foundation beam and to level the floor slab. The key to leveling is the floor. The contractor shall determine by means of instruments the condition of the slab and the amount and location of leveling requirements. The results of this investigation will dictate the degree of beam jacking, and later, of slab "mud pumping."

2. At the location shown on the floor plan, pier holes shall be dug and/or drilled 8" or greater in diameter, and to a depth of not less than 12 feet. Hole bottoms shall be splayed to a 20" minimum diameter footing. The direction and positioning are shown on the sectional drawings. As soon as each group of pier holes have been dug, cleaned, and approved by FHA they shall be filled with 2500 p.s.i. concrete to within jack height of bottom of foundation beam. Install three 11-foot #4 reinforcing rods in piers as poured. These rods shall be accurately bent and secured in place. Piers shall be poured in such a way that no more grout than necessary will be used to grout the concrete blocks into their permanent supporting position.

3. It is an essential part of this contract, demonstrated by previous construction, that piers which are started shall be completed and poured on the same day on which digging is commenced, in order to preclude the possibility of water filling and cave-ins due to rain or high water table.

4. The caps of piers are splayed to a minimum of 24" in width and to a sectional width of 20" to 24" at top. The purpose of the 24" width is threefold: first, to provide space for installation of jacks and the 8" x 8" x 16" permanent piers, secondly, to reduce the span of the foundation beam; and thirdly, to permit reinstallation of jacks at some future date if need be, without damage to this installation.

5. After seven days of curing has elapsed, jacks shall be set on the pier cap, in the center of the 24" width and leveling operations shall proceed with sufficient jacks, operators, and controls to lift the beam in large sections uniformly and in unison. The same system shall be observed for all piers even though some piers will be only for stabilizing purposes.

6. If the builder so elects, he may use (a) high, early strength cement, or (b) a water reducing and accelerating admixture, proportioned according to the manufacturer's recommendations. If either option (a) or (b) is used, the curing time may be reduced to 3 days. Proof of use must be furnished FHA in writing.

7. Two 8" x 8" x 16" precast piers, constructed of 3000 p.s.i. concrete shall be grouted in place on each side of jack over each pier. After the grout has set, remove jack and fill the voids of pier hole with bank sand, well tamped and leveled. Remove all wood forms.

8. Since it is not feasible to specify each step of operation, the superintendent must be present to direct each phase of construction.

10/16/67

## LEVELING THE SLAB

1. Leveling the Slab: After the exterior beams have been leveled and stabilized, the house slab shall be brought to the proper elevation by the mud pumping process.

2. Set positive gauges and markers prior to pumping in order that the Superintendent and inspector may check the slab movement.

3. The pumping slurry shall be a mixture of top soil and cement, in the ~~preparation~~ proportion of 2½ sacks of cement to one cu. yd. of top soil, mixed mechanically in a mortar mixer to a highly liquid plasticity by addition of water.

4. This material shall be pumped under pressure through 1½" or larger pipes to the required location. The pump shall be capable of up to 150# per sq. in. pressure. The pipe shall be inserted under, not through, the concrete grade beam, and driven to the desired location.

5. The greatest care shall be taken to fill all voids and in raising the low slab without employing pressure which would cause damage. A capable man shall continually check the inside of the building during this operation. Continual inspection of plumbing pipes and drains is essential.

6. The Contractor shall determine the safe amount of pumping required to meet the objective of filling all voids and of leveling the slab.

# GUARANTEE

This Contractor shall furnish the following warranty-guarantee:

"This is to certify that the materials, labor, and fabrication equal or exceed the requirements set forth in these specifications and that any deviation therefrom has been by authority of the FHA and is described below.

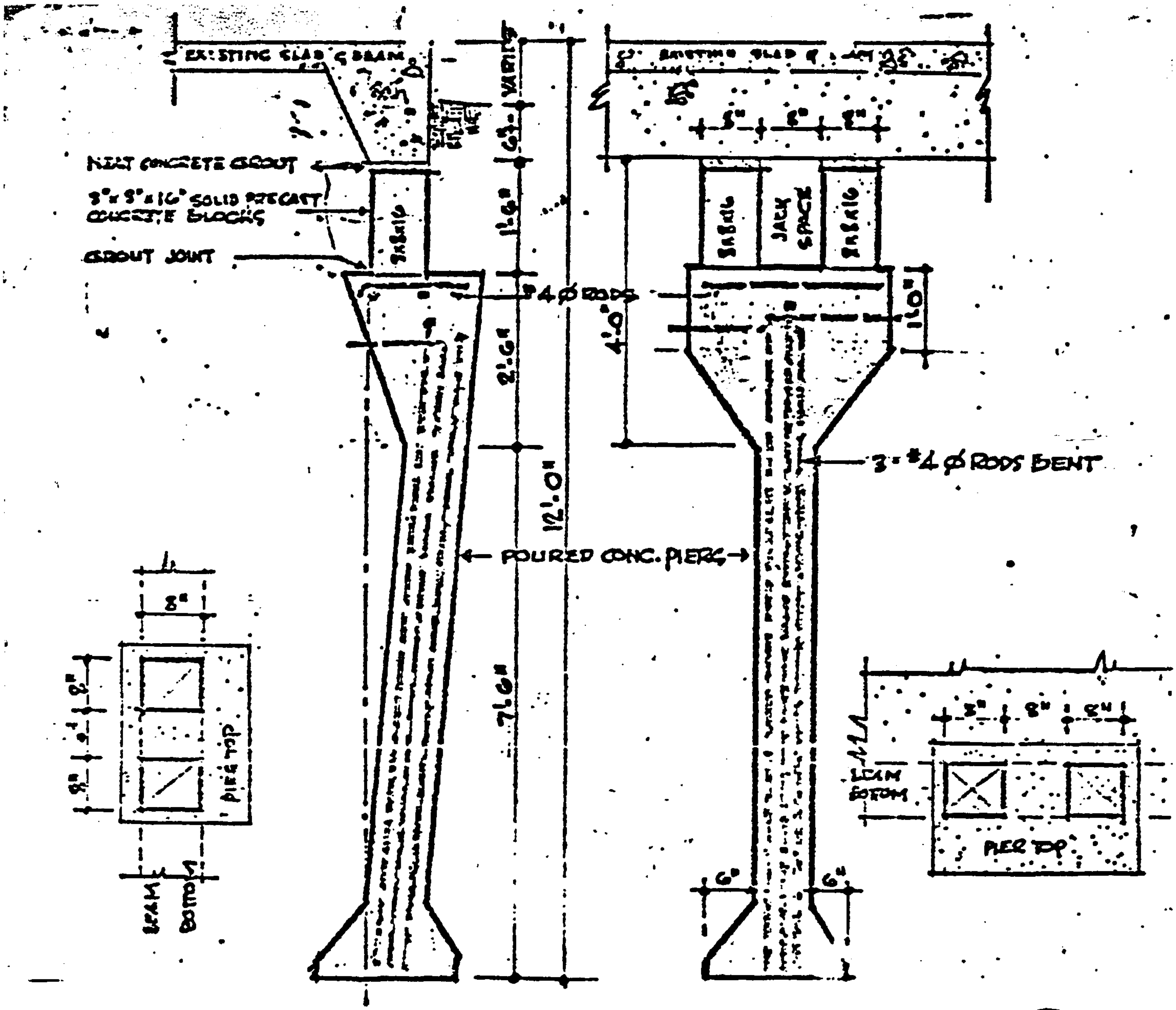
"This work is guaranteed against failure for a period of one year from the date of completion and acceptance by FHA. If substantial movement or change occurs, we will readjust the foundation as required without additional cost to owners or FHA.

"This guarantee applies only to the foundation of this building, or to the part of the foundation which was repaired under this contract, including the slab, and not to any resultant damage which might appear in the superstructure."

Signed: ____________________

By: ____________________

Date: ____________________




PLAN & SIDE VIEW

FRONT VIEW & PLAN

# PIER FC NDATION DETAILS

SCALE 1/2"=1'-0

FOR REPA... Y TO DAMAGED FOUNDATIONS WHERE SPECIFICAL DESIGNAT...

REPAIR TYPE - I.

# ATTACHMENT B

# HELICAL PIER PAPER

# Helical Bearing Plate Foundations for Underpinning

by
Stan Rupiper and William G. Edwards

presented at
Foundation Engineering Proceedings
Congress/SCE/CO Div.
Evanston, IL
June 25-29, 1989



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*Presented at Foundation Engineering Proceedings*
*Congress / ASCE / CO Div. / Evanston, Ill.*
*June 25-29, 1989*

*(Converted to imperial quantities)*

# HELICAL BEARING PLATE FOUNDATIONS FOR UNDERPINNING

Stan Rupiper[1], M. ASCE and William G. Edwards [2], Aff. ASCE

ABSTRACT: Helical anchor foundations have been used in construction to provide resistance against uplift and compressive loading for structures such as transmission towers, tied walls, pipelines, and offshore structures. Requirements for underpinning for the rehabilitation of building and structures are for sectional piles and low vibration installation equipment. The helical foundation meets these requirements using helical bearing plates and bolted coupling method for the sectional piles. Installation by rotary drive tools minimizes vibration during installation.

The paper covers the methods used to determine the load requirements, the theoretical design of the helical foundations for the specific soils and the techniques used to install the helical foundations in confined areas. Test methods and test data from an actual test of a helical foundation is included in this paper.

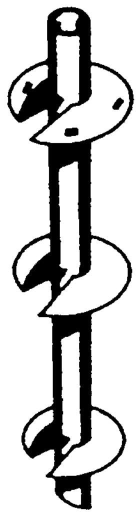
## INTRODUCTION

This paper defines the structural aspects of the helical foundation and the methods used in determining the helix sizes for an application in a clay soil. A soil boring was taken and the foundation was sized for a typical structure. The selected foundation was installed, measuring the torque during installation and then the foundation was tested applying the load incrementally.

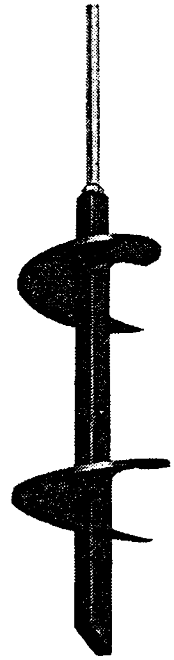
The calculated load and the tested load reflect favorably as does the ratio of torque to capacity but all are subject to defining what is ultimate. The writers selected to go to the calculated limit so a "failure" present was not determined.

Following this foundation selection process, field installing techniques are described with an emphasis on expansive soils.

1 - Consulting Engineer, 1033 Villa Maria Ct., San Jose, CA 95125
2 - Manager, A.B. Chance Co., 210 N. Allen, Centralia, MO. 65240



Pipe Type Shaft



Square Shaft

Figure 1. Illustration of Foundation Anchor

Description of Helical bearing Plate Foundation (Foundation Anchor). The helical foundation anchor can be simply described as single flights of screw helix spaced three diameters apart along a shaft designed to withstand the compressive and tensile loads as well as the installing torque.

These shafts commonly are made from round cornered square (rcs) bar in 1½", 1¾", 2" and 2¼" steel (standard and high strength) as well as from pipe 2", 3", 4", 6", 8", 10", (standard and heavy wall) and rod in ¾", 1" and 1¼" diameter. The helices normally have a 3" pitch manufactured from the proper strength and thickness steel to match the load requirements. The finished assembly is galvanized to ASTM A153 specifications. Typical foundation anchors as shown in Figure 1. Should the soil be excessively corrosive special coatings can be applied at the time of installation or cathodic protection can be used.

## GENERAL DESIGN CRITERIA

The selection of a helical bearing plate foundation (foundation anchor) type and size is a function of the loads and soil for the application, and methods of capacity prediction are well documented in studies by Kulhawy (1985), Clemence, etal.(1985) and Bobbitt and Clemence (1987). Extensive studies and tests have been done on helical bearing plates and it has been shown that this information can be directly applied to anchors used for foundations in either tension or compression. The shear and multiple tip bearing components that are used for calculating helical bearing plate capacity are in Figures 2 and 3, respectively.

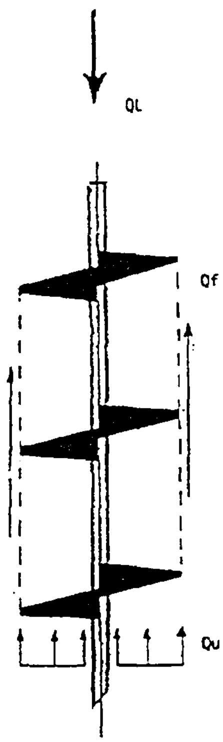



**Illustration of**
**Bearing & Cylindrical**
**Shear Soil Reaction**

**Figure 2**

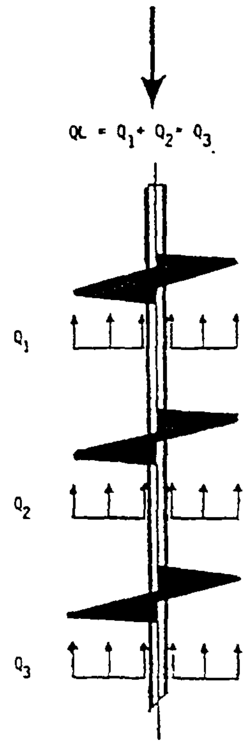



**Illustration of**
**Bearing Soil**
**Reaction**

**Figure 3**

The bearing plate method uses the general bearing plate formula of:

$$q_u = cN_c + qN_q \qquad (1)$$

where: $q_u$ = ultimate soil bearing factor, c = cohesion of soil, q = overburden pressure, $N_c$ and $N_q$ = Bearing capacity factor for local shear conditions. The helix selected for the foundation anchor is selected based on the surface area required to develop the capacity to withstand the load including the safety factor. The lateral load is considered and if determined to be a factor, tension anchors are used to offset the lateral loading.

DESIGN EXAMPLE

An example of the selection of one of these foundation anchors is as follows. The soil test report shown in Table 1 is representative of the soil's condition at a typical site with expansive soil.

## Table 1. Soil Information/Installing Torque

| Depth from grade (feet) | Soil Description | ASTM D-1586 blow count (n-values) blows/at feet | Torque re-corded ft-lbs |
|---|---|---|---|
| 1 | Firm wet brown clay | | |
| 3 | | 7 | 480 |
| 4 | Firm wet grey clay | | 1965 |
| 5 | | 5 | 1570 |
| 7 | Firm moist grey silty clay | | 1770 |
| 8 | | 7 | 1870 |
| 11 | c=1900 PSF | | 1770 |
| 12 | | | 2160 |
| 13 | Stiff-Very Stiff Brown | 18 | |
| 14 | Grey Silty Clay,Till | | |
| | c=2300 PSF | | |

Soil Boring

### Table 1

The structure to be supported had a vertical loading of 500-700 lbs. per lineal foot on the existing strip foundation of 12 inches wide. A spacing of 10 feet between foundation anchors was selected to support the load on the foundation. This required the anchor to support a 5000-6000 lb. compression load; a minimum safety factor of 3 results in a 20,000 lbs. vertical anchor design load.

The 20 kip design load plus the soils data given when used with the bearing equation produced a 14" helical bearing plate needed to meet the load. The same result is obtained using the bearing and cylindrical shear method given by Bobbitt and Clemence (1987).
This equation for cohesive soils is:

$$Q_u=A_1cN_{cu}+pD_ac(H_n-H_1)+P_sH_1c_a \quad (2)$$

In which $Q_u$=ultimate anchor capacity, $A_1$= area of top helix, c=cohesion at helix plate, $N_{cu}$=bearing capacity factor for cohesive soils, $D_a$ =average helix diameter, $H_n$=depth to bottom helix, $H_1$=depth to top of helix, $P_s$=perimeter of anchor shaft, $c_a$=adhesion to anchor shaft. This example is for a single helix anchor and the results are identical for both equations. Multihelix anchors produce similar results between the bearing plate equation and the cylindrical shear plus bearing plate equation when the helix spacing is three diameters.

## INSTALLING TORQUE CONSIDERATIONS

With the bearing plate now selected, a shaft of 1½" rcs steel was selected to transmit the load from the foundation to the bearing plate, and since the foundation is surrounded by soil, the slenderness ratio is not considered a factor for these reasonable heavy shafts and small loads.

Field experience has also shown that installation in this type of soil does not develop significant installation resistance so a shaft of 1½" rcs steel could easily withstand the torque loads of installation. The torque capability of this shaft has a minimum ultimate of 5500 ft-lbs. and will be within the limits to drive the 14" diameter helix to the 10' depth.

Experience has also shown that a correlation of installing torque to capacity, although not addressed theoretically, can act as a quality assurance method at time of installation. This torque value, when monitored during the installation, would result in an estimate of the soil profile as the helix penetrates through the different strata. Correlation of the installing torque to the anchor capacity may be used as a site specific production control method of foundatin anchor installation. Torque indicators are commercially available. Two examples are shown in Figures 6 and 7.

### TORQUE INDICATORS

ELECTRONIC
TORQUE INDICATOR

**Figure 4**

SHEAR PIN
TORQUE INDICATOR

**Figure 5**

When confirmed with actual on-site tests, the installing torque becomes the quality assurance record of each anchor foundation. Mitsch and Clemence (1985) recorded torque and capacity in their field test program and obtained ratios of capacity to installing torque as high as 26 and as low as 12 in their test in sand. A ratio of 10 to 1 is commonly used by practitioners today.

Actual testing of selected anchors is recommended to assure that the design capacity of 20 kips is met. A suggested method of testing is to use two helical reaction anchors to secure the reaction beam as load is applied on the anchor. A dial indicator should be used to measure movement of the anchor. The load should be applied in at least four increments and movement recorded as required by the engineer. Long term tests (24 hours) should be done in similar soil in the locality and these creep characteristics compared to the initial reaction (movement) of the foundation anchor in the short term loading.

Field Tests: Field installations are tested to the design loads and not to the ultimate capacity, informing the engineers only that the design is adequate but not if it is over designed. Initial testing in controlled situations reflect good correlation in clays. An example follows:

This clay site is located adjacent to Missouri Highway 22 in Centralia, MO. The soil profile can be described as consisting of 10 to 20 feet of normally consolidated firm to stiff silty clay (highly plastic) underlain by an over consolidated very stiff to hard silty clay (till).

The following load criteria was set forth to determine an anchor design and a single helix design was determined to be desirable. Therefore, equation 2 becomes:

$$Q_u = AcN_{cu} \qquad (3)$$

Substituting the values and solving for the required helix area, a 14" diameter helix was selected to support an ultimate load of 20 kips.

A helical foundation was selected with a 14" diameter helix, a 1½" rcs steel shaft 10' in length to reach the soil indicated on the boring having more than an undrained shear strength of 2300 psf. The foundation was installed using a hydraulic drive head to a 10' depth with continually monitoring of torque.

Two reaction anchors were installed 5' on either side of the foundation and a compression jack was mounted on the foundation with a test beam mounted above the jack. The test beam was restrained by the reaction anchors. Movement was measured by means of a dial indicator attached to the anchor shaft. Load was applied incrementially in approximately 2,500 lb. steps. Table 2 gives the test results for the above test.

**Table 2.**
**Test Results for 14" helix anchor embedded at a depth of 10'**

| Applied load (lbs.) | Movement (inches) |
|---|---|
| 2800 | 0 |
| 4950 | .012 |
| 6000 | .023 |
| 8000 | .063 |
| 10,000 | .089 |
| 12,500 | .119 |
| 14,700 | .141 |
| 16,150 | .144 |
| 17.500 | .145 |
| 20,000 | .146 |
| 10,000 | .145 |
| 200 | .089 |

## USE IN EXPANSIVE SOILS

Recent applications have shown that the helical foundation anchors have an excellent application for underpinning in expansive soils. The helical plates are installed to a depth where the constant water content of soils is presumed to have a negligible effect on the change of the swelling or shrinking characteristics of the soil. These plates have the capability of resisting both downward and uplift loads and they can be designed to remain as stable as the soils in which they are imbedded. The shaft transmitting the load to the bearing plate through the active zones is subject to the expansion/ contraction of the soil. The soils when expanding will apply forces to the shaft laterally and upwardly which can move the foundation upward unless properly designed. The designing engineer should consider the uplift load due to the expansive soils on the shaft and footing, and dynamic loads such as the wind and seismic loads on the structure.

The uplift due to the skin friction on the smaller shaft of the helical foundation-anchor is considerably less than that experienced on a larger diameter concrete pier. The designer will find this uplift contribution small compared to other loads he must consider. On new construction the structure uplift due to expansion can be reduced by designing a void under the structure's bearing areas, if this is not acceptable, then the design should allow the soil to move around the foundation area thereby reducing the upward forces on the foundation.

## UNDERPINNING EXISTING STRUCTURE

A foundation anchor system for underpinning an existing structure with sufficient compression and/or tension capacity can be attached directly to the existing foundation A void should be constructed under the existing foundations to eliminate the upward swell pressure. The void should be constructed in a manner that will remain open to allow the soils to expand into the void. A redwood board or sheet plastic along the foundation to serve

## SUMMARY

The helical bearing plate foundations have been utilized in several residential and commercial buildings with successful results. The predictability of the capacity based on work done for tension anchors proved invaluable for this application, but it is evident that additional work is required, especially in the relationships of installing effort (torque) to load capacity. The capacity ratio of 10 to 1 is a conservative number often used by practitioners to act as a practical field measurement to guide the installer.

Many additional structures are presently scheduled to use helical bearing plate foundations to mitigate their distress. New installation equipment, field monitoring equipment, and techniques will assist in gaining additional information concerning these foundations.

## ACKNOWLEDGMENTS

The installation work preceding this paper done by Rick Fuller of Sunstone Construction was invaluable in developing the techniques described in this paper. The writers would also like to thank Dr. Sam Clemence and Dr. Fred Kulhawy for their assistance and work on the theory predicting the performance of these helical bearing plate foundations.

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