SOIL-STRUCTURE INTERACTION SEMINAR 1994

Sponsored by

Foundation Performance Committee

Lovell's Slides

Ramada Hotel NW
12801 NW Freeway
Houston, Texas
July 28, 1994

1994 SOILS-STRUCTURE INTERACTION SEMINAR

Introduction

FOUNDATION PERFORMANCE COMMITTEE

The Foundation Performance Committee was formed by a group of Houston area engineers of various disciplines. The original committee members were limited to engineers who were speakers at the first Soil-Structure Interaction seminar three years ago. All of the members are involved in some way with the design, construction, diagnosis of failure, repair and/or litigation related to foundations in the Houston area. In the past year, our committee has grown in numbers and includes representatives (engineers and non-engineers) from various related industries, such as: foundation repair contractors, attorneys, builders, geo-technical engineers, structural engineers, geologists and others.

The Committee Chairman has conducted surveys in the name of the Committee as relates to foundation deflections measured just prior to foundation repair and to real estate inspectors methods and procedures in performing the foundation portion of their pre- purchase real estate inspections.

The Foundation Performance Committee is a non-profit group dedicated to the promulgation of uniform guidelines for predicting, measuring and evaluating foundation performance.

The Committee members who are speakers today are, without exception, recognized leaders in their respective industries. They are all highly qualified with extensive experience.

Lowell PT SLABS
— WATER IN BEAMS

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SOIL-STRUCTURE INTERACTION SEMINAR
SPONSORED BY THE FOUNDATION PERFORMANCE COMMITTEE

The purpose of the seminar is to develop a more consistent foundation design and construction procedures for residential and light commercial projects in the Gulf-Coast area. In addition, our experts will define what constitutes foundation failure and will develop a failure criteria. In this seminar, we get all of the design and construction team members together to develop a more uniform design, construction and quality control procedure. This is becoming more and more important in light of the significant number of foundation failures and potential litigation and exposure of the design and construction professionals.

PROGRAM AGENDA

o	Introduction David Eastwood, P.E.	(5 minutes)
o	Foundation Failures Committee Report Jack Deal, P.E. - Jack Deal Consultants, Inc.	(25 minutes)
o	Foundation Failures In The Houston Area Richard Peverley, P.E. - Peverley Engineering, Inc.	(30 minutes)
o	State-of-Practice of Foundation Design Tract Homes - Lowell Brumley, P.E., - TSG Consultants, Inc. Custom Homes - Brad Crane, P.E. - Shepard Crane & Associates, Inc.	 (20 minutes) (20 minutes)
o	Future of Post-Tensioned Slabs on Ground Don Illingworth, P.E. - Don Illingworth & Associates	(15 minutes)
o	Break	(10 minutes)
o	Guidelines For Geotechnical design, Construction, Quality Control, and Failure Evaluation For Residential Projects In The Houston Area David Eastwood, P.E. - Geotech Engineering and Testing, Inc.	(60 minutes)
o	Recommended Quality Control and Inspection Platt Thompson, P.E. - Thompson Engineering, Inc.	(25 minutes)
o	Break	(10 minutes)
o	Foundation Repair Techniques Don Lenert, P.E. - Don Lenert Engineers, Inc.	(25 minutes)
o	Builders Point Of View To Be Announced	(20 minutes)
o	Government Agency Point Of View Joe Edwards - City of Bellaire Building Official	(20 minutes)
o	Deceptive Trade Practice Act James Moriarty - Attorney-At-Law Daniel Shank - Davis & Shank, P.C.	 (20 minutes) (20 minutes)
o	Break	(10 minutes)
o	Panel Discussion Special Panel Guest - Brent Barron, President Foundation Repair Association of Texas (FRAT)	(70 minutes)

The seminar will be held at the Ramada Hotel N.W., Located at 12801 Northwest Freeway, Houston, TX 77040 (corner of 290 and Pinemont), in the Austin Ballroom from 12:30 to 7:15 p.m. on July 28, 1994. RSVP to Geotech Engineering and Testing, Inc., Ms. Kimberly Robinson (713) 683-0072 before July 22, 1994. The registration fee is $49 if registered before July 22, 1994. If registered after this date, the fee will be $59. This fee includes course notes, beverages, and desserts. This fee is Non-Refundable. Please make your check payable to Geotech Engineering and Testing, Inc., Dept. Seminar, 5889 W. 34Th, Houston, TX 77092. Please feel free to distribute this seminar information to your colleagues and clients.

FOUNDATION FAILURE DEFINED IN TERMS OF PERFORMANCE

by

JACK DEAL, P.E.
JACK DEAL CONSULTANTS, INC.

E.J. (JACK) DEAL, JR., P.E.

GENERAL QUALIFICATIONS

Mr. E.J. (Jack) Deal, Jr. is a Registered Professional Structural Engineer, and President of Jack Deal Consultants, Inc. His primary professional activity is that of inspecting and reporting on structural and mechanical aspects of residential and commercial properties.

Subsequent to receiving his B.S. degree in Architectural Engineering from the University of Texas in 1960, Mr. Deal's work has included: engineering design, general construction supervision, cost estimating and extensive inspection of residential and commercial properties. Over the last 26 years, he has personally performed over 10,000 structural and foundation inspections. His companies have issued over 45,000 inspection reports, the majority of which have included an evaluation of the foundation of the subject structure.

Due to his specialized knowledge and his experience in construction disputes, Mr. Deal has acted as an engineering expert giving testimony in deposition and in court extensively during the last 20 years. In addition, he has acted as conciliator, arbitrator and has on occasion been appointed by the Court as Master in Chancery. His specialty is that of problem identification and evaluation of structures and foundation failure.

PROFESSIONAL AFFILIATIONS

National Academy of Forensic Engineers (NAFE)
* Fellow and Charter Year Member.
* Standards of Practice for property inspectors and guidelines for foundation performance.

National Association of Building Inspectors (NABI)
* Diplomat, Charter Member.
* Standards of Practice for property inspections and due-diligence surveys of commercial properties.

National & Texas Society of Professional Engineers (NSPE, TSPE)
* Member (over 25 years).
* Member of two Chartered Affinity Groups: National Academy of Forensic Engineers (NAFE), and Board certified Diplomat of the Council of Engineering Society Boards (CESB).

National Society of Architectural Engineers (NSAE)
* Founding Member.

American Society of Home Inspectors (ASHI)
* Previously served as Officer; National Director; and Chapter President & Director for the last 10 years.
* 1986 World Conference of Independent Building Inspectors - Committee Chairman and chief architect of this first of its kind event.
* Committee work included: Chairman of Standards Committee (re-drafted proposed changes in Standards and Code of Ethics).

Texas Association of Real Estate Inspectors (TAREI)
* Co-founder and Charter Year President.
* Instrumental in development of Standards, Ethics, Educational Programs and passing by State Legislature of Licensing laws for inspectors. Authored the original draft of Inspection Guidelines and Ethics.

Greater Houston Builders Association (GHBA)
* Associate Member for 20 years
* Twenty years on Consumer Affairs Committee (formerly known as Ethics & Arbitration Committee).
* On sub-committee which drafted construction guidelines and conducted educational seminars.

Foundation Performance Committee (a group of independent engineers)
* Chairman
* Speaker at annual SOG Seminar

FOUNDATION FAILURE DEFINED IN TERMS OF PERFORMANCE

By
Jack Deal, Chairman
Foundation Performance Committee

1) WHAT IS FOUNDATION FAILURE?

To the knowledge of this writer, there is no "widely accepted and purely objective" set of performance standards for determining foundation failure. And, due to the large number of variables which influence such determination, we may never see such idealistic standards published.

Determining foundation failure is for the most part by "subjective opinion" based on the knowledge and experience of the person giving the opinion without the benefit of widely accepted guidelines. Such opinion may be: 1) purely subjective "the foundation has failed because I say it has failed" or 2) based on comparison of quantitative measurement and other observations with normal performance. It is the latter of these two that this paper addresses.

There have been many studies and excellent papers written on foundation performance and/or failure. None known of by the writer has been widely accepted as a formal standard or even as a "general guideline". Due to the spiraling incidence of litigation concerning foundation failure, the need for a widely accepted, published guideline for foundation performance is near critical.

To use an old cliché: Foundation Performance Guidelines "is an idea whose time has come." The need is here, it is everywhere and, it is now.

The cry for help is loud and clear, particularly from those who have been hammered in the court room by frivolous claims based on opinions by experts who proclaim

"IT HAS FAILED BECAUSE I SAY IT HAS FAILED"

The Foundation Performance Committee was formed for the purpose of conducting studies and cooperating with other entities in our industry to define foundation failure expressed in terms of performance and to promulgate guidelines for measuring such performance.

Following is a Glossary of terms and phrases which are a basis for our definition of foundation failure and of our initial effort to outline performance guidelines:

2) GLOSSARY:

Evidences & Consequences of Foundation Movement (E&C): Cracking of rigid materials, (sheetrock, brick, concrete, etc.); separation of joined materials (frieze moldings, rafters to ridge, window to brick, cabinet to wall, etc.); sloping of naturally horizontal surfaces, wracking of structural members, etc.

Fail - (Dictionary definition): 1) to fall short; be deficient, inadequate, or lacking; ..... 6) to stop functioning.

Foundation: That structural component (footing, pile, pier, slab, etc.) of a total structural system which transfers load directly to the load-bearing soil or other earth element.

Foundation - intended function: To provide a stable support for applied loads.

Guidelines: Generally accepted range of criteria for evaluating foundation performance (primarily as relates to movement and consequences of movement).

Movement: **Differential movement** is that movement (up or down) which causes significant stress in the foundation and supported structure; **uniform movement** is that movement or tilting of the foundation and supported structure as a whole which causes little or no stress.

Soil - Structure Interaction: Soil - structure interaction is the combined resultant action of both the soil and the foundation. Foundation performance includes the performance of the soil as well as the performance of the structural aspects of the foundation.

Stable: Having a normal and expected degree and rate of movement. (Movement/time.)

Standard: The word standard refers to a formal or required level of performance or compliance.

So what is foundation failure? If we accept and use the above definitions of terms and phrases, we have the following simple definition of "foundation failure":

3) DEFINITION OF FOUNDATION FAILURE:

Foundation failure has occurred when a foundation no longer performs its intended function of providing a stable support for applied loads (observed performance falls short of normal or expected performance).

The word "**stable**" is the key word in defining foundation failure in terms of performance. To measure stability we must measure movement and compare it with time.

FOUNDATION FAILURE DEFINED IN TERMS OF PERFORMANCE

A FOUNDATION CONDITION IS STABLE (ACCEPTABLE) WHEN THE AMOUNT OF MOVEMENT EXPERIENCED IS WITHIN NORMAL EXPECTATIONS FOR A GIVEN WINDOW OF TIME.

Sound simple? Yes! Is it simple? No, not when you consider that there are an infinite number of combinations of types of movement patterns within specified periods of time and that there are numerous variables which affect such movements.

It gets more complex when we expand our evaluation beyond the simple question of foundation failure (yes or no) to include the consideration of "cause" as well as affect. To do this we must delineate the functions of three key elements: 1) the supporting soil, 2) the foundation mass as a structural element, and 3) the supported structure. Discussion of "cause" is beyond the scope of this paper and will be covered in a future work. For this paper, we will consider that failure due to interaction between these three elements is a "system failure" and that it is synonymous with the term "foundation failure".

Examples of the **numerous variables** mentioned above include but are not limited to: soil properties and conditions of all kinds; erosion; geological faults; drainage; antecedent rainfall; vegetation; loading conditions of the structure; age of the structure; revisions to the structure; type of foundation; add-ons to the foundation; cover-up repairs to cracks, separations, doors, etc.; geometric shape of the foundation; latent defects (such as void boxes channeling water under house); compaction of improper structural fill; homeowner maintenance; and more.

Considering all the above it becomes easy to understand why formal, widely accepted performance standards have not evolved.

IT APPEARS THE BEST WE CAN HOPE FOR AT THIS TIME IS GENERAL PERFORMANCE GUIDELINES BASED ON THE MOST COMMONLY USED AND READILY AVAILABLE MEASUREMENTS OF FOUNDATION MOVEMENT AND CONSEQUENCES OF SUCH MOVEMENT TO DETERMINE STABILITY AND THEREFORE WHETHER OR NOT FAILURE HAS OCCURRED.

4) FOUNDATION PERFORMANCE GUIDELINES

What are the readily available measurements and how much movement is allowed before a foundation is to be labeled "unstable" or "failed".

Foundations with a degree of movement which have not caused problems (frustration or spending of money) can be and probably would be labeled as performing their intended function or in a stable condition.

If on the other hand, a given amount of movement causes undue frustration or major expenditure of money, is it reasonable to label it as not performing its intended function or as unstable? Probably so.

So how do we determine what degree of movement constitutes the norm? Should it be based on the science of engineering using a formula such as L/360 as allowable deflection? Should it be based on the knowledge and experience of the individual giving the opinion? Or should it be based on a broad scale history of actual action taken in cases where the question of foundation failure and/or repair has been resolved.? The answer is probably all of the above with emphasis on the latter.

The above general questions were phrased eloquently in an excellent paper by Author Koenig, titled "Slab On Grade Deflection: How much Is Too Much?, presented at the 1992 7th International Conference on Expansive Soils. Following is a direct quote from the paper:

"SURPRISINGLY , ALTHOUGH ENGINEERS HAVE BEEN LOOKING AT "STRUCTURALLY FAILED" FOUNDATIONS FOR YEARS, THERE HAS NOT BEEN MUCH PRESENTATION OF DATA THAT CAN ANSWER THE QUESTIONS OF "NORMAL EXPECTATIONS" AND "WHAT EVERYONE ELSE IS DOING".

"REMARKABLY, IN ALL THE STUDIES AND REPORTS IT SEEMS THAT THE MAIN QUESTION THAT WOULD ENABLE RATIONAL DISCUSSION OF FOUNDATION FAILURE IS LEFT UNANSWERED. THAT MAIN QUESTION IS "WHEN DO PEOPLE FIX THEIR FOUNDATIONS.", AND BY EXTENSION, "CAN WE, AS ENGINEERS, USE THIS AS A BASIS UPON WHICH TO BUILD A DEFINITION OF FAILURE."

The answer to this basic question appears to be a resounding yes (it is important to point out that this basis for determining foundation failure emphasizes comparison of observed performance with normal or expected performance, but not to the exclusion of applicable engineering principles).

In support of this point of view it is emphasized that it is the homeowner (or prospective homeowner) who first reacts to perceived foundation related problems. It is the homeowner who writes the check for the "fixing". It is also the homeowner who is the most frequent plaintiff in the spiraling litigation mentioned. It seems more than reasonable therefore, to consider the average homeowner expectations of foundation performance as a primary factor in determining foundation failure.

If we average measurable conditions (particularly amount of foundation movement) just before repair we have a reasonable performance guideline for this important aspect of foundation performance.

To this end, Mr. Koenig's paper goes on to present the results of a survey of maximum elevation variations (amount of foundation movement) taken prior to repair of each of 148 foundations in 1989 in San Antonio, Texas. Readings were limited to difference between high and low point prior to repair. The results were: minimum difference - .63 Inch, maximum difference - 7.75 inch The mean was 2..5 inch. (See chart on next page.)

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A similar survey is now being conducted by this writer in Houston. Several local contractors agreed to fill out survey forms. Data requested includes: measurements such as number and size of cracks and separations; floor slopes; elevation readings; and other data. Elevation variations ranged from a minimum of 0 to a maximum of 7.25 inch The mean based on limited current data is 1.75 inch (This is 3/4 inch lower than the 2.5 inch mean arrived at in the San Antonio survey approximately 5 years ago.)

Another survey being conducted at this time by the Slab On Grade committee of the Post Tension Institute (PTI) takes the opposite approach from the two surveys above which attempt to record actual conditions at time of repair. In the PTI survey, an initial set of guidelines were developed and presented to engineers to be compared with their findings at time of recommendation for foundation repair. The bottom line of this type survey is that it compares engineers opinions on when repair is required without taking into consideration whether or not those recommendations actually resulted in repair.

Although information from the PTI survey of how engineers quantify foundation failure is unquestionably valuable the results of surveys based on "what everybody else, as well as engineers, is doing" such as the two surveys mentioned above, may provide a more rational basis for widely accepted performance guidelines for all concerned, including engineers.

The writer also conducted two surveys of Real Estate Inspectors aimed at how they evaluate foundations during a pre-purchase home inspections and how they define failure. The surveys included 300 plus members of the American Society of Home inspectors at their 1994 educational conference in San Diego Calif. and over 100 inspectors at a seminar presented in Feb. 94 by the Texas Association of Real Estate Inspectors in Austin, Tex..

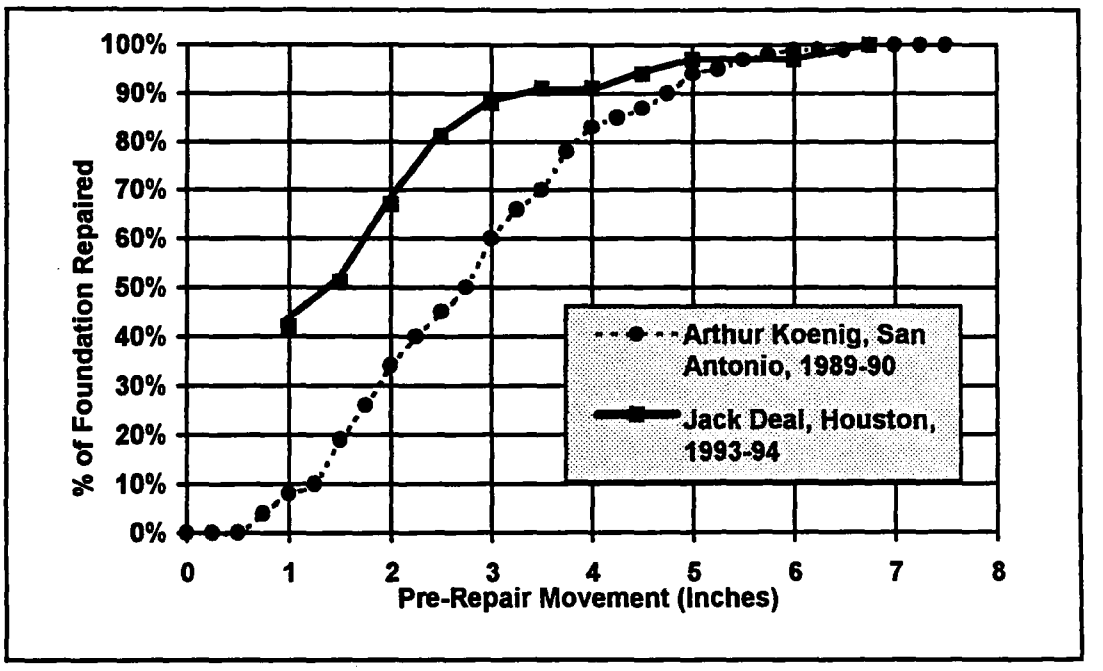
Approximately 20% of the respondents were PEs. Other respondents included: ex-contractors, architects, ex service company owners and others. A high percentage were college graduates. Average experience inspecting foundations was 5 - 10 years. Results of this survey are to be tabulated and presented in another writing. Examples of the results included:

* Less than 50 percent used an instrument (hand held spirit level or water level)

* Wording for foundation ratings included: not performing intended function., unsound, unstable, "Not OK"-or some variation of these wordings.. Very few used the word "Failed".

* Key words from their definitions of "foundation failure" were similar to their words for rating a foundation: unstable, unsound, not performing intended function, inability to support applied loads, etc.

Essentially all of the respondents of both surveys expressed belief that Guidelines for measuring foundation performance was an excellent idea. Most stated that if they observed evidence of foundation failure they would refer their client to further investigation by a registered professional engineer.



Cumulative Distribution Plot of Percentage of Foundations Repaired vs. Pre-Repair Movement.

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5) CONCLUSION

"The foundation has failed because I say it has failed" This statement reflects the essence of where we are today.

"The foundation has failed. I say so, because:--" This is where we are headed.

There is no substitute for individual knowledge and experience. However, it is limited and varies greatly from one individual to another. It is an accepted fact that individuals will give widely varying opinions on the same set of criteria when coming solely from their own knowledge and experience.

In order to minimize the damaging consequences which come from this divergence, we (our industry) must provide guidance for engineers and others who are called on to give their opinion on the condition of or performance of a foundation, especially if their opinion is to be expressed in a courtroom.

We have found it easy to arrive at an agreeable word definition for "Foundation Failure". It is based on the simple premise that if a foundation is performing its intended function, it is stable. If it is not performing its intended function it is not stable and can be said to have failed.

Conversely we have found that development of guidelines for measuring performance is somewhat complex. We have said that the key elements or variables to be used as a basis for development of guidelines are movement and time. When combined these words by definition determine stability

Since the critical nature of the need for guidelines is largely a consequence of consumerism (homeowners filing lawsuits under the Deceptive Trade Practice Act - DTPA), it is not only appropriate but absolutely necessary to base performance guidelines, at least in part, on criteria used to establish such damage claims. Namely, the claim that the foundation did not perform up to the homeowners' rightful expectations.

And no where is the homeowners expectations of performance (or non performance) manifest in a more concrete way than when he signs a check for major foundation repair or when he demands someone else sign the check (such as in homeowner claims against builders or homeowner claims against previous owners). Many others can be, and usually are, at the affect of this demand and are therefore becoming overly sensitive to "Homeowners Expectations" and of the possible consequences of misjudgment of same.

Homeowner expectations of foundation performance are based primarily on his observations of the following:

* Floor slopes and other out of level conditions.

* Sticking, non-latching or otherwise dysfunctional doors.

* Cracks in rigid interior and exterior wall materials

* Cracks in foundation concrete.

* Separations of adjoining materials.

* And other conditions to a diminishing degree.

All of the above are a consequence of movement. They are quantifiable. The degree of normalcy with which the homeowner views these is influenced by the age of the house (time). He is also influenced by the general aggregate attitude or opinions of others in the community, such as Realtors, appraisers, mortgage lenders, inspectors, and others who have a reason to question stability of foundations of the houses with which they deal.

Surveys already conducted and which are underway have not yet provided sufficient results from which meaningful guidelines can be derived. But since even a beginning set of approximate quantities is better than no guide at all, the writer will attempt to combine his own personal knowledge and experience with limited results from surveys and apply quantities to the above list of homeowner expectations:

* **Floor slopes** -- The average person cannot perceive floor slopes smaller than 1/4 inch in 4 ft. Most persons will perceive floor slopes of 1/2 inch in 4 ft. Almost everyone will object to floor slopes in the 3/4 to 1 inch in 4 ft (and the vast majority of foundations will have been repaired prior to reaching this degree of slope).

* **Movement** - Using the above floor slopes to derive **vertical displacement** we have a majority of foundations being repaired by the time movement has reached 3 inches in the San Antonio area. In the Houston area, the current indications are 2 inch (see chart on Page 3). It has also been observed that the average movement for all houses in the general Houston area, with time, is approximately 1/16 inch per year, resulting in 2 inch to 3 inch movement occurring in 30 to 40 years.

* **Doors** - 1 to 3 doors are likely to be sticking, non-latching or otherwise affected when foundation repair is performed. (this will vary with the tightness of the fit at time of construction of the door)

* **Cracks** - not yet repaired cracks in interior wall materials at time of foundation repair range in number from 3 upwards and in size from 3/16 inch upwards.

Cracks in exterior walls at time of repair range in number of 3 upwards and in size from 1/4 inch upward at time of repair.

Cracks exist in essentially all foundation concrete to some degree. Size of at least one crack at time of repair will be in excess of 1/16 inch The majority of foundations will usually have been repaired prior to size reaching 1/4 inch.

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* **Separations** - At time of repair, separations of adjoining materials can usually be observed at three or more points and 1/4 inch or more in size.

It is pointed out that the above are not intended or presented as tentative guidelines but rather as a statement of the writer's observation in his extensive experience looking at foundation failure problems. These are presented solely as a beginning point from which it is hoped that guidelines for the Houston area can be developed.

In closihg, it is pointed out that it is not the intention of the Foundation Performance Committee to be the entity which develops widely accepted guidelines for a wide area. Rather, it is our intention to make a meaningful contribution to the industry in general which clearly is in the process of developing foundation performance guidelines.

The bottom line opinion or statement that a foundation has failed still does and will continue to lie with the PE (registered professional engineer) who will provide his subjective opinions based on his own knowledge and experience including the laws of science and engineering. It will, however, become more and more incumbent on the engineer to broaden his knowledge to include more careful consideration of "what other people are doing" including activities of the Foundation Performance Committee and of surveys such as those mentioned in this paper.

To state "It has failed because I say it has failed" is no longer acceptable. Within the near future there will be no option for the engineer other than to make reference to quantifiable guidelines as the basis of his opinion.

It is happening and it is happening now. You are invited to join, to make a contribution. If you wish to participate in surveys or in any other way , please call Jack Deal at (713) 667-1158.

FOUNDATION FAILURE IN THE HOUSTON AREA

by

RICHARD W. PEVERLEY
PEVERLEY ENGINEERING, INC.

RESUME

Richard Peverley is a Registered Professional Engineer, not only in the State of Texas but also in the State of California. He graduated from Colorado State University in 1954 with a degree in Civil Engineering. After spending three years in the Air Force as a pilot and radar controller, he spent three years in the aerospace business as a structural engineer, reliability engineer, and project manager working for Norair, Martin Marietta, General Electric Company, and Boeing Aerospace. He was the project engineer for the NASA Reliability, Quality Control, and Safety Division on the Apollo 13 and 15 Missions. Peverley then moved into the engineering and construction business working for Bechtel Power Corporation and Brown & Root, Inc. He ended his career with Brown & Root as an Assistant Engineering Project Manager on the South Texas Project. He founded Peverley Engineering, Inc. in 1975. This firm, which has 3 Engineers, is engaged in property inspections, structural design and consultation, and forensic engineering. Peverley is a member of the American and Texas Society of Professional Engineers, American Society of Civil Engineers, American Concrete Institute, and the National Academy of Forensic Engineers.

FOUNDATION FAILURES IN THE HOUSTON AREA - 1994

Richard W. Peverley
Peverley Engineering, Inc.

Presented at the
SOILS-STRUCTURE AND INTERACTION SEMINAR
FOR RESIDENTIAL AND LIGHT COMMERCIAL FOUNDATIONS
Houston, Texas
July 28, 1994

INTRODUCTION

In the month of August, 1993, Hurricane Alicia passed directly over the City of Houston, Texas and caused damage estimated to be as high as $600,000,000 to homes in a 6 county area (1). In 1987, we estimated that individuals spent approximately $28, 500,000 to repair foundations in the greater Houston area in conjunction with the sale of their homes (2). One could then conservatively conclude that at least half that amount of money was spent by individuals who were not selling their home but simply wanted to repair them. Since hurricanes seem to occur on a 10 year cycle in the Texas gulf coast, one could then logically opine that $456,000,000 was spent on foundation repair, or 75% that spent to repair the damage by a Force 3 hurricane. It is no wonder that there were 117 firms listed under the heading of "Foundation Repair" in the September 1993-94 Southwestern Bell Yellow Pages. It is significant to observe that most of the money paid to repair the damage caused by hurricanes comes from insurance companies and governmental agencies; whereas, most of the dollars spent on foundation repair came directly from the individuals who owned the home that was repaired.

We have been in the business of inspecting residential structures since 1976. This means that we have inspected somewhere between 35,000 to 45,000 foundations. It is easy to get the feeling that we have seen every type of failure that there is; then, a new situation presents itself and our ego is summarily brought into the realization that there is no standard for foundation failures and that they occur for a seemly endless variety of reasons. In this paper, we have presented some case histories where a foundation failure has occurred. These are only examples. Any attempt to present even a representative example of all of the various types of foundation failures that have occurred in the greater Houston area would be a text book undertaking. This is then followed by a discussion of failure analysis, some pertinent observations, and our opinions and conclusions regarding this subject.

We have <u>not</u> attempted to define the term "foundation failure" in this paper. We are, in fact, of the opinion that to achieve such a definition may not be feasible. In all of the cases we are presenting, the foundations are assumed to be in a failed condition.

CASE STUDIES

CASE NO. 1 - IMPROPER SOILS PREPARATION

This case is a residential building located in Kingwood, Texas. It is a two story dwelling with a reinforced concrete slab-on-ground foundation. The foundation was measured to be significantly lower in the center of the building than around the perimeter, which, by itself, was not level. This condition is diagramed in Figure 1. There was a significant amount of foundation induced damage observed. Soil testing showed that there were fill soils under the building which had a Plastic Index of from 38 to 52 percent. The soils near the perimeter had a Plasticity Index of from 18 to 38 percent. The soils under the foundation were significantly drier than those at the perimeter. The Soils Engineer concluded that the problems with this foundation occurred because of soils swelling at the perimeter.

The foundation was repaired using pressed, interconnected piles. There were 51 piles placed under the perimeter grade beams and 35 placed under interior grade beams at a cost in the neighborhood of $25,000, for the foundation repair only. The cost to fix the damage created by the repair process is not known, but would be extensive. The cause of this problem was the failure to construct a foundation that was appropriate for the soil conditions that existed and the improper use of fill soils.

CASE NO. 2 - INADEQUATE DESIGN

This case concerns a two-story residence located in the city of Fulshear, Texas. This was a custom designed and constructed building for the owner. There was no previous building on this site. Shortly after the owners moved in, the walls began to show cracking, separations occurred, and some doors became difficult to close. The relative heights of the foundation slab were measured and the results are shown in Figure 2. Obviously, there was a significant amount of subsidence that occurred in the center part of the building. Soil testing showed the soils under the foundation to have low strengths and to be moisture sensitive; whereas, the soils around the perimeter had high PI's and good strengths. The concrete strength was only 2,000 psi versus the 3,000 psi that was specified.

The basics of the foundation design are shown in Figure 2. Of significance, the interior grade beam stiffeners were only 12 inches deep and 6 inches wide. There was no reference to a soil test indicated on the drawings. In our opinion, the design was not appropriate for the soil conditions at this site and the low concrete strength precluded repair. No attempt was made to repair this building. Instead, it was torn down.

CASE NO. 3 - POOR DRAINAGE PRACTICES

The building that is the subject of this analysis is a two story dwelling which is located in the Village of Southside Place, which is an enclave of Houston, Texas. This was a custom designed and constructed building whose foundation was a reinforced concrete, grade-beam-stiffened, slab type of foundation supported on drilled piers. The foundation plans contained the Seal of an Engineer and a soil test was referenced on the plans. These soils had a very high plasticity index.

Inspections of the foundation had been made in 1988 and 1989 to assess cracking in the floor tiles. An inspection made in 1991, however, showed the presence of a significant amount of damage. The contour height measurements that were made during the 1991 inspection are shown in Figure 3. Obviously, the foundation had heaved upward along the rear wall. Testing showed that the contractor had failed to use a select fill, as specified. Of more significance, it was learned that the owner had a drain sump installed in the back yard. The drain pipe was routed to the drain line that the builder had previously installed at the edge of the foundation; however, the new pipe was never connected to the original pipe. As a result, any rainwater that was collected in the yard sump was routed directly to the edge of the foundation and hence down the sides of the piers.

There were two errors which contributed to the problems with this foundation. One was the placement of the non-select fill; however, since the heaving was limited to the area where the non-select fill existed, once can but conclude that the error in the placement of the fill was not the major contributor. To the best of our knowledge this foundation has yet to be repaired.

CASE NO. 4 - CONSTRUCTION DEVIATIONS

This was a two story residential building which was constructed in the early 1990's. The building plans showed the foundation to be a reinforced concrete, grade-beam-stiffened, slab type supported on drilled piers. The soils were known to be very expansive (3).

The building began to develop some cracking early in its life and, based on a recommendation from a foundation repair contractor, pressed piles were place under one of the front corners. The cracking continued. The results of the foundation height measurements we made on this project are shown in Figure 4. The owner then hired an engineer and excavations were made at several locations around the perimeter of the building. Among other things, it was discovered that there were significant gaps between the top of the piers and the bottom of the grade beams - some gaps being as wide as one foot. The foundation grade beams were also found to be of a height that was somewhat less than specified. No meaningful data were acquired that would lead to any conclusions with regard to the placement of the reinforcing steel.

The builder then agreed to have the foundation repaired. Bids were acquired to have the foundation underpinned using pressed piles with the interior piles being placed using tunneling techniques. A question was raised concerning the potential vertical rise of the soil near the center of the foundation. The soil testing showed the moisture content of the soil to be 6 to 10 percent above the plastic limit. It was concluded that the soil under the slab was near an equilibrium moisture content; however, there was also a concern that the potential vertical rise could be as much as 4 inches. The foundation repair contractor was asked to revise his bid to include raising the entire foundation 4 inches. This bid was equal to approximately 50% of the cost of the original construction. It was then decided that stabilization would be achieved using a vertical moisture barrier. The design of this barrier is shown in Figure 4. The builder agreed to also underpin the foundation around the perimeter and under the interior of the foundation where point loads existed; however, no serious attempt was made to raise the foundation any significant amount. The vertical moisture barrier was installed with the total cost being approximately 50% of the bid cost to raise the building 4 inches. The long-term results are yet to be assessed.

CASE NO. 5 - FRAMING PROBLEMS

This case concerns conditions where the foundation was incorrectly blamed for problems that had to do with framing. The first example is a three story residential building. The slab type of foundation was supported on drilled piers. At the time of our inspection, the building was so new that it was in its final stage of construction. Relative height measurements were taken on all three of the floors and the results are shown in Figure 5. On the first story, there is an unexplained drop in the foundation at the outside, front corner. On the second and third stories, there is a comparatively steep change in floor height of 1.4 inches over a distance of less than 4 feet at the top of the stairs which had been blamed by someone else as a foundation problem. The relative height measurements taken on the first story proved this finding not to be valid. The drawings showed the loads from the roof and second story floors to be supported by two pieces of lumber which had a 22 foot span. Obviously, something went wrong during the construction of this building and nothing was apparently done to correct these conditions before the building was presented for sale.

The second example concerns a 20 year old building which was resting on a loam soil that had incurred some deflections along the rear wall, apparently because the soil became overloaded. The measured slopes are shown in Figure 6. The most severe damage was not on the first story, however, but on the intersections between the two story interior walls and the ceilings. An examination of the framing in the attic showed the reason. From Figure 6, it can be seen that there were no collar ties and that the downward movement of the foundation caused the rafters to deflect from an overload condition. Thus, fixing the foundation alone would not have totally resolved this problem.

CASE NO. 6 - REPAIR OF A HOME ON A SEVERELY SLOPING LOT

This case is of interest not only because of its uniqueness but also because of the method of repair that was used. The house sits on the edge of Buffalo Bayou and, as shown in Figure 7, and the ground drops sharply downward from the outside of the kitchen area to the bayou approximately 25 feet below. The floors in the kitchen had a drop of 7 inches over a distance of approximately 20 feet, as shown in Figure 8. In the second story over the kitchen area, there were significant distortions and separations in the door frames which provided evidence that this was not a built-in condition.

Soils testing was done. It was the opinion of the soils engineer that a classic slope failure had not occurred but, instead, soils had been lost from under the grade beams because of erosion. It was recommended that the bottom of any underpinning be at a minimum depth of 15 feet below the surface of the soil at the edge of the grade beam. After assessing the available options, we recommended to the owner that perhaps the best option was to use Chance helical anchors not only because of their comparative installation ease but also because more is known of their installation characteristics than is known of other systems. The recommended location for these anchors is shown in Figure 8. The diameter of the helical plate was initially 30 inches. Steel stiffeners were attached to the side of the grade beams, as shown in Figure 8, because of cracking that had occurred in the grade beams.

The only installation problem occurred when it became apparent during the raising when the foundation was not properly responding to the lifting forces. It was determined that the anchor pier had either been installed at an improper angle to the vertical or the depth was not adequate. Additional piers, which had a smaller diameter, were then driven deeper into the soil and at a more true vertical plane to provide temporary support for the building while the first set of pier were screwed out of the ground and reset at a true vertical plane. The foundation was then successfully raised to within two inches of a horizontal plane. The foundation raising was stopped when the counters became level and there were audio indications that excessive stresses were being induced into the framing.

CASE NO. 8 - FOUNDATION FAILURES WHERE REPAIR ECONOMICS ARE QUESTIONABLE

This case concerns foundations that have failed but the cost of repair is sufficient to raise questions of economics. The first example is a residence that is approximately 40 years old and is located in the Southwest part of Houston, Texas. The results of the foundation height measurements are shown in Figure 9. There is a significant amount of foundation subsidence in the front, and front center, of the building. There was a sufficient amount of cracking and distortions in the framing to conclude that the major degree of the slope was not built-in.

This case presents a dilemma since the cost of investigation and repair could equal approximately 25% of the market value of the property. Certainly, the foundation is repairable using drilled piers or driven piles; however, such repairs would not be prudent until the cause of the failure has been found. This building lot is adjacent to a bayou and the existence of a defect in the soil can not be discounted. Thus extensive testing will be required.

The second example is a comparatively small residence in a neighborhood in Southwest Houston where the average sales price of homes is on the order of $90,000. The owners, who are retired, are planning to put their home on the market. There is a significant sloping condition on the interior floors despite some foundation repairs having been done, as shown in Figure 10. The home is in pristine condition and there is no evidence of recent foundation shifting. The cost of repairing this foundation would be on the order of $15,000 to $20,000. Thus, it is questionable that there would be anything to be gained by spending this much money to attempt to simply level the foundation that is otherwise stable.

CASE NO. 9 - HOUSE ON A FAULT

This is the type of case that can bring tears to your eyes. This is a delightful residential building that is located in an upper middle class neighborhood in the West part of Houston, Texas. When we inspected the property for a potential purchaser, we observed a sharp rise in the floors at one isolated corner of the building, as shown in Figure 11. Since we had inspected the adjacent building which was obviously located across a geological fault, we were left but to conclude that the sharp rise in the floors was fault related. This was later confirmed by a Geologist who specializes in fault related activities.

The dilemma is this case is what to do. It is inconceivable that the remainder of this foundation could be raised to be level with the high spot. Even if it could, there is no way we know of to prevent further fault induced movement. To the owners consternation, this is an unmapped fault whose existence is not widely known. As stated in a Kriss Kristofferson ballad - whose to bless and whose to blame.

CASE NO. 10 - THE USE OF SPREAD FOOTINGS

During a recent meeting of the Soils-Structure Interaction Subcommittee, there was a discussion on the use of spread footings. I promised to provide information on buildings I had inspected with such foundation types. So, here they are.

The first example is a very large home located in the Farnham Park subdivision, where home dollar costs can be on the order of seven digits. The relative heights we measured on the interior floors of this building are shown in Figure 12. It is very obvious that the floors are very flat. In addition, there was no visible evidence of foundation shifting. The building plans showed the foundation to be a reinforced concrete, grade beam stiffened slab type supported on spread footings. The soils are known to be of a loam consistency on the surface with strong clays from 6 inches to a depth of 6 feet below the surface (3).

The second example is a residential building located in the River Oaks Subdivision of Houston, Texas. The soil type and the foundation design, at least according to the drawings, was similar to that described in the first example; except, this structure was over 50 years old. Further, it had not been occupied for at least 2 years and, although it was in a general run-down condition, there were no cracks in the brick, no cracks in sheetrock, and all of the doors fit well in their frames, despite the fact that no one had watered the ground next to the slab nor had pruned the trees. The floor height measurements, contained in Figure 13, show the floors to be comparatively flat.

Certainly, this can not be considered to be a significant statistical sample; but, it does serve to demonstrate that there is perhaps a place for this type of foundation design.

CASE 11 - SLOPE VERSUS FAILURE

A concern was recently made that perhaps too much emphasis was being placed in the use of water level measurements in determining the need for repair. Since I can only deny this allegation as it might apply to our firm, I wish to cite the two following examples.

The first example is displayed in Figure 14. This is a one story building that has a comparatively high degree of slope on the interior. The building is located in West Houston on a soil that is shown to be relatively non-expansive. Our clients had executed an Earnest Money Contract on the property and our inspection was in conjunction with the sale. Despite this slope, there was no foundation-induced damage on the interior or exterior of the building. Counters, frames, and sills were level. In fact, the kitchen counter was measured to be over one inch higher off of the floor at one end than at the other. Obviously, the counter was installed over a sloping floor. We understand that our clients did purchase the property.

The second example is just the opposite. From Figure 15, it can be seen that the floors are comparatively level. Yet, there was a comparatively severe amount of damage to the front of the building. The damage seemed to indicate that the front showed the soils to be expansive. The movement apparently occurred when gutters were added. The sequence of events that caused the damage must be speculated. Certainly, corner had settled but the measurements indicated that it had not. Soils testing

the water level measurements failed to shed any real light on any answer that may be forthcoming except to eliminate settlement as the cause of the damage.

CASE 12 - THINGS THAT GO BUMP IN THE NIGHT

This case concerns two events which we, as structural engineers, must defer to our geotechnical brothers for answers.

The first example concerns the upward movement of bell-bottomed piers. Figure 16 shows the results of two sets of floor height measurements taken on a relatively new home with a pier and beam foundation. Near the center of the building, there has been an upward movement of the foundation of 0.8" between measurements made six months apart. On one of the outside corners, there has been an upward movement of 0.4 inches over the same time period. There were no other upward movements of any significance in the building. There were 42 inch diameter bells at the bottom of the piers where the upward movements occurred. The piers were inspected by the design engineer at the time of placement. What is going on?

The second example is a two story home with a garage that is detached and on its own foundation in the first story but attached in the second story. The foundations are supported on correctly designed bell-bottomed piers. The soil is expansive. There was an older building on this lot. The original soils testing was done in the front yard. The water level measurement results of the first and second visits we made to this site are shown in Figure 17. The foundation had subsided at the rear of the building and at the front of the garage. There was a severe amount of damage. The foundation was underpinned using pressed piles. We returned several months thereafter only to find the sloping condition to have worsened. A third visit was then made two weeks later and we found that the residential foundation had further subsided 0.5 inches to the rear. The builder has commissioned some soil testing to determine what is going on. Hopefully, we will find out.

FAILURE ANALYSIS

In most manufacturing processes, there was a quality control tool that was once used called a bath tub curve. It was based on a plot of the number of failures that occurred on an item versus time. A typical bath tub type curve that may have existed for an automobile manufactured in the United States in the 1970 to 1980 time period is shown as Curve 1 of Figure 13. It is not to difficult to remember that it was necessary to take a new American car back to the dealer after it had been driven a short while to have the "bugs" repaired. Also, how frequently was one able to drive an American car of this vintage over 1,000,000 miles without an engine overhaul or at least one major repair? We refer to this as a past event because in the mid-1980's, the American consumer began to realize that automobiles manufactured by the Japanese did not have such baggage to contend with. A comparison of an American and Japanese failure curve is characterized in Curve 2 of Figure 13. American manufacturers were then moved, by market forces, to adopt those types of quality control program that the Japanese found to be so successful; i.e., a total commitment of management to quality, effective inspections, failure analysis and recurrence control, supplier commitments to quality, quality circles, etc. and, as a result, American automobiles have approached the same quality as their Japanese counterparts.

For the purpose of our analysis, we can place residential foundations into the three following groups; the pre-war pier and beam types, the post-war slab-on-ground types built on expansive clay soils, and the more recent pier supported concrete slab types. We must limit the second group to being founded on expansive soils since the failure rate of slab type of foundations on non-expansive soils is very different and is often very low. The first group are too diverse for analysis. In our experience, the failure rate of the second group resembles the classic bath tub curve, as shown as Curve 3 in Figure 13. The infant mortality rate is difficult for us to determine because we were not in this business at the time most of these homes were constructed and the infant mortality failures occurred.

We have observed, however, that the time it took for the curve to begin to climb was approximately 20 years. Houstonians value trees and it takes most trees approximately 20 years before they become large enough to drain a significant amount of the moisture from under the exterior grade beam of a foundation. During this time period, it was also probable that this area under went an extended drought, the interior sheetrock began to age, and brick mortar began to fatigue. As a result, it was a forgone conclusion to many homeowners that it would be necessary to have their foundation repaired during that time period. What is disturbing to many of us that the infant mortality rate of some of the newer homes in the greater Houston area is significantly high. An unanswered question is how long will it be until wear-out occurs.

OBSERVATIONS

One of the things that the Soils-Structure Interaction Committee has been attempting to do is to define what constitutes a residential foundation failure. This has been a difficult task and we may not be able to provide the type of definitive answer we had hoped to produce. There are too many groups involved that have diverse stakes in this such a definition that we may not be able to reach a total reach a consensus. Unfortunately, many of us in this business find our selves arguing over such things as the definition of such terms as unsanitary, unsafe, and uninhabitable. In such an argument, extremes tend to emerge. For example, a hut in Rangoon would certainly be more sanitary than any residence which only had a foundation problem. However, if foundation shifting caused a roof or a wall to leak rain water, is this a sanitary condition? Does a building have to fall down to be unsafe or is the fact that the structure was not designed to accommodate severe foundation deflections make it automatically unsafe? Is a dog pound habitable or is habitability something a consumer pays for when that consumer buys a home; i.e., is habitability a function of the price of a home or is there a minimum standard for any home?

CONCLUSIONS AND OPINIONS

The American automobile industry was forced to recognize the need for rigid quality control programs through the capitalistic process of competition. Certainly, there had to be those in the American automobile industry that said the American people would never pay more for quality. Has this statement been made in the residential construction business in the greater Houston area? There were also those in the American automobile business who said they would only make changes if their competitors did. I think we have also heard this statement made in the residential construction business in the greater Houston area? In my experience, and in our firm's experience, the home consumer is not only ready for quality construction but is

insisting on it. Law suits are the bane of every home builder. They have the same status in the engineering business. Many of us in the this business are learning what our clients expect and are providing the services that are in demanded. The result is more happy clients and less money being spent to defend law suits. I recognize that there are still some engineers who will work cheap and will cut corners as requested; but, they, like the rest of us will recognize the coerror of such an approach and will become fewer in number as time passes.

I believe that we will, through this Committee, continue to search for methods to improve the way in which we provide support to the residential construction industry. We can only urge that the builders in this City provide those means whereby out Clients can receive the product they desire - not the one we think they deserve.

/REFERENCES

1. Telephone communications with the U.S. Weather Bureau Information Service.

2. R. Peverley & D. Hanys:"Residential Construction Problems" presented at the Fall Meeting of the American Society of Civil Engineers, Houston, Texas, 1987.

3. "Soil Survey of Harris County" U.S. Soil Conservation Service, Department of Agriculture.

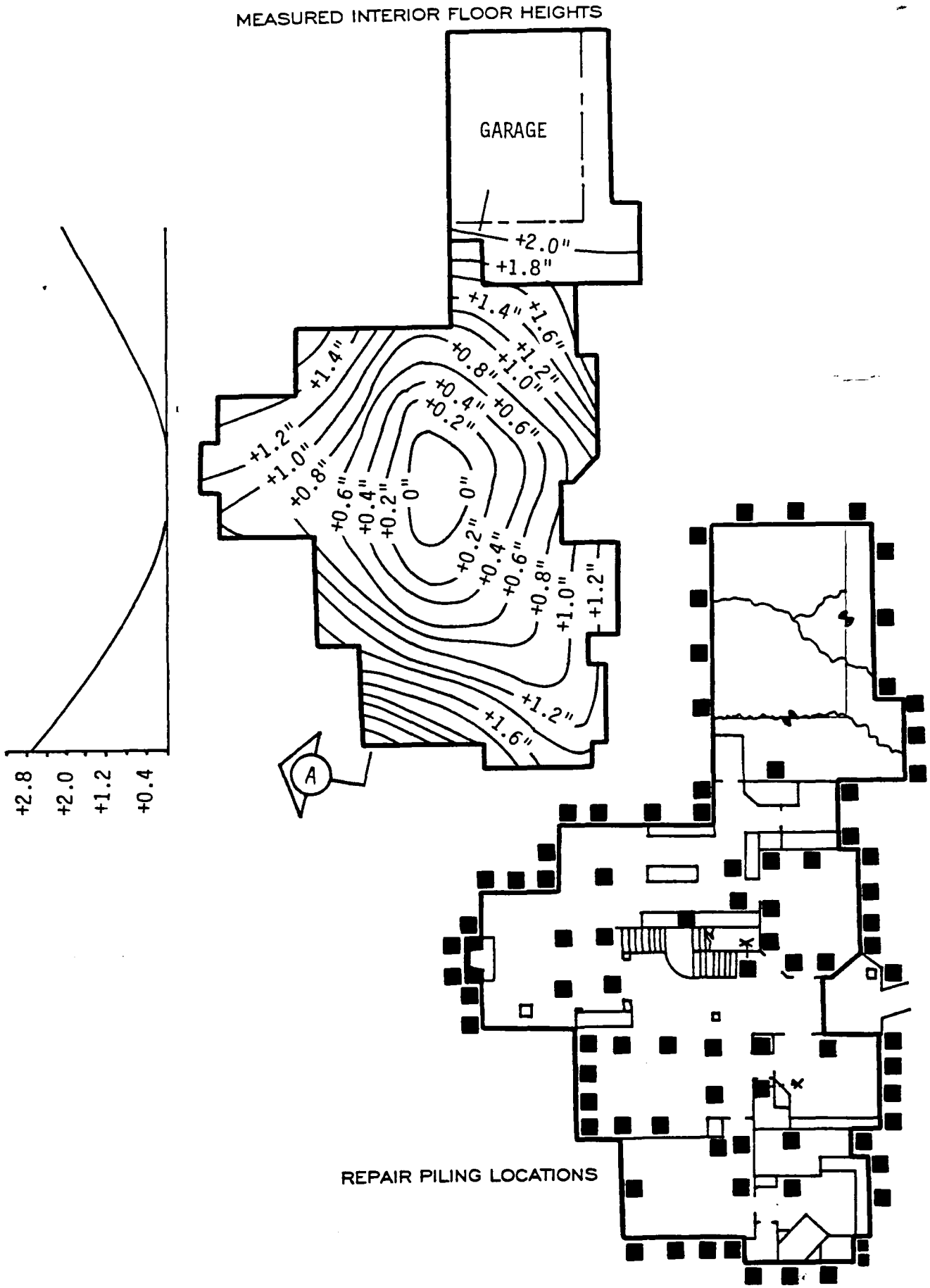


FIGURE I . AN EXAMPLE OF A FOUNDATION WHICH INCURRED SEVERE DEFLECTIONS AND REQUIRED EXTENSIVE REPAIR.

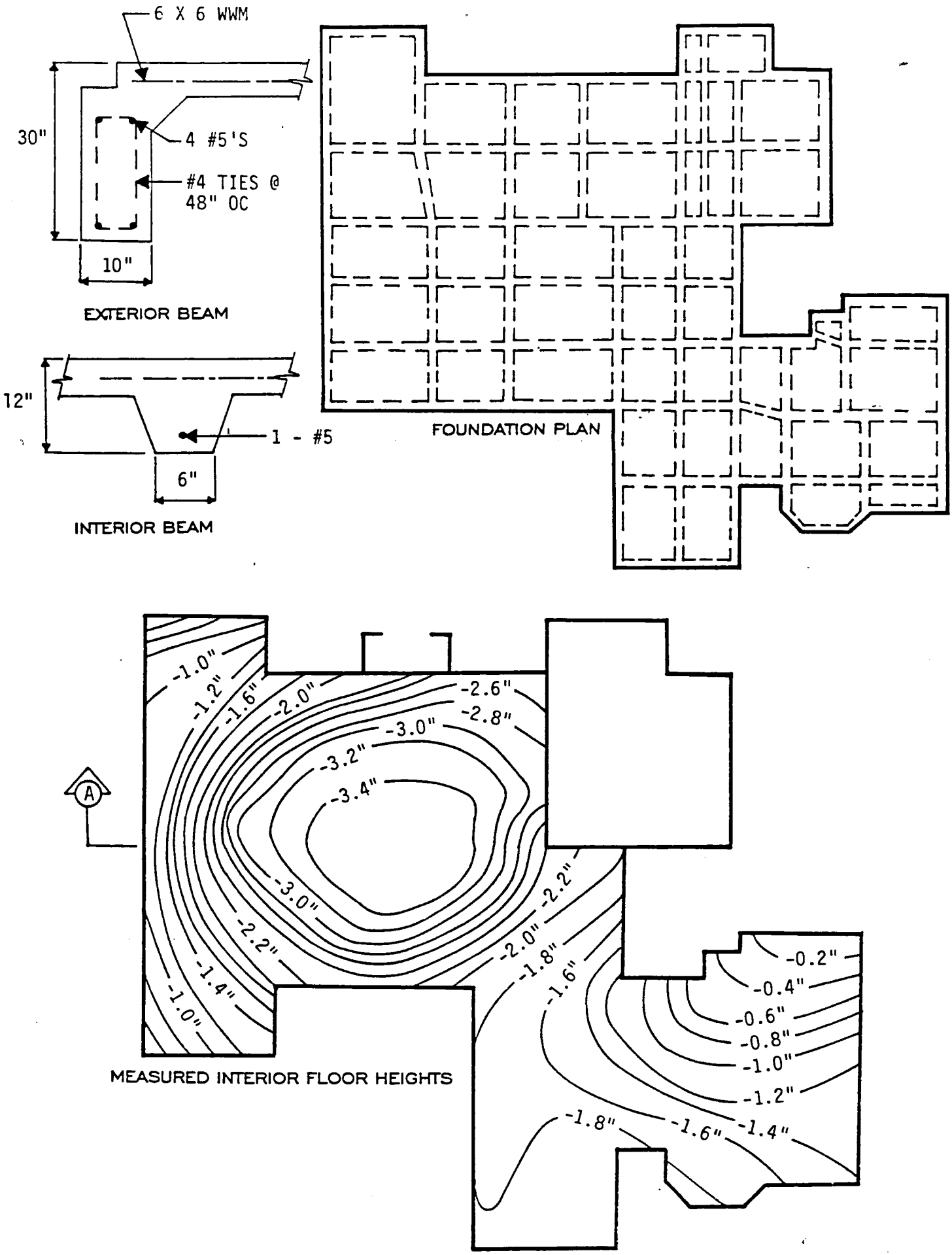


FIGURE 2. AN EXAMPLE OF A FOUNDATION THAT FAILED BECAUSE OF A COMBINATION OF DESIGN AND CONSTRUCTION ERRORS.

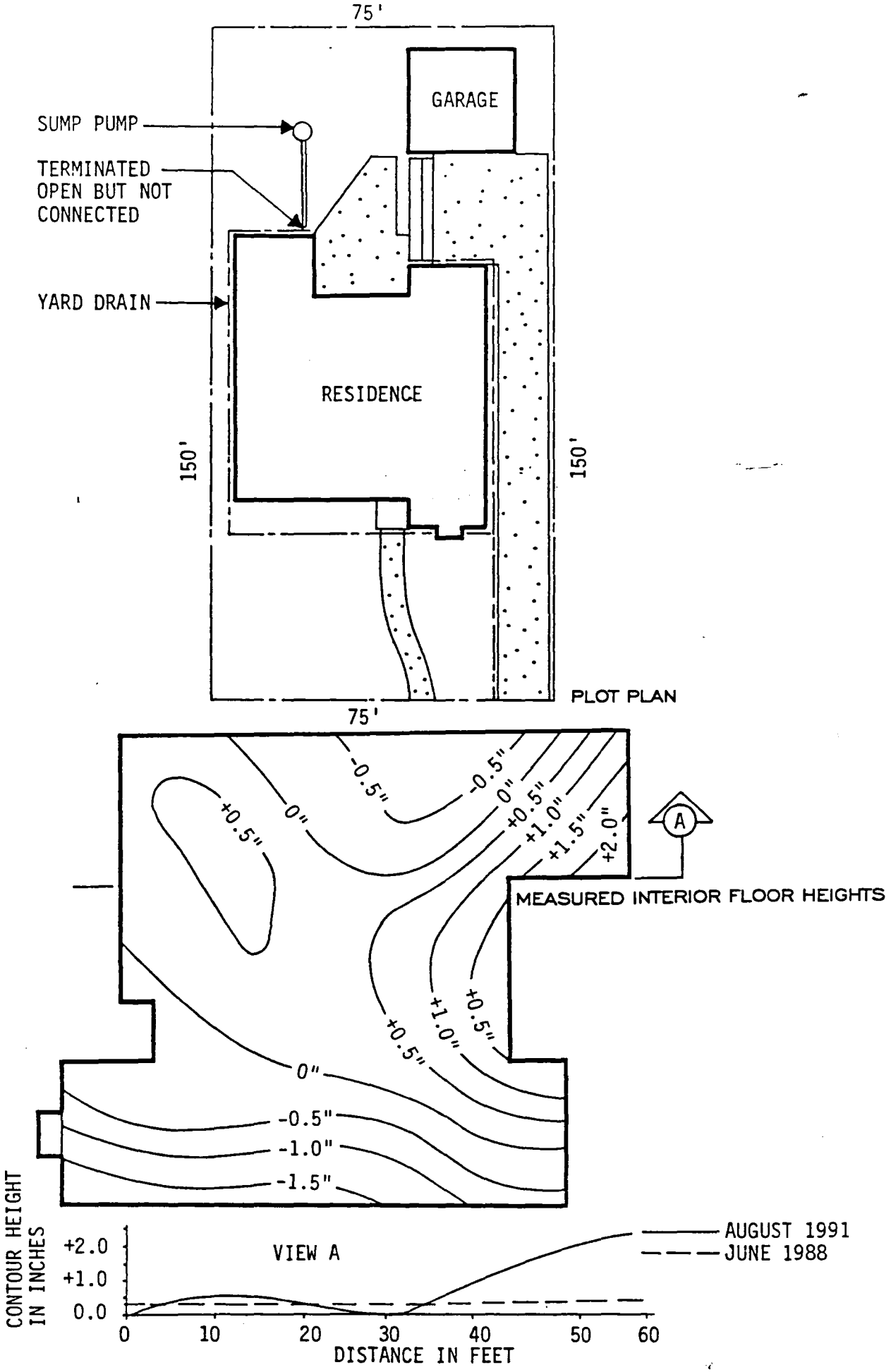
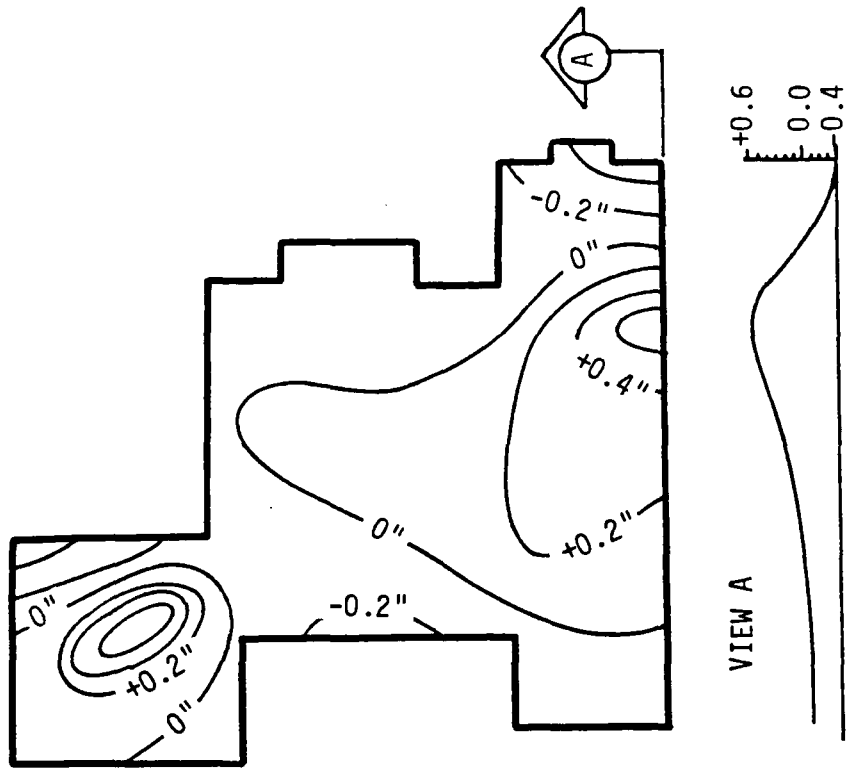
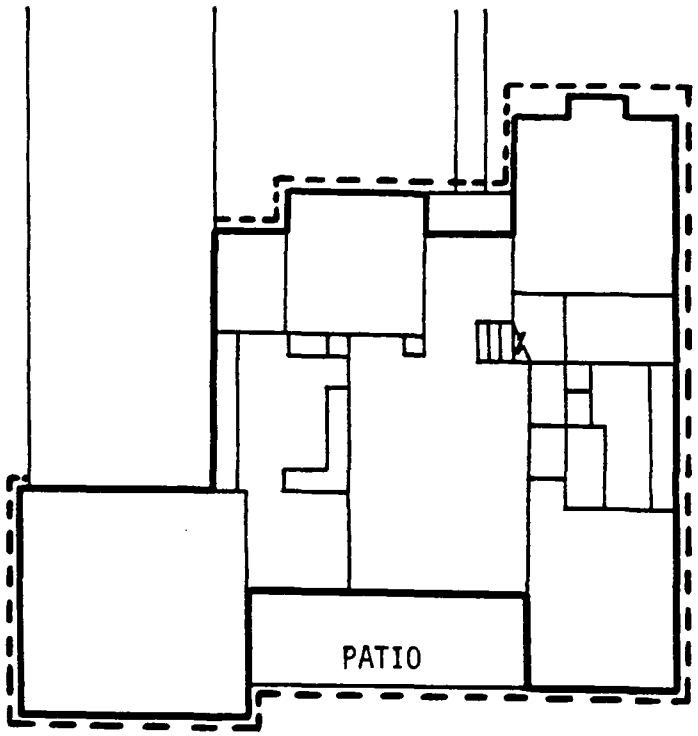


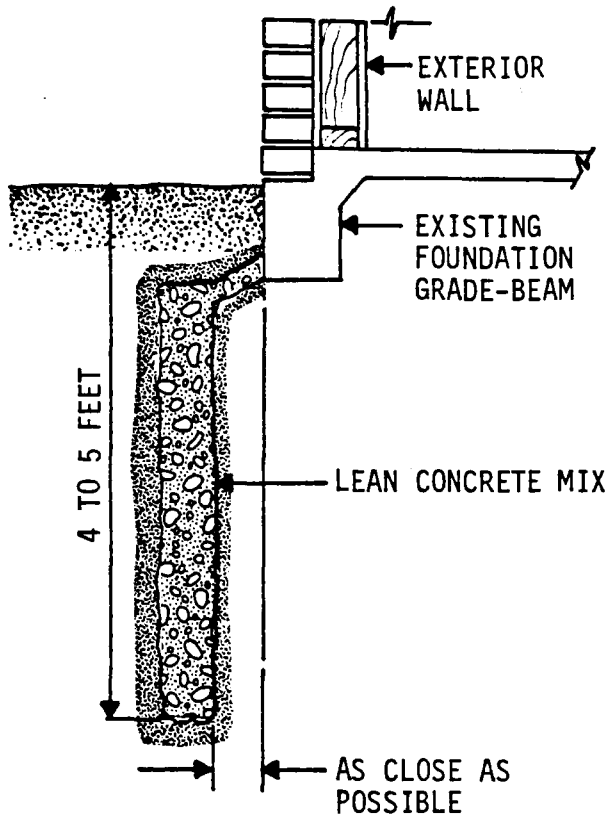
FIGURE 3. A PIER SUPPORTED SLAB TYPE OF FOUNDATION THAT FAILED BECAUSE THE OWNER HAD A YARD DRAIN SUMP IMPROPERLY INSTALLED.



CONTOURS OF EQUAL HEIGHTS



- - - - VERTICAL MOISTURE BARRIER LOCATION



CONCRETE SHALL BE ORDERED AS A 3000 PSI MIX WITH A FIBERMESH ADDITIVE

FIGURE 4. THE MEANS BY WHICH A PIER SUPPORTED SLAB TYPE OF FOUNDATION WAS REPAIRED USING A VERTICAL MOISTURE BARRIER COMBINED WITH PRESSED PILINGS.

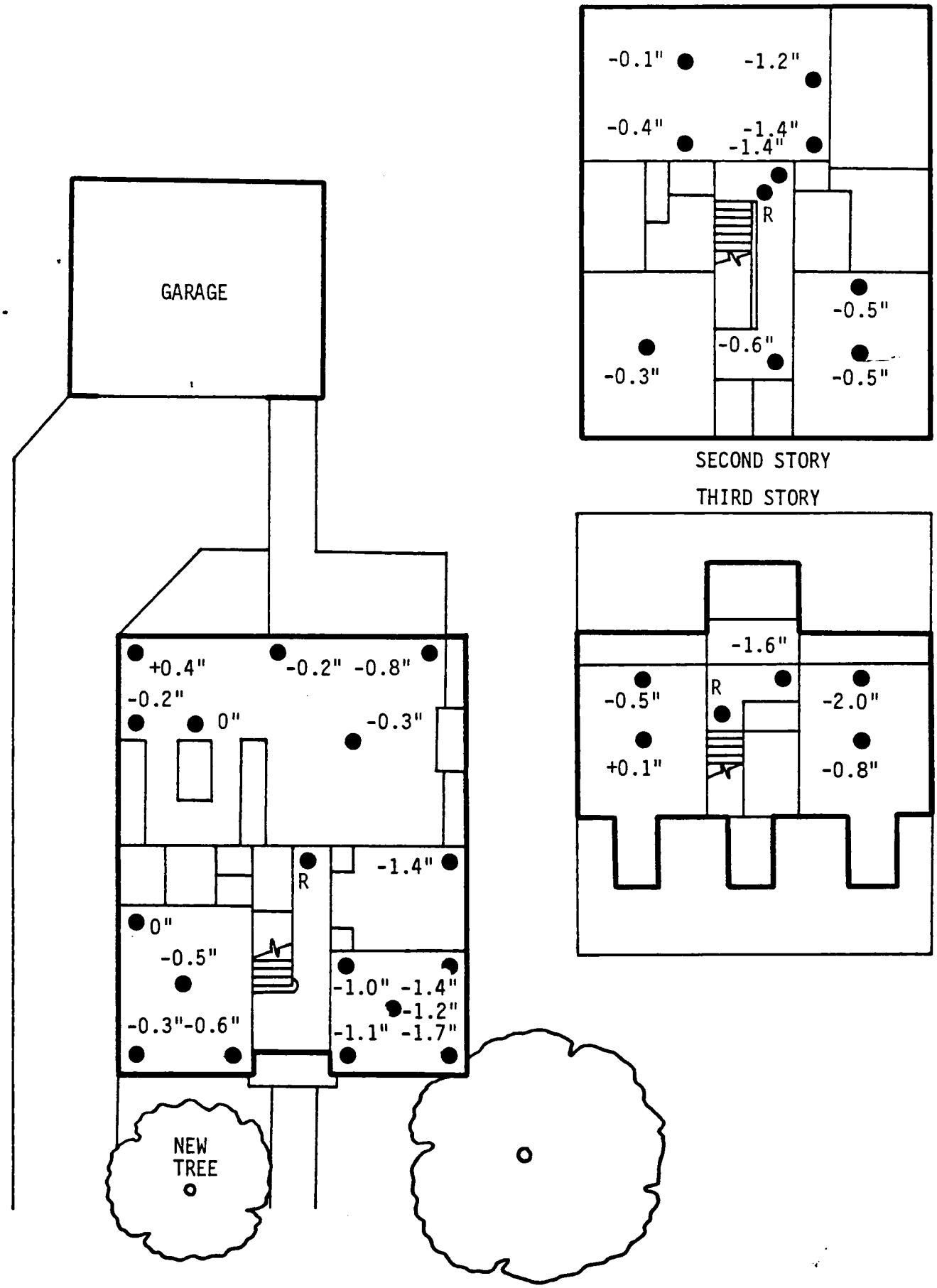


FIGURE 5. A NEWLY CONSTRUCTED RESIDENTIAL BUILDING WHICH HAS SHOWN EVIDENCE OF BOTH FOUNDATION AND FRAMING PROBLEMS.

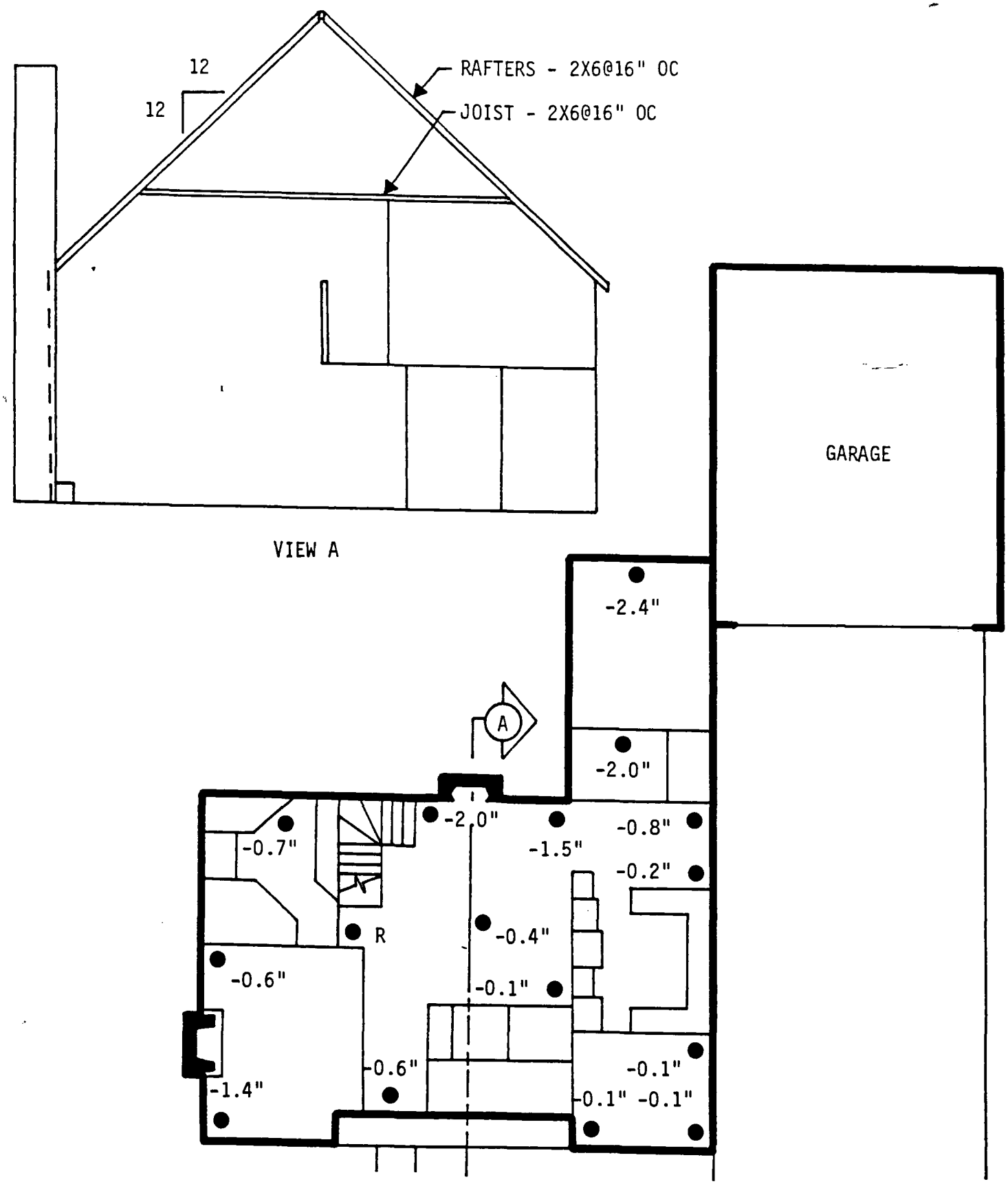


FIGURE 6. AN EXAMPLE OF A RESIDENTIAL BUILDING WHICH SHOWS HOW A LIMITED AMOUNT OF FOUNDATION DEFLECTION WHICH WAS EXACERBATED BY INADEQUATE FRAMING

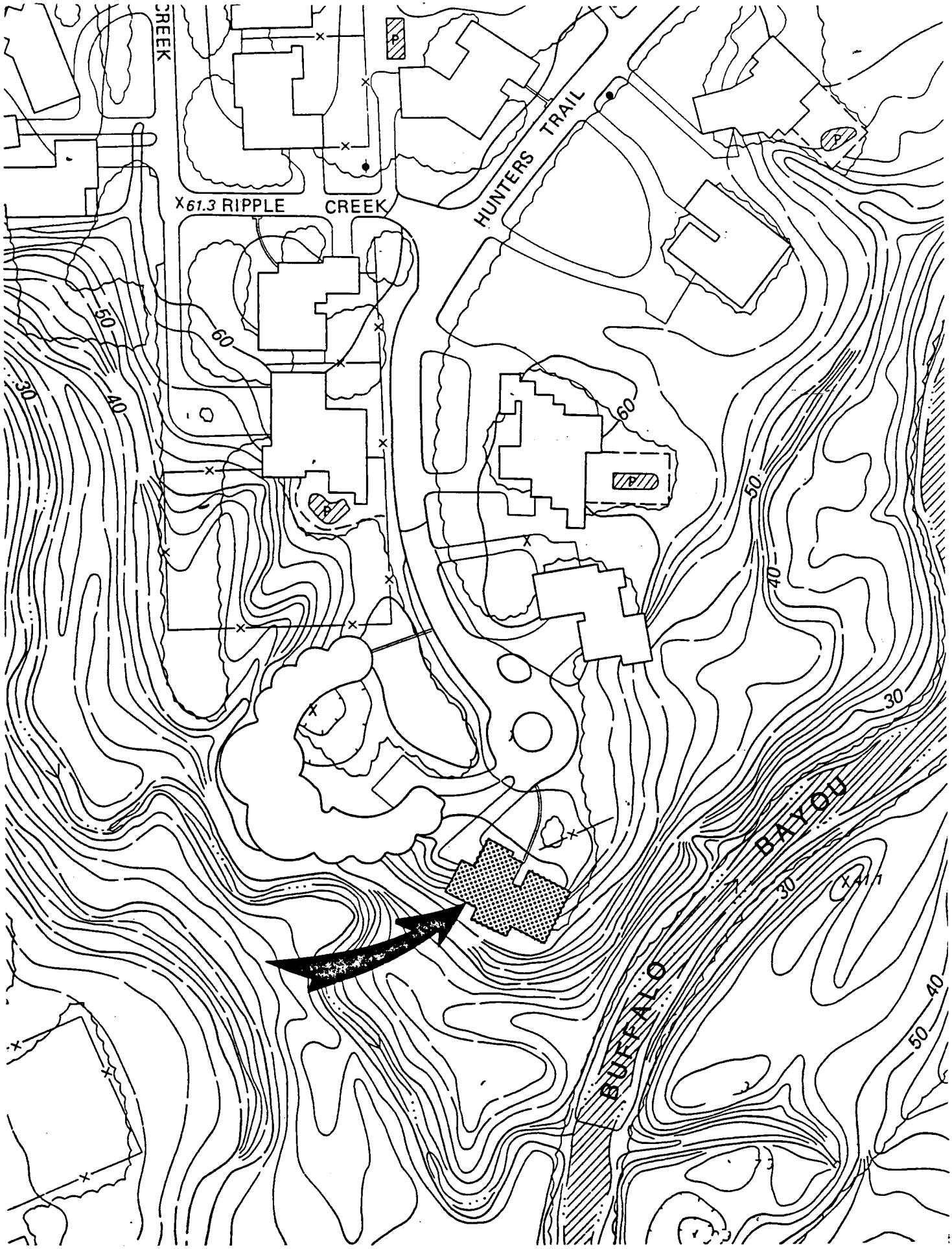


FIGURE 7. A RESIDENTIAL BUILDING WHICH HAS INCURRED FOUNDATION PROBLEMS AND IS LOCATED ON THE EDGE OF A DEEP BAYOU.

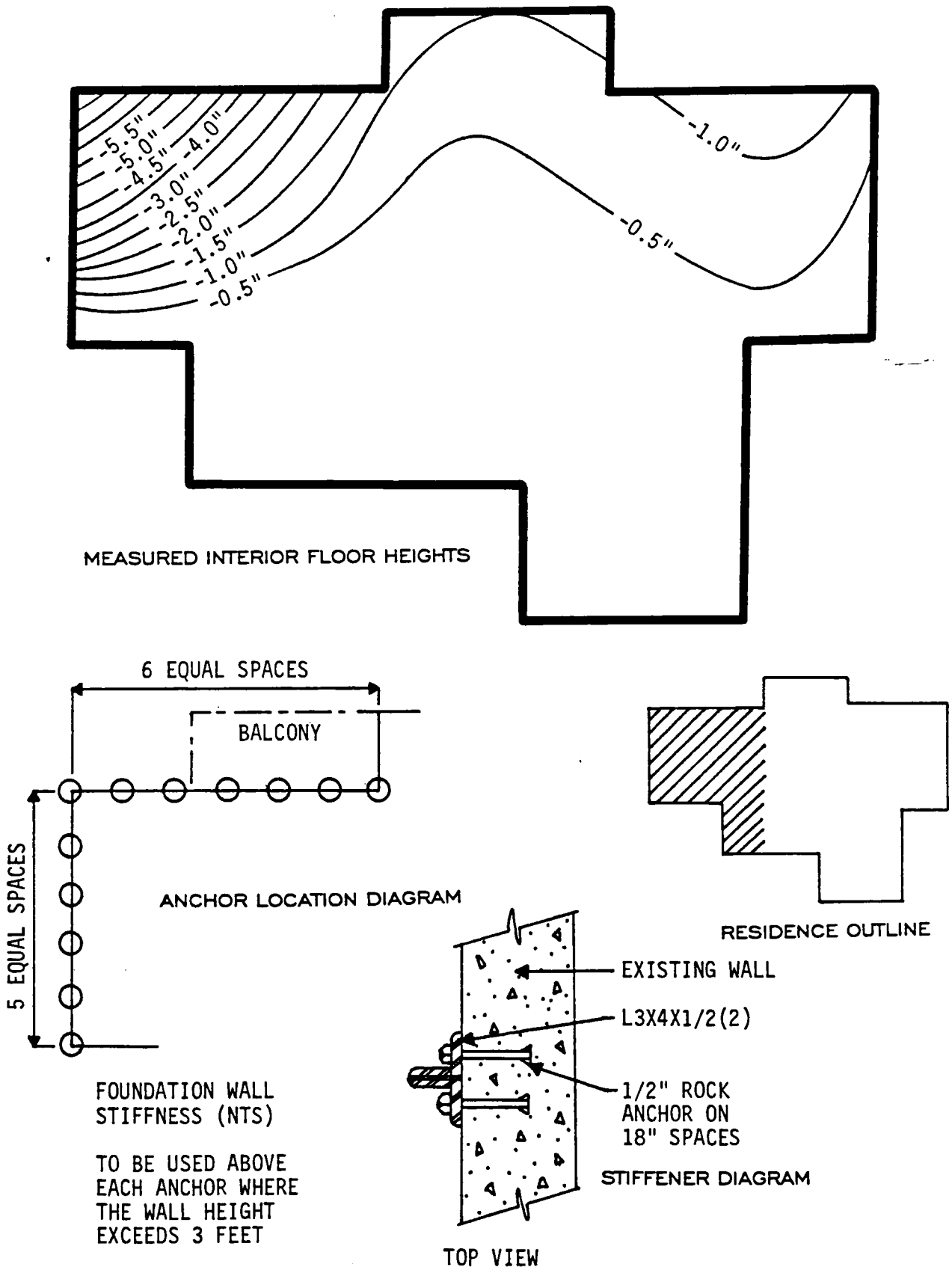


FIGURE 8. MEASURES USED TO CORRECT THE DEFLECTIONS THAT OCCURRED IN THE FOUNDATION OF THE RESIDENTIAL BUILDING SHOWN IN FIGURE 7.

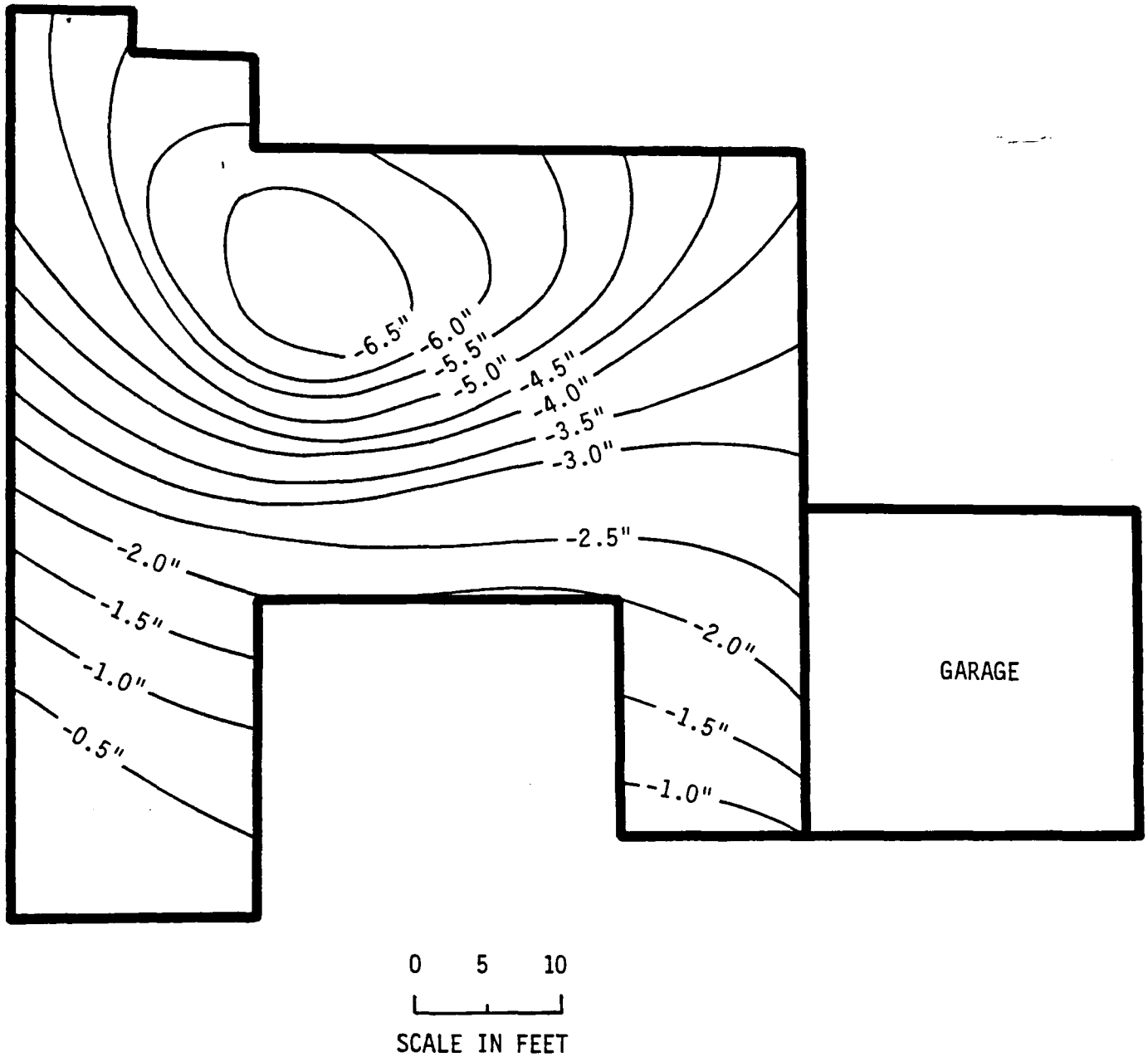


FIGURE 9. AN EXAMPLE OF A FORTY PLUS YEAR OLD HOUSE WHERE THE CORRECTION OF FOUNDATION DISTRESS CAN COST AN AMOUNT EQUAL TO APPROXIMATELY 25% OF THE MARKET VALUE.

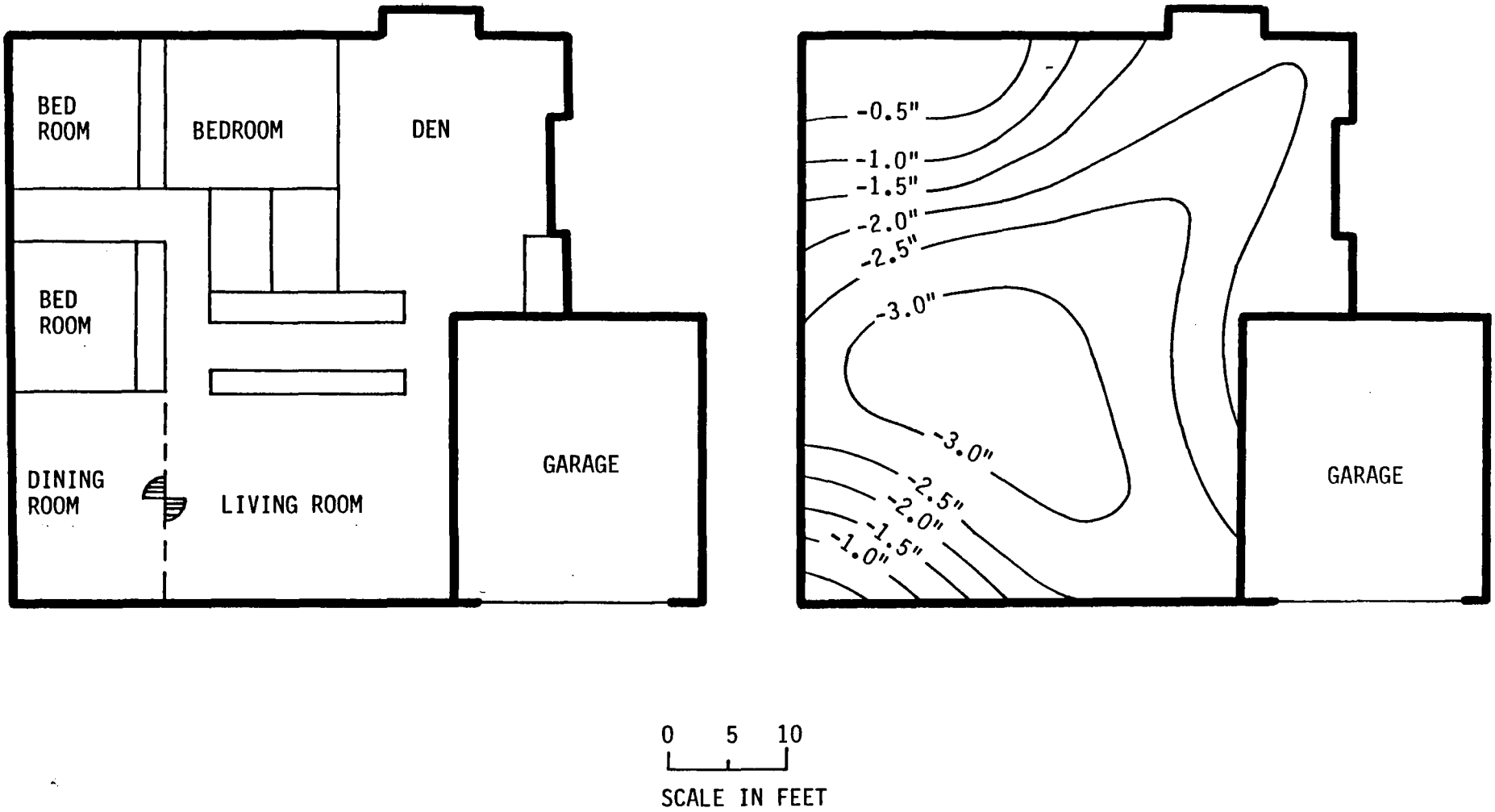


FIGURE I O. AN EXAMPLE OF A HOME WHICH HAS A SIGNIFICANT AMOUNT OF SLOPE BUT OTHERWISE IS IN GOOD CONDITION WITH NO SIGN OF CURRENT SHIFTING.

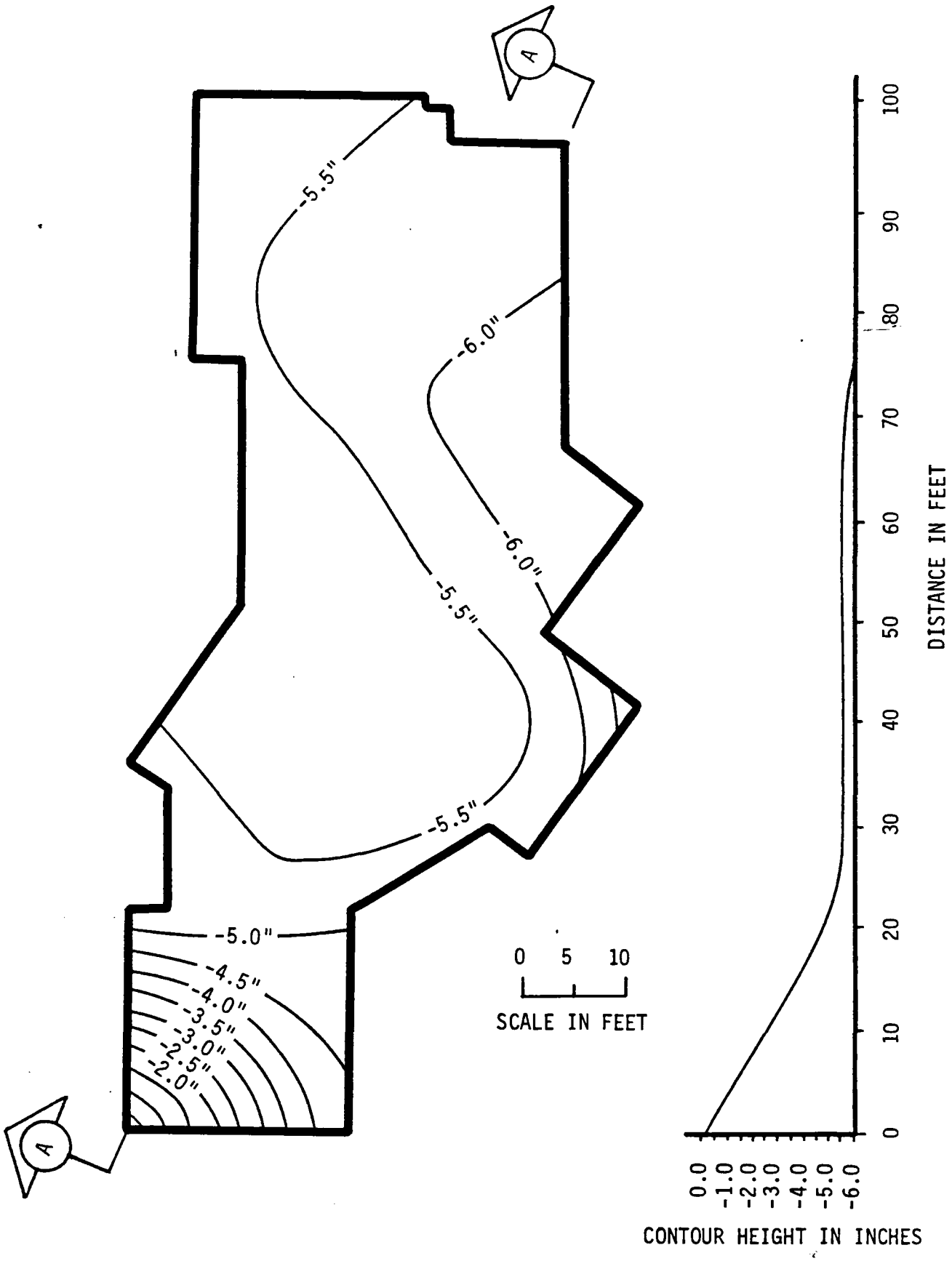


FIGURE 11. THE INTERIOR MEASURED FLOOR HEIGHTS OF A FOUNDATION SLAB WHOSE CORNER HAS BEEN AFFECTED BY A FAULT MOVEMENT.

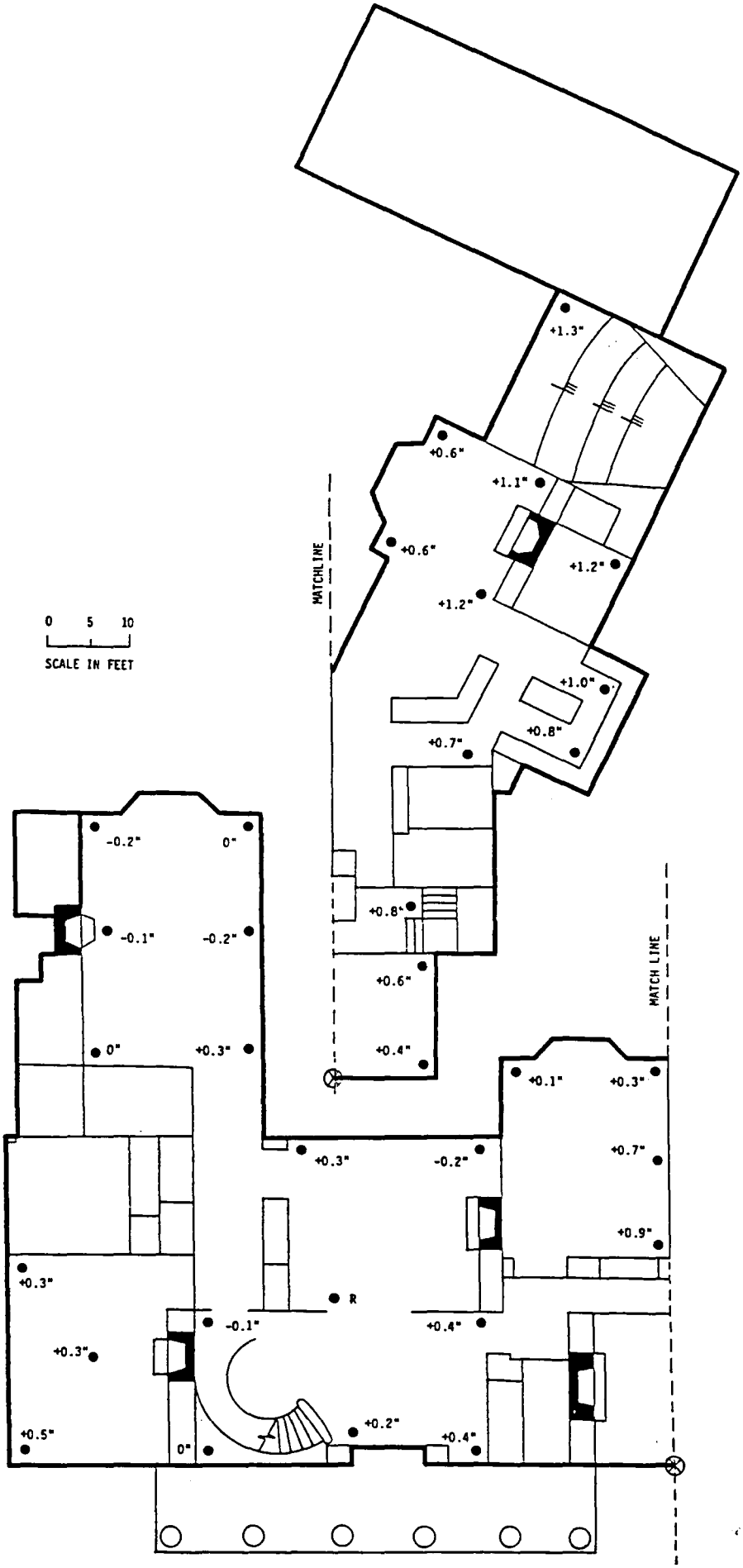


FIGURE 1 2. THE MEASURED FLOOR HEIGHTS OF A RELATIVELY NEW RESIDENTIAL BUILDING WHICH IS RESTING ON A SLAB SUPPORTED ON SPREAD FOOTINGS.

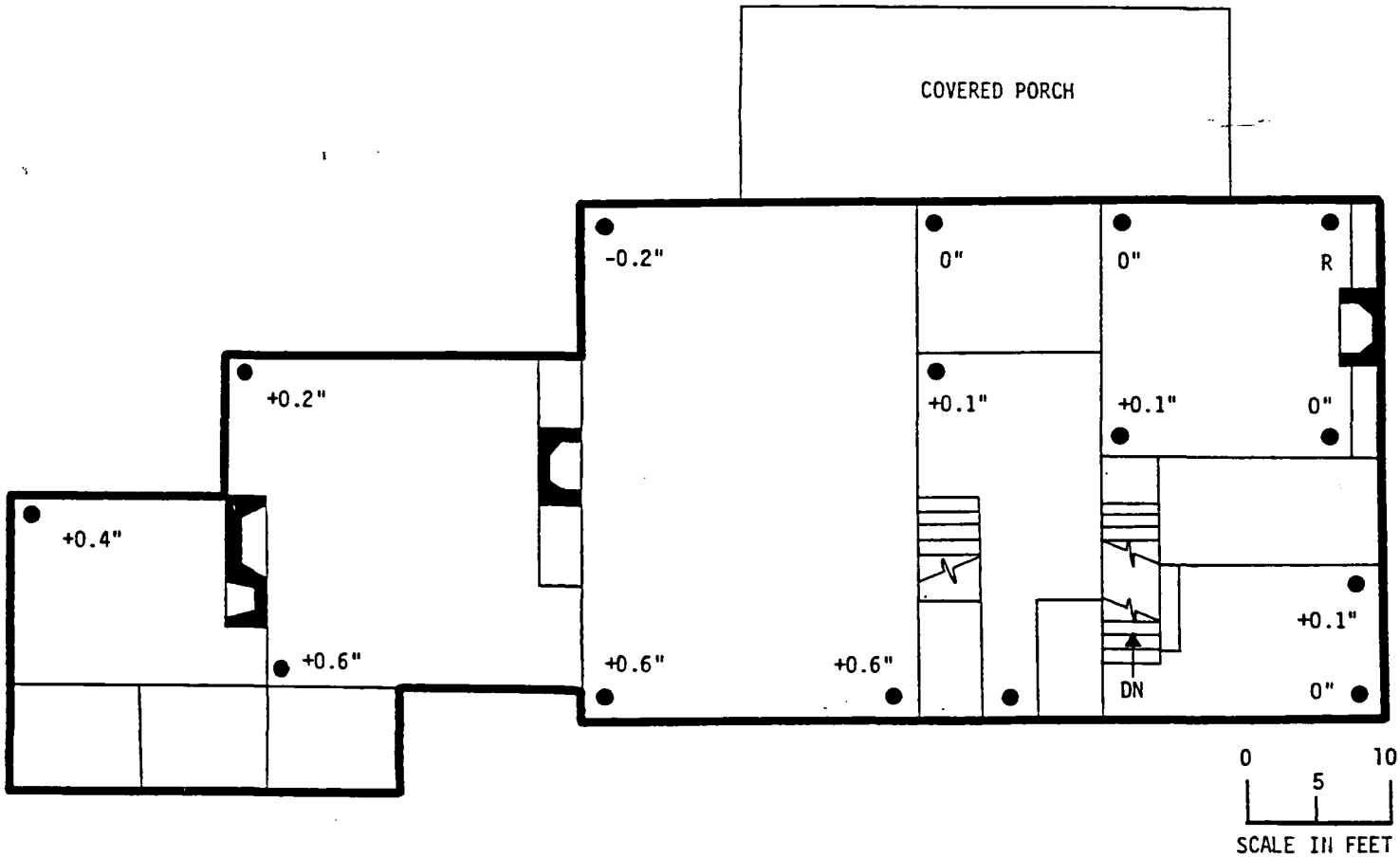


FIGURE 13. THE MEASURED FLOOR HEIGHTS OF A FIFTY YEAR OLD BUILDING WHICH IS RESTING ON A SLAB SUPPORTED ON SPREAD FOOTINGS.

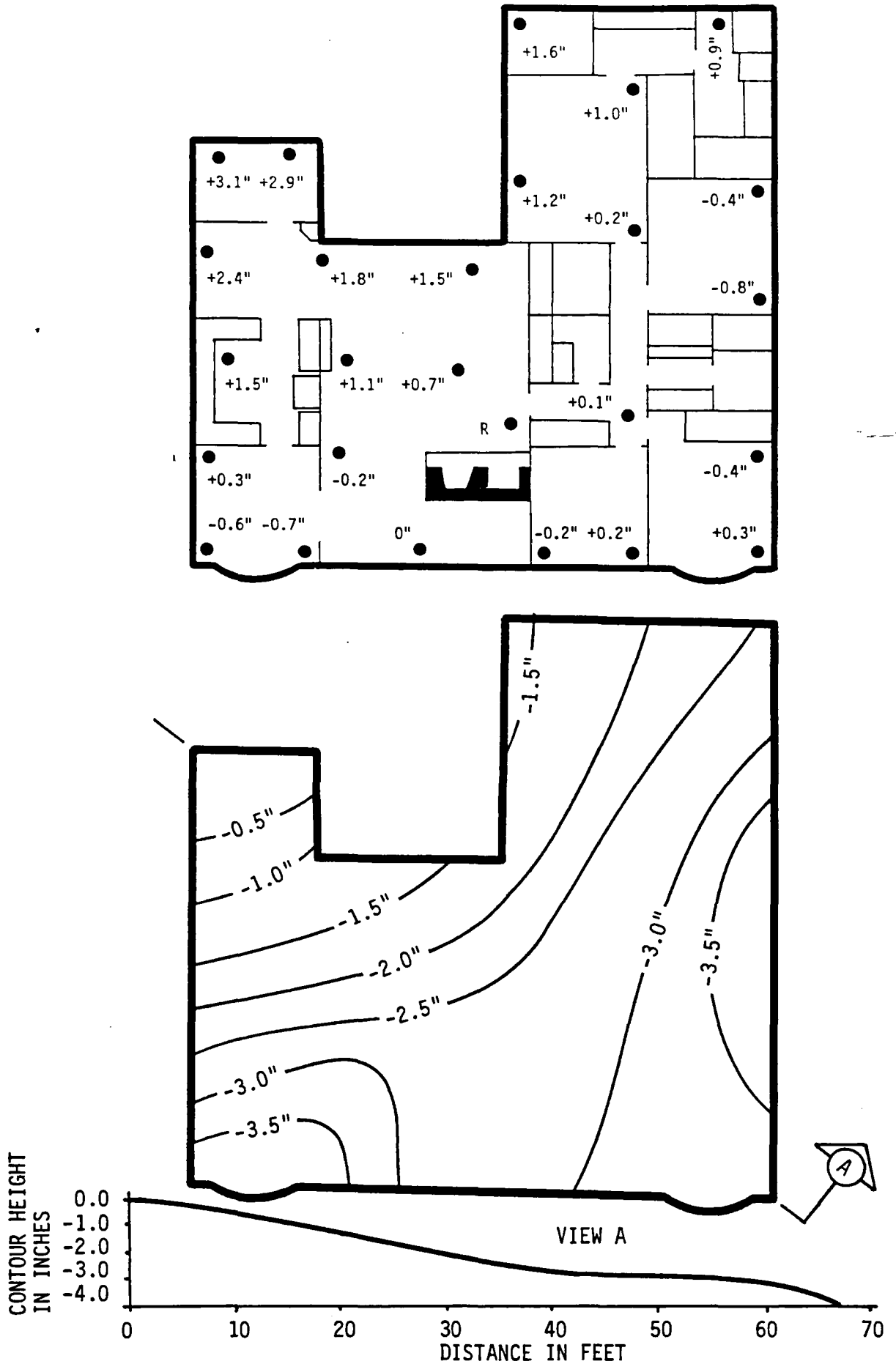
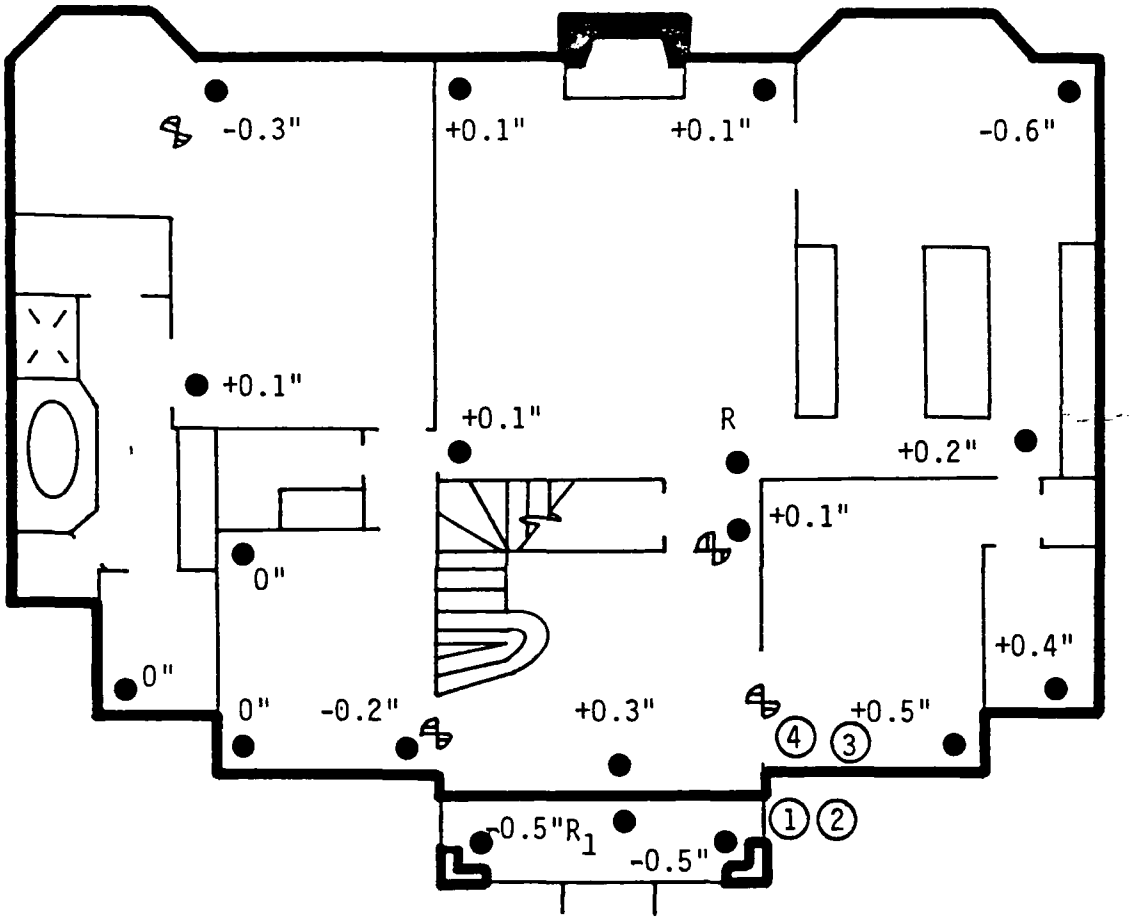


FIGURE 14. AN EXAMPLE OF A 20 YEAR OLD HOME WITH A SIGNIFICANT AMOUNT OF SLOPE BUT NO VISIBLE SIGN OF FOUNDATION DAMAGE. COUNTERS, SILLS, DOOR FRAMES, ETC. WERE LEVEL.



1. SEPARATION OF 1/2" @ BRICK CORNER
2. SEPARATION OF 1/2" @ WINDOW/BRICK JOINT
3. CRACKING OVER WINDOWS
4. SEPARATION @ WINDOW/SHEETROCK JOINT

FIGURE I 5. AN EXAMPLE OF A RELATIVELY NEW HOME WITH LEVEL FLOORS BUT A SEVERE AMOUNT OF FOUNDATION INDUCED DAMAGE.

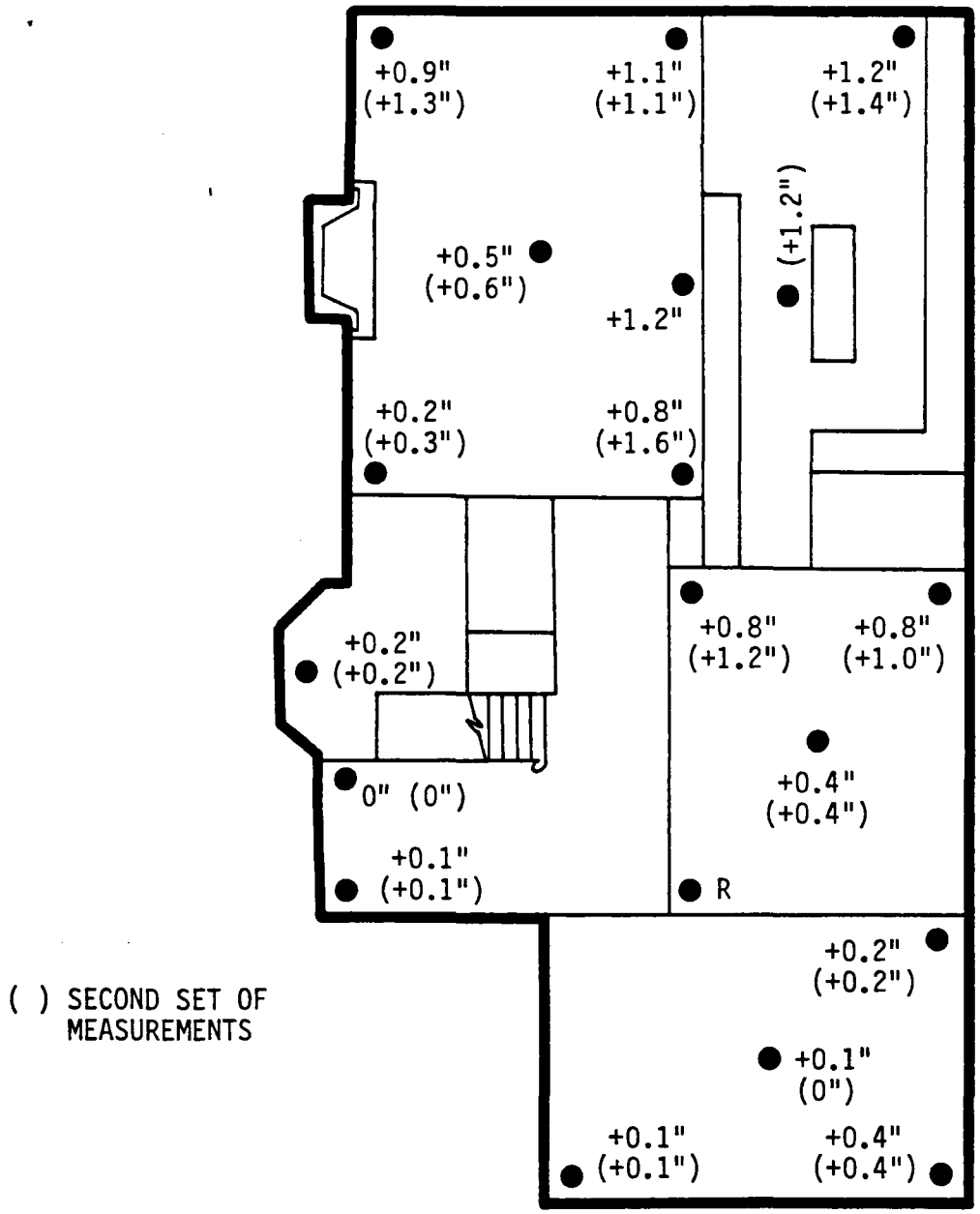


FIGURE 16. MEASUREMENTS TAKEN 6 MONTHS APART ON A NEW HOME WHICH HAS A PIER & BEAM FOUNDATION. THE DATA SHOWS THE FOUNDATION HAS MOVED UPWARD.

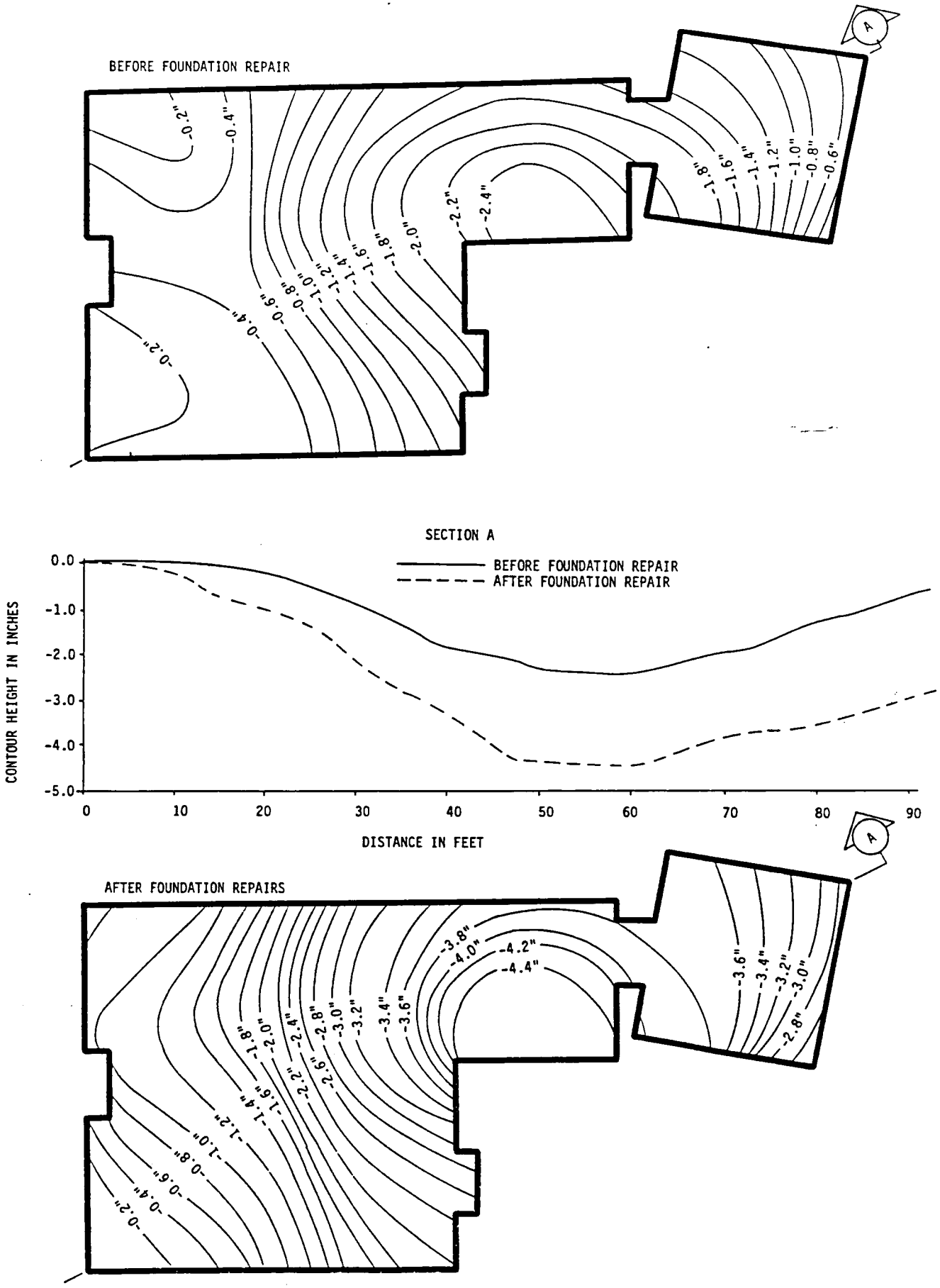


FIGURE 17. A NEW HOME WITH THE GARAGE & RESIDENCE ON SEPARATE FOUNDATIONS BUT JOINED ON THE SECOND STORY. SUBSIDENCE OCCURRED @ FRONT OF GARAGE & REAR OF RESIDENCE.

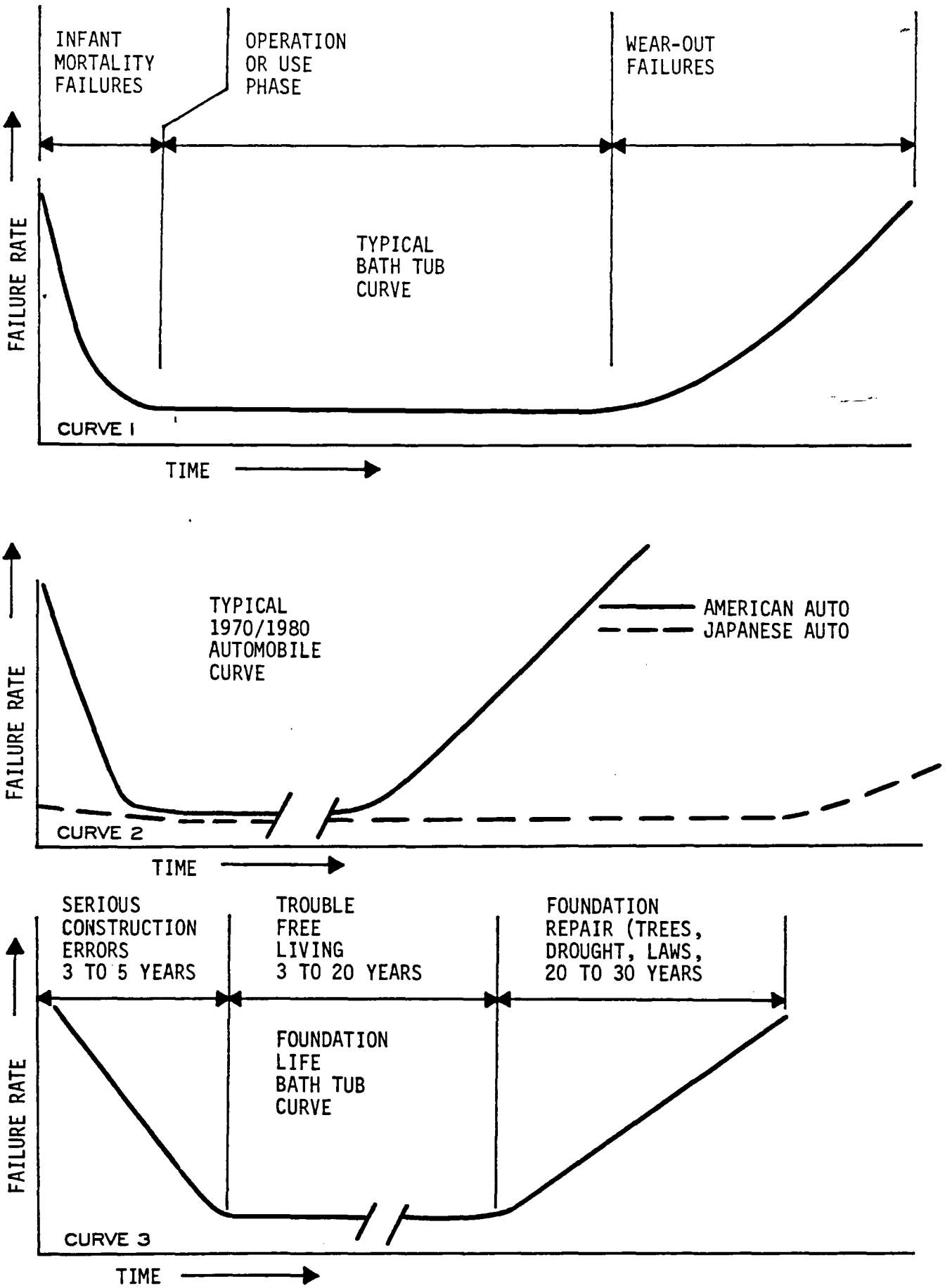


FIGURE 1 8. EXAMPLES OF QUALITY CONTROL BATH TUB CURVES.

CURRENT PRACTICES FOR DESIGN OF TRACT HOME FOUNDATIONS IN HOUSTON

by

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April 1993 to Present:
Chief Engineer for TSG Consultants, Inc., a small structural engineering firm that does design for home builders, architects, etc.

March 1989 - April 1993:
Chief engineer for Structural Engineering Consultants, Inc., a small structural engineering firm that performs design and inspection work for several large residential home builders, architects, etc.

April 1986 - March 1989:
Chief engineer for FRW and Associates, the design subsidiary for Gemcraft Homes, Inc. Responsibilities included direct supervision and design for all major structural components for production homes in all cities in which they build.

Mid-August 1982 - April 1986:
Chief engineer for MFI Associates, Inc., a small consultant firm, which designed post-tension foundations, shopping centers, warehouses, medical buildings, etc.

June 1981 - Mid-August 1982:
Chief design engineer for Amega Construction. I was directly responsible and supervised all the design, production, and quality control of precast products for a total of 6 projects, which used a patented column, beam, joist system to build warehouses, industrial buildings, and office spaces. The average project size was 60,000 sq. ft. The average shell cost was $1,000,000 per project. Additional responsibilities included the preparation of preliminary designs for marketing, the supervision of all drafting personnel, and the coordination of design projects with our overseas home office.

June 1979 - May 1981:
Employed by Everman Corporation and was directly responsible for the design of all the precast components for jobs ranging from 500,000 sq. ft. parking garages to the new Dallas County jail. Design included all component and connection designs, plus wind and gravity analysis of the structures.

January 1978 - May 1979:
Employed by Brockette/Davis/Drake, Inc.

EDUCATION

From 1971 to 1977, attended the University of Texas at Arlington, and received a Bachelor of Science in Civil Engineering (12/20/76) and a Master of Science in Civil Engineering (12/19/77).

PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers.
Currently registed to practice in 13 states.

CURRENT PRACTICES FOR DESIGN OF TRACT HOME FOUNDATIONS IN HOUSTON

INTRODUCTION

When the phrase "tract housing" is mentioned, people generally think of "affordable housing", "starter homes", FHA, or neighborhoods where your house looks like every third home on the street. 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 neither "starter homes" nor FHA, and most builders 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 95% 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. 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. However, the PTI method is currently under evaluation to see what changes and improvements can be made. As with all standards, updates and revisions are required to accommodate various changes in design philosophy, products, and current building practices.

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 more expensive than the average home of 15 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. Litigation costs.

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 soils 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.

The only solution we see which will help alleviate these problems, is 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 involved. Without this type of coordinated cooperation, conditions can easily arise which will be detrimental to the performance of a home's foundation.

CURRENT DESIGN PROCEDURE
RECOMMENDED BY
THE POST TENSION INSTITUTE

Reference: Design and Construction of Post-Tensioned Slabs-On-Ground - First Edition, PostTensioning Institute Copyright 1980. Reprinted with the permission of Post-Tensioning Institute.

4.0 DESIGN PARAMETERS

4.1 General

There are eight design parameters that must be known to successfully design a slab-on-ground. These eight parameters include three soil and five structural quantities. If the slab is to be constructed over expansive clays, a ninth parameter – climate – must also be considered. The design parameters discussed in this chapter are applicable to both prestressed and conventionally reinforced 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.

4.2 Design Parameters

(A) Climate. Clay soils with the potential to shrink or swell are found in almost all parts of the United States (Fig. 4.1), but this potential is only realized in climates that have periods of rainfall followed by extended periods without rainfall. These necessary 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 U.S.

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 is in an area that has low rainfall and the climate 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 his foundation will reside.

One such environmental indicator is the index of potential evapotranspiration which was introduced by Thornthwaite. 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 State is shown in Figure A.3.2 in Appendix A.3, and a larger scale map of the State of Texas is shown in Figure A.3.3. The maps in Figs. A.3.2 and A.3.3 represent twenty year average values of the Thornthwaite Index for the period 1955-1974. A positive Thornthwaite Moisture Index (I_m) measurement 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 "center heave" and "Doming") or an edge lift mode (also called "edge heave" and "dishing"). The center lift condition is a long term condition and occurs, either due to the

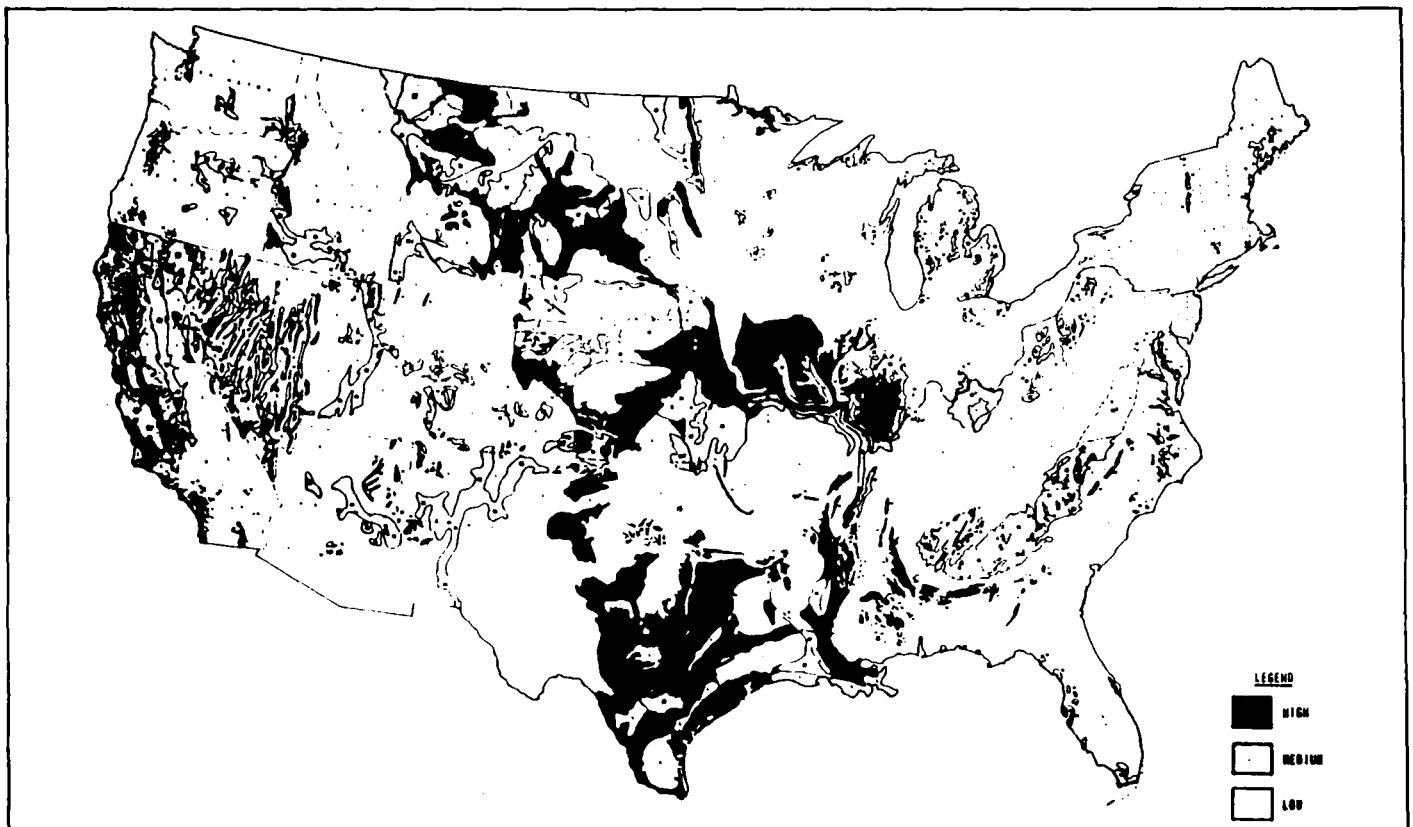


Fig. 4.1 Distribution of expansive soils in the United States and their relative activity after Wiggins (104)*.

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

soil beneath the interior of the slab becoming wetter and expanding, or because the soil around the perimeter of the slab dries and shrinks, or a combination of both. Conversely, the edge lift condition 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 modes are depicted in Figure 4.2.

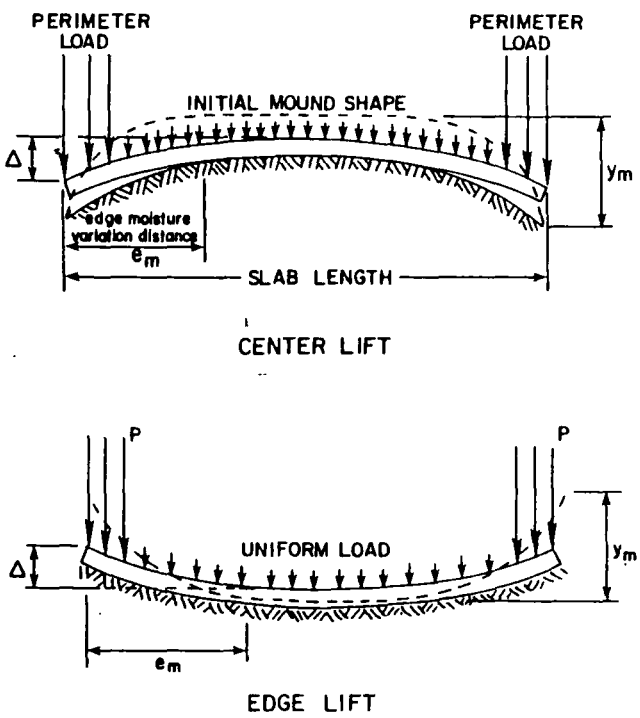


Fig. 4.2 Soil-structure interaction models (108)

(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 strong evapo-transpiration 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 environment. The value of 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.

(3) Differential Soil Movement, y_m. The amount of differential soil movement, y_m, to be expected depends upon a number of conditions, including the type and amount of clay mineral, its initial wetness, the depth of the active zone (the depth of soil suction variation), the velocity of moisture infiltration or evaporation as well as other less easily measured effects such as type and amount of site post-construction and pre-construction vegetation cover, slope of the site, drainage conditions, 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 differential soil movement is presented in Appendix A.3.

(4) Factors Not Related to Climate. The use of an environmental indicator as an aid in estimating the amount of shrink-swell that a soil will exhibit does not account for factors that cause soil movement that are not related to climate. Factors not related to climate may induce soil movements larger than those resulting from climatic influences alone. For this reason, some degree of engineering judgement or special measurements must also be utilized to consider the potential influence of these factors on the soil design parameters. 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 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:

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

B. 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 evaporization. Ponding or trapping of water in localized areas adjacent to the foundations can cause differential moisture levels in subsurface soils.

C. 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.

D. 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 Length. This parameter is generally fixed by the owner or by functional requirements.

(2) Beam Spacing. Spacing of stiffening beams is usually dictated by the slab geometry, which is generally fixed by the owner or by functional requirements. Although actual spacings are a function of structural design considerations, spacings between 10 and 20 feet are found most often in practice. Additional beams may be required where heavy loads are applied to the slab, as in the case of a fireplace.

(3) Loading. The loading applied to the slab is dictated by local building codes as well as the architecture of the building and the arrangement of the building superstructure. In most cases, the magnitude of the applied loading is a parameter which the engineer cannot adjust significantly in his design.

Concentrated loads, such as chimneys, should be located so that these loads are transferred to the stiffening beams. Concentrated loads in excess of 100 psf per bay area (including slab load) should be located to rest on at least two stiffening beams.

In the case of smaller concentrated loads between stiffening beams, the tensile stresses in the concrete slab should be checked to see that they are within the allowable tensile stresses permitted by the ACI 318-77 Building Code. A formula for calculation of tensile stresses beneath concentrated loads is presented in Chapter 6.0, Section 6.13.

(4) Depth of Stiffening Beams. The depth of stiffening beams is usually the controlling parameter in structural design of stiffened slabs. Beam depth is the quantity that the design engineer usually increases in order to increase moment and shear capacity, and to reduce deflections. Frost depth may be a limiting factor for determining minimum edge beam depth in certain locales.

(5) Width of Stiffening Beams. The minimum width of stiffening beams may be governed more often by either the soil bearing capacity or excavation equipment limitations than by other design considerations such as limiting shear stress. It is difficult to excavate a trench for stiffening beams less than 8 inches wide. Stiffening beam widths most commonly found in practice are 8-12 inches wide.

6.0 STRUCTURAL DESIGN PROCEDURE

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. Design formulas are also presented for slabs constructed on compressible soils. An equation is developed for calculation of the stress due to concentrated line loads on slabs.

The design procedure for post-tensioned slabs constructed over expansive clays should include the following steps:

1. Assemble all the known design data.
2. Divide an irregular slab plan into overlapping rectangles and design each rectangular section separately (Fig. 6.1).
3. Assume a trial section in both the long and short directions of the design rectangle.
4. Calculate the service moment the section will be expected to experience in each direction for either the center lift or edge lift conditions.
5. Determine the allowable moment capacity of the assumed section in each direction and compare to the expected service moment.
6. Determine if the trial sections will meet differential deflection criteria in each direction.
7. Calculate the expected shear force in the assumed sections.
8. Determine the maximum allowable shear capacity of the sections and compare to the expected shearing force.
9. Repeat steps 4 through 8 for the opposite swelling condition.
10. 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 (Step 9).
11. 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.
12. Calculate stresses due to any heavy concentrated loads on the slab and provide special load transfer details when necessary.

The design procedure for slabs 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 Fig. 4.2 for the edge lift loading case.

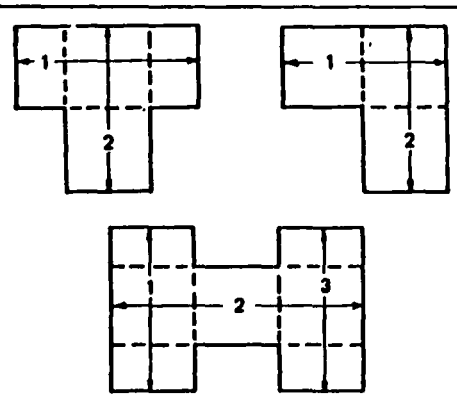


Fig. 6.1 Design rectangles for slabs of irregular shape

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, μ.

B. Structural data and materials properties.
   (1) Slab length, L, in feet.
   (2) Perimeter loading, P, in lbs. per foot.
   (3) Stiffening beam spacing, S, in feet.
   (4) Beam depth, d, in inches.
   (5) Compressive strength of the concrete, f'_c.
   (6) Allowable tensile stress in the concrete, f_t.
   (7) Allowable compressive stress in the concrete, f_c.
   (8) Type, grade, and strength of the prestressing steel.
   (9) Grade and strength of conventional reinforcement, if needed.
   (10) Prestress losses.

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 beam can be obtained from solving either Equation (27) or Equation (28) for the beam depth yielding the maximum allowable differential deflection. A preliminary estimate of the allowable differential deflection can be made 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 permissible deflection ratio, e.g.,

   (a) Center Lift

$$\frac{\Delta}{L \text{ or } 6\beta} = \frac{1}{360} \qquad (1)$$

   (b) Edge Lift

$$\frac{\Delta}{L \text{ or } 6\beta} = \frac{1}{1700} \qquad (2)$$

   The 1/1700 deflection ratio is only used to initially estimate the required beam depth for the edge lift condition.

   (3) Assume a beam spacing and solve for beam depth, d:

   (a) Center Lift

$$\text{Set } X = \frac{(y_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{380\,\Delta} \qquad (3a)$$

Then

$$\log_{10}(d) = \frac{1}{1.214}\log_{10}(X) \qquad (3b)$$

or,

$$d = X^{0.824} \qquad (3c)$$

(b) Edge Lift

$$\text{Set } X = \frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(y_m)^{0.76}}{12\,\Delta\,(P)^{0.01}} \qquad (4a)$$

Then

$$\log_{10}(d) = \frac{1}{0.85}\log_{10}(X) \qquad (4b)$$

or,

$$d = X^{1.176} \qquad (4c)$$

In most cases, the depth of the beams should be the same for all beams in both directions.

B. Determine Section properties. The moment of inertia, section modulus, cross sectional area of the slab and eccentricity of the prestressing force may be calculated for the trial beam depth determined above in accordance with normal structural engineering procedures, or by use of design aids presented in Appendix A.5. These procedures are illustrated in the design examples presented in Appendices A.6, A.7 and A.8.

6.5 Design Stresses

The following design stresses are recommended:

A. Allowable Tensile Stress

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

B. Allowable Compressive Stress

$$f_c = 0.45\,f'_c \qquad (6)$$

C. Estimated Tensile Cracking Stress

$$f_{cr} = 7.5\sqrt{f'_c} \qquad (7)$$

D. Bearing Stress at Anchorages

(1) At Service Load

$$f_{bp} = 0.6\,f'_c\sqrt{(A'_b/A_b)} \le f'_c \qquad (8)$$

(2) At Transfer

$$f_{bp} = 0.6\,f'_{ci}\sqrt{(A'_b/A_b)} - 0.2 \le f'_{ci} \qquad (9)$$

where,

A_b = Loaded Area

A'_b = Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area

E. Shear Stress

(1) Permissible Shear Stress

$$v_c = 1.5\sqrt{f'_c} \qquad (10)$$

(2) Design Shear Stress

$$v = \frac{VW}{ndb} \qquad (11)$$

F. Prestressing Steel. The maximum stress in the prestressing steel during stressing shall not be greater than 0.80 times the guaranteed ultimate strength of the prestressing steel, or 0.94 times the specified yield strength of the prestressing steel, whichever is smaller. The maximum stress in the prestressing steel immediately after anchoring shall not exceed 0.70 times the guaranteed ultimate strength of the steel.

6.6 Prestress Losses

Loss of prestress due to elastic shortening of the concrete, creep of concrete, shrinkage of concrete, and steel relaxation shall be taken as 30,000 psi for wire and strand tendons and 20,000 psi for bar tendons, unless more exact determination of these individual losses can be made. The losses specified in this section comply with those presented in the Post-Tensioning Manual.[70] Prestress losses due to intentional and unintentional curvature of the tendons shall be calculated in accordance with Chapter 18 of the ACI 318-77 Building Code.

6.7 Slab-Subgrade Friction

The effective prestressing force in post-tensioned slabs-on-ground is 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 or temperature variations. 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 should be taken as 0.75 for slabs on polyethylene and 1.00 for slabs cast directly on a sand base.

To provide assurance against cracking resulting from subgrade frictional resistance to movements induced by prestressing, concrete shrinkage, or temperature variations, the prestressing force provided in each direction shall not be less than provided by Equation (12). For very short slabs where the β-length is approximately equal to one-half of the length of the design rectangle, a prestressing force equivalent to one-half the weight of the stiffened slab multiplied by the coefficient of friction shall be deducted from the total prestressing force in calculating the net prestressing force available to provide resistance to applied bending moments.

$$P_r \ge \frac{W_{slab}}{2000} + .05A \qquad (12)$$

The maximum spacing of tendons should not exceed that which would produce a minimum average effective prestress of 50 psi after allowance for slab-subgrade friction. The maximum spacing of tendons placed in the slab portion of the cross-section can be estimated from Figure 6.2 (coefficient of friction assumed to be 0.75). Figure 6.2 was developed presuming tendons stressed from both ends. This is usually not necessary or practical for slabs for single family residences. Tendon spacings obtained from Figure 6.2 may have to be reduced to provide sufficient post-tensioning force to satisfy moment requirements.

6.8 Maximum Design Moments

The maximum moment will vary, depending upon the swelling mode and the slab direction being designed. Moments for the center lift condition will, in general, be greater than edge lift moments. Moments in the short direction will, in general, be slightly greater than moments in the long direction.

A. Center Lift Moment

(1) Long Direction. The following equations may be used to calculate maximum design moments for center lift bending in the long direction:

$$M_\ell = A_o\left[B(e_m)^{1.238} + C\right] \qquad (13)$$

where,

M_ℓ = Design moment in long direction, ft-kips/ft.

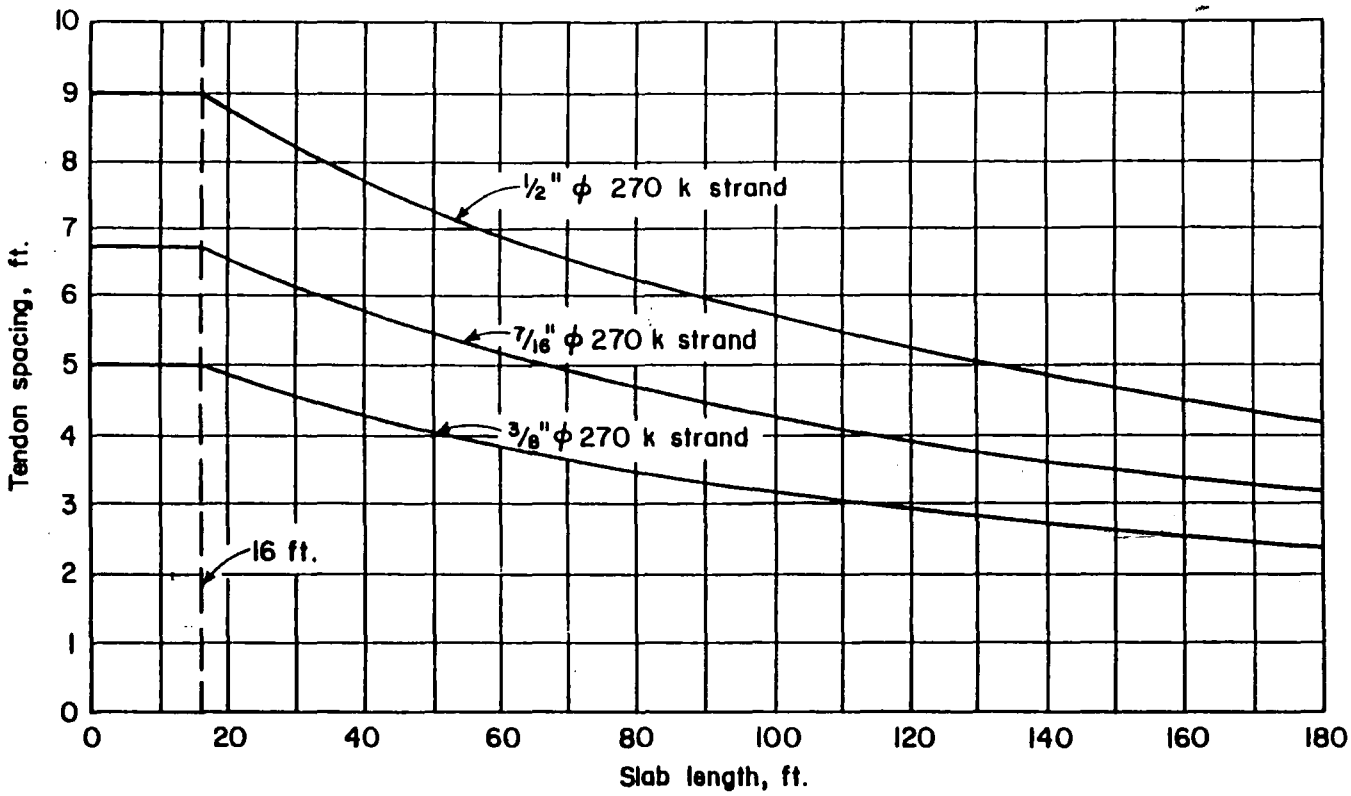


Fig. 6.2 Maximum slab tendon spacing to overcome slab-subgrade friction (friction coefficient = 0.75) and retain 50 psi residual prestress compression at midpoint of stiffened slab on ground.

$$A_o = \frac{1}{727}\left[(L)^{0.013}(S)^{0.306}(d)^{0.688}(P)^{0.534}(y_m)^{0.193}\right] \quad (14)$$

and for

$0 \le e_m \le 5 \quad B = 1, \quad C = 0$

$5 < e_m \quad B = \left(\frac{y_m - 1}{3}\right) \le 1.0,$

$$C = \left[8 - \left(\frac{P - 613}{255}\right)\right]\left[\frac{4 - y_m}{3}\right] \ge 0 \quad (15)$$

(2) Short Direction. The maximum design moment in the short direction for center lift bending may be calculated as follows:

$$M_s = \left[\frac{58 + e_m}{60}\right] M_\ell \quad (16)$$

where

M_s = Design Moment in short direction, in ft-kips/ft.

B. Edge Lift Moment

(1) Long Direction. The maximum design moment in the long direction for edge lift bending may be calculated as follows:

$$M_\ell = \left[\frac{(S)^{0.10}(de_m)^{0.78}(y_m)^{0.66}}{7.2\,(L)^{0.0065}(P)^{0.04}}\right] \quad (17)$$

(2) Short Direction. The maximum design moment in the short direction for edge lift bending may be calculated as follows:

$$M_s = (d)^{0.35}\left[\frac{19 + e_m}{57.75}\right] M_\ell \quad (18)$$

6.9 Maximum Allowable Service and Cracking Moments

A. Allowable Service Moments. The maximum moments to which the assumed sections can be subjected, consistent with the design stresses, can be determined from the familiar bending stress formula, rearranged so as to be able to solve for the maximum allowable external moments. The sign convention adopted is to represent compressive forces and eccentricities above the neutral axis as positive. The form of Equations (19) through (24) have been adjusted for the sign difference between tensile and compressive stresses. The following equations for allowable service moments must be evaluated for both the long and short direction.

(1) Negative Bending Moment, $_nM$

(a) Tension in Top Fiber

$$_nM_t = S_T\left(\frac{P_r}{A} + f_t\right) + P_r e \quad (19)$$

where

S_T = Section modulus for top fiber, inches^3
P_r = Prestressing force, kips
A = Cross-sectional area, inches^2
f_t = Allowable tensile stress, kips/inches^2
e = Eccentricity of prestressing force, inches

(b) Compression in Bottom Fiber

$$_nM_c = S_B\left(f_c - \frac{P_r}{A}\right) + P_r e \quad (20)$$

where

S_B = Selection modulus for bottom fiber, Inches^3

f_c' = Allowable compressive stress, ksi

The maximum external negative moment that can be carried by the section is the smaller of the moments calculated by equations (19) and (20).

(2) Positive Bending Moment, $_pM$

(a) Tension in Bottom Fiber

$$_pM_t = S_B \left(\frac{P_r}{A} + f_t\right) - P_r e \qquad (21)$$

(b) Compression in Top Fiber

$$_pM_c = S_T \left(f_c - \frac{P_r}{A}\right) - P_r e \qquad (22)$$

The maximum external positive service moment that can be carried by the section is the smaller of the moments calculated by equations (21) and (22).

B. Tensile Cracking Moments. Stiffened slabs-on-ground are usually designed to be under reinforced. As long as the actual moment acting in the slab is below the tensile cracking moment, the stiffening beams may be assumed to act in their elastic range, and the assumed use of the gross section in computing deflection criteria is justified.

(1) Negative Bending Moment, $_nM_{cr}$

$$_nM_{cr} = S_T \left(\frac{P_r}{A} + f_{cr}\right) + P_r e \qquad (23)$$

(2) Positive Bending Moment, $_pM_{cr}$

$$_pM_{cr} = S_B \left(\frac{P_r}{A} + f_{cr}\right) - P_r e \qquad (24)$$

C. Compare Allowable and Cracking Moments to be Expected Service Moment. The design moments expected to occur in both directions, as calculated from Equations (13) and (16) through (18) must be compared to the allowable moments determined in Equations (19) through (22). If either the short direction or long direction design moments exceed the allowable service moments, the moment capacity of the section must be increased. Means of increasing the moment capacity include:

(1) Deepening the stiffening beams (for deficient negative and positive moment capacity);

(2) Decreasing the beam spacing (for deficient negative and positive moment capacity);

(3) Increasing the prestressing force (for deficient negative moment capacity);

(4) Decreasing the prestress eccentricity by carrying tendons below the neutral axis (for deficient positive moment capacity).

If the moment capacity of the assumed section exceeds the design moment, economies may be realized by performing the opposite to the actions suggested above for increasing moment capacities.

6.10 Differential Deflection

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:

$$\beta = \frac{1}{12} \sqrt[4]{\frac{E_c I}{E_s}} \qquad (25)$$

where

β = Relative stiffness length, in feet

E_c = Creep Modulus of Elasticity of Concrete, psi

E_s = Modulus of Elasticity of Soil, psi

I = Gross Moment of Inertia of Section, Inches^4

If the creep modulus of elasticity of the concrete is not known, it can be closely approximated by using 0.5 of the normal or early life concrete modulus of elasticity. If the modulus of elasticity of the clay soil is not known, use 1000 psi.

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.

(1) Center Lift.

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

where

Δ_{allow} = Allowable differential deflection, in inches

L = Total slab length, in feet

β = Relative stiffness length, in feet

(2) Edge Lift.

$$\Delta_{allow} = \frac{12\ (L \text{ or } 6\beta)}{800} \qquad (26)$$

The more stringent allowable differential deflection for the edge lift is specified because edge lift deflections are normally much less than center lift deflections and stems of beams resisting positive moments may be unreinforced.

D. Expected Differential Deflection Without Prestressing.

(1) Center Lift.

$$\Delta_o = \left[\frac{(\gamma_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{380\ (d)^{1.214}}\right] \qquad (27)$$

where

Δ_o = Expected differential deflection, in inches

(2) Edge Lift

$$\Delta_o = \left[\frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(\gamma_m)^{0.76}}{15.90\ (d)^{0.85}(P)^{0.01}}\right] \qquad (28)$$

E. Deflection Reduction Due to Prestressing. Normally, most of the prestressing is placed in the slab, and the centroid of the prestressing force is above the center of gravity of the section. Because of this, any deflection due to negative bending must first overcome a slight amount of positive deflection or camber caused by the prestressing. This differential deflection advantage of prestressing can be calculated by:

(1) Calculate the percent of differential deflection reduction

$$\Delta_c = e\sqrt{\frac{6400}{9L}} \qquad (29)$$

where

Δ_c = Differential deflection correction, in percent.

e = Eccentricity of prestressing force, in inches.

(2) Calculate corrected differential deflection.

$$\Delta = \Delta_o \left[\frac{100-\Delta_c}{100}\right] \qquad (30)$$

where

Δ = Expected deflection, in inches

The effect of prestressing usually adds to the deflection due to edge lift bending. However, deflections due to this bending mode are usually smaller than center lift deflections.

F. Compare expected to Allowable Differential Deflection. If the expected differential deflection as calculated by either Equations (27) or (28) adjusted for the effect of prestressing exceeds that determined from Equations (1) or (26), respectively, the assumed section must be stiffened. This can be accomplished in at least three ways:

(1) Deepening the stiffening beams,

(2) Decreasing the beam spacing, or

(3) Adding additional prestressing tendons above the neutral axis.

6.11 Shear

A. Expected Service Shear. Expected values of service shear forces in kips per ft. of width or length of slab may be calculated from the following formulas:

(1) Center Lift.

(a) Short Direction Shear.

$$V_s = \frac{1}{1350}\left[(L)^{0.19}(S)^{0.45}(d)^{0.20}(P)^{0.54}(y_m)^{0.04}(e_m)^{0.97}\right] \quad (31)$$

(b) Long Direction Shear.

$$V_\ell = \frac{1}{1940}\left[(L)^{0.09}(S)^{0.71}(d)^{0.43}(P)^{0.44}(y_m)^{0.16}(e_m)^{0.93}\right] \quad (32)$$

(2) Edge Lift

For both directions:

$$V = \left[\frac{(L)^{0.07}(d)^{0.40}(P)^{0.03}(e_m)^{0.16}(y_m)^{0.67}}{3.0\,(S)^{0.015}}\right] \quad (33)$$

where

V, V_s, V_ℓ = Shear force, in kips/ft.

B. Allowable Shear Stresses

(1) Nominal Total Design Shear Stress, v. Only the beams may be considered in calculating the cross-sectional area resisting shear force.

$$v = \frac{VW}{ndb} \quad (11)$$

where

V = Total shear force acting on the section, kips.

(2) Nominal permissible Shear Stress, v_c. Unless the permissible shear stress can be determined by testing or by more rigorous analysis, the maximum shear stress permitted shall be given by

$$v_c = 1.5\sqrt{f'_c} \quad (10)$$

where v_c and f'_c are both expressed in psi

C. Compare v to v_c. If v exceeds v_c, shear reinforcement in accordance with ACI 318-77 and the Post-Tensioning Manual must be provided. Possible alternatives to reinforcement include:

(1) Increasing the beam depth,

(2) Increasing the beam width, or

(3) Increasing the number of beams (decrease beam spacing).

D. Shear Reduction Due to Prestressing. An advantage of curved or draped prestressing tendon in beam stems is that due to the upward force exerted by the tendon on the concrete, shear compensation in an amount equal to $P_r \sin\alpha$ is obtained. The design shear force carried by the beams is reduced accordingly. Figure 6.3 shows the effect of draped prestressing tendons on shear reduction.

6.12 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 Conventionally Reinforced Slabs-on-Ground. The design equations presented (Equations 13, 16-18, 27, 28, and 31-33) produce the values of bending moment, shear, and differential deflection that can be expected to occur using a given set of soil and structural parameters. These design values may be calculated for slabs reinforced with mild steel as well as for post-tensioned slabs. Once these design parameters are known, design of either type of slab can proceed. However, reinforcement calculations and limiting values of deflections for non-prestressed slabs must be developed by interested parties using conventional reinforced concrete technology. To conform to the same deflection

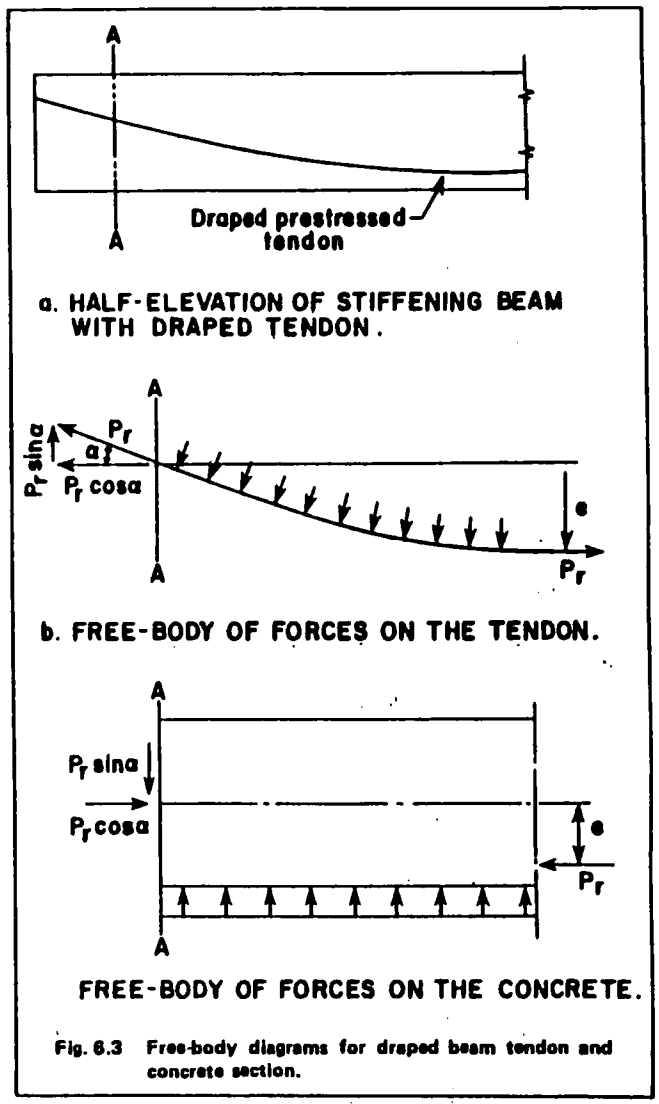


Fig. 6.3 Free-body diagrams for draped beam tendon and concrete section.

criteria, conventionally reinforced slabs designed on the basis of cracked sections will have to have significantly deeper beam stems than prestressed slabs. The deeper sections will, in turn, result in larger design moments.

B. Design of Slabs Subject to Frost Heave. Design 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 e_m for frost heave would have to be estimated from values comparable to those for expansive soils.

C. Slabs-on-Ground Constructed over Compressible Soils. Design of slabs constructed over compressible soils can proceed 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, q_{allow}, equal to or less than 1500 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. These equations are:

(1) Moment

(a) Long Direction

$$M_{cs_\ell} = \left(\frac{\delta}{\Delta_{ns_\ell}}\right)^{0.50} M_{ns_\ell} \qquad (34)$$

where

$$M_{ns_\ell} = \left[\frac{(d)^{1.35}(S)^{0.36}}{80(L)^{0.12}(P)^{0.10}}\right] \qquad (35)$$

$$\Delta_{ns_\ell} = \left[\frac{(L)^{1.28}(S)^{0.80}}{133(d)^{0.28}(P)^{0.62}}\right] \qquad (36)$$

and,

δ = Expected settlement, in inches, due to the total load expressed as a uniform load; reported by the Geotechnical Engineer, and all other symbols are as previously defined.

(b) Short Direction

$$M_{cs_s} = \left(\frac{970 \cdot d}{880}\right) M_{cs_\ell} \qquad (37)$$

(2) Differential Deflection

$$\Delta_{cs} = \delta \exp(Z) \qquad (38)$$

where

Δ_{cs} = Expected Differential Deflection, in inches

$Z = 1.78 - 0.103\,(d) - 1.65 \times 10^{-3}(P) + 3.95 \times 10^{-7}\,(P^2)$ (39)

exp(Z) = Natural base e raised to the exponent Z, that is, e^Z

(3) Shear

(a) Long Direction

$$V_{cs_\ell} = \left[\frac{\delta}{\Delta_{ns_\ell}}\right]^{0.30} V_{ns_\ell} \qquad (40)$$

where

$$V_{ns_\ell} = \left[\frac{(d)^{0.90}(PS)^{0.30}}{550(L)^{0.10}}\right] \qquad (41)$$

(b) Short Direction

$$V_{cs_s} = \left[\frac{116-d}{94}\right] V_{cs_\ell} \qquad (42)$$

6.13 Calculation of Stress in Slabs Due to Load Bearing Partitions

The equation for the allowable 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 given by

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

where

M_{max} = the maximum moment in (in-lb) per linear foot of bearing partition in a direction at right angles with the bearing partition

$$\beta = \left[\frac{4\,E_c I}{kb}\right]^{1/4}, \text{ relative stiffness length in inches} \qquad (44)$$

where

E_c = Creep modulus at elasticity of concrete

I = Moment of inertia of loaded slab width

B = Assumed beam (slab) width

k = Soil modulus

P = Bearing partition load in lb/ft of length, + upward

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

with the concrete and soil properties generally assumed ($E_c = 1.5 \times 10^6$ psi, k = 4 pci),

$$\frac{4\,E_c}{k} = 1.5 \times 10^6 \text{ in.}$$

and β becomes:

$$\beta = 18.8\,t^{3/4}$$

therefore

$$M_{max} = -\frac{18.8\,Pt^{3/4}}{4} = -4.7\,Pt^{3/4} \qquad (45)$$

The equation for allowable tensile stress, f_t, is

$$f_t = \frac{M_{max}C}{I} - f_p \qquad (46)$$

where f_p = minimum compressive stress in the concrete due to prestressing (usually 50 psi).

Since

$$\frac{I}{C} = \frac{b\,t^3}{12}\left(\frac{2}{t}\right) = \frac{b\,t^2}{6}$$

and

b = 12 in. (one linear foot of bearing partition)

then

$$\frac{I}{C} = 2t^2$$

Thus, the allowable tensile stress is

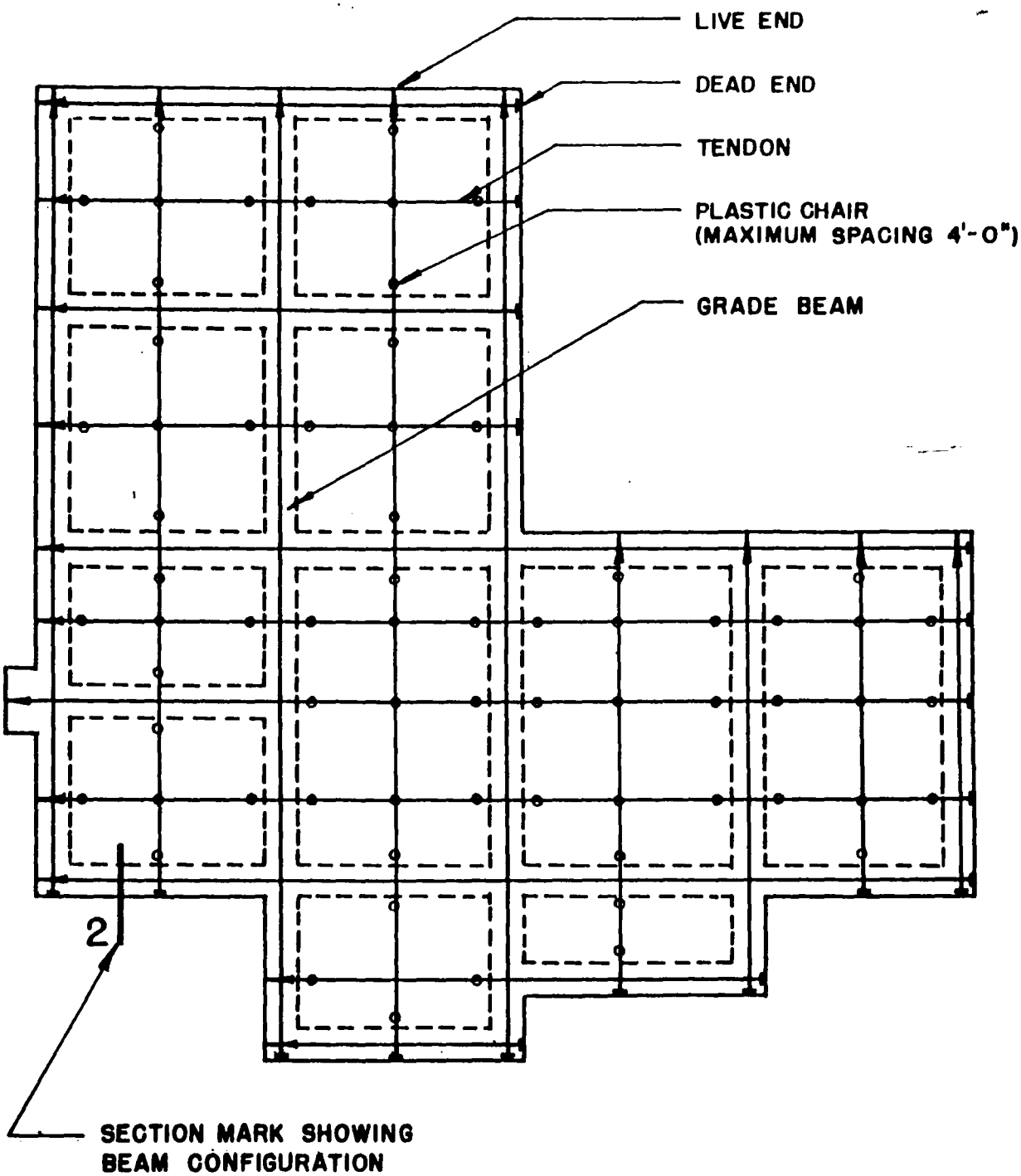
$$f_t = \frac{4.7\,P\,t^{3/4}}{2\,t^2} - f_p$$

$$f_t = 2.35\,\frac{P}{t^{1.25}} - f_p \qquad (47)$$

The constant 2.35 depends upon the assumed value of subgrade modulus, k. The following table illustrates the variation in this constant for values of the subgrade modulus:

Type of Subgrade	k, lb/in^3	c
Lightly compacted, high plastic, Compressible Soil	4	2.35
Compacted, low plastic soil	40	1.34
Stiff, compacted, select granular or stablized fill	400	0.74

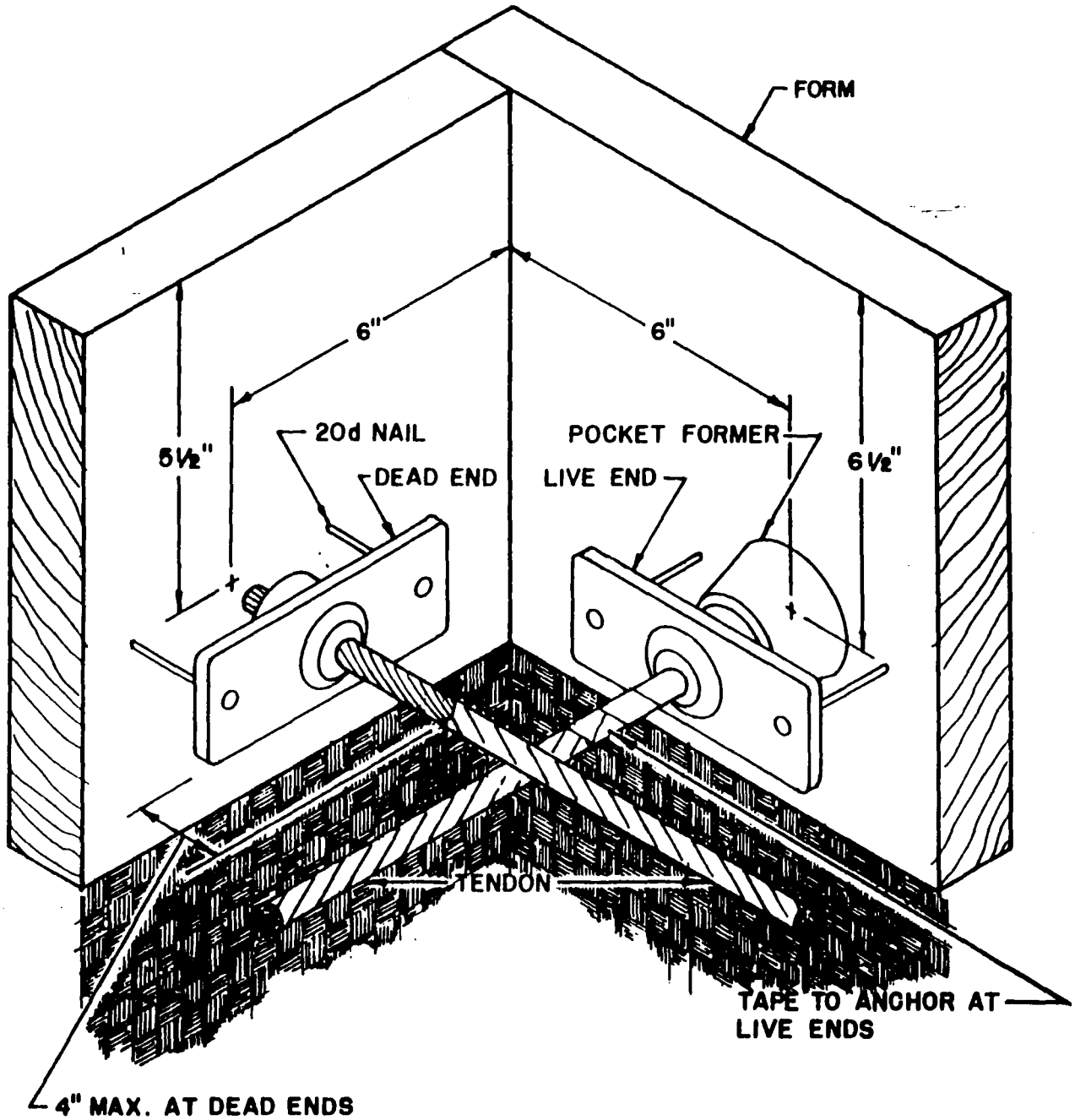
If the allowable tensile stress (say $6\sqrt{f'_c}$) is exceeded by the results of the above analysis, a thicker slab section should be used under the loaded area, or a stiffening beam should be placed directly beneath the concentrated line load.



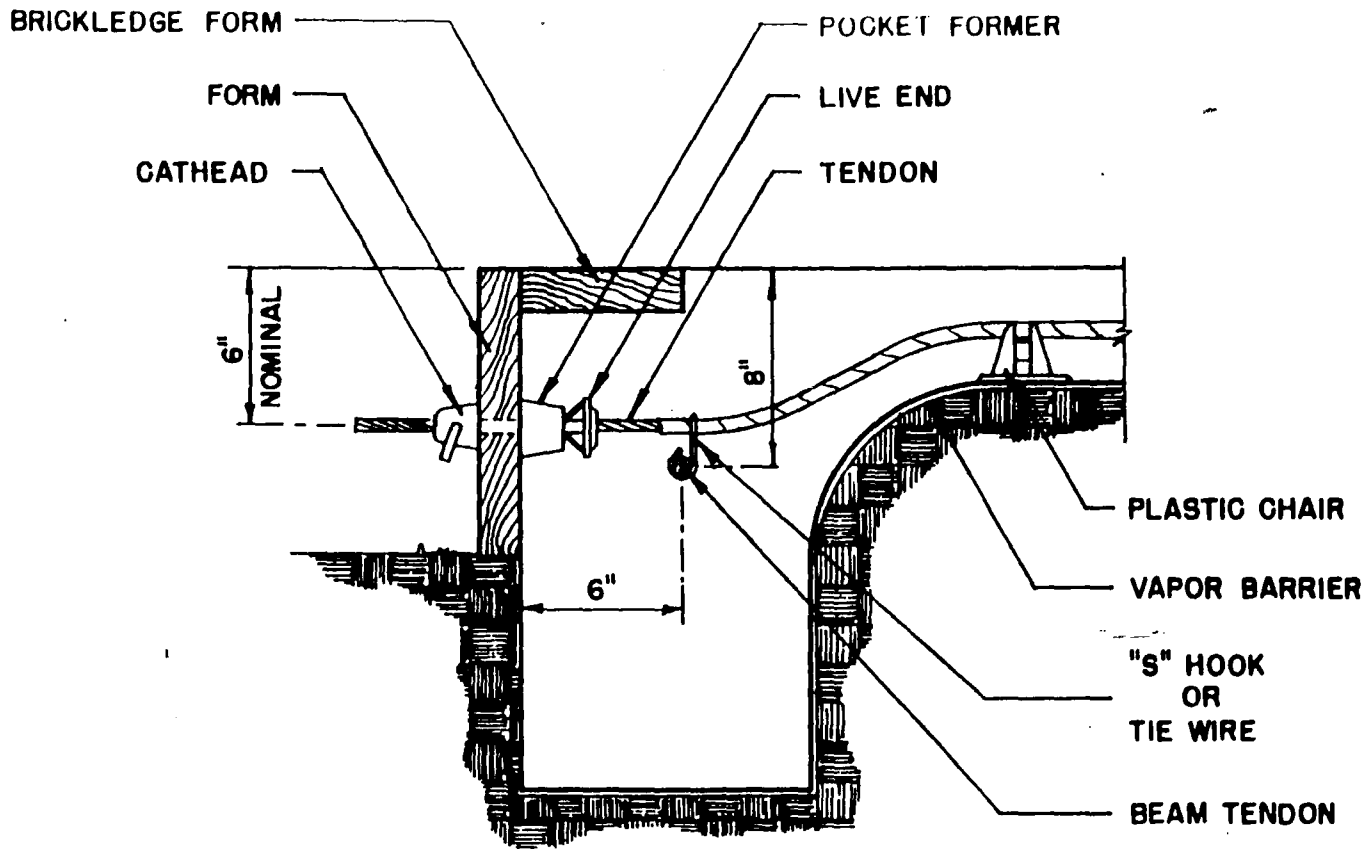
TYPICAL FOUNDATION PLAN

APPENDIX A

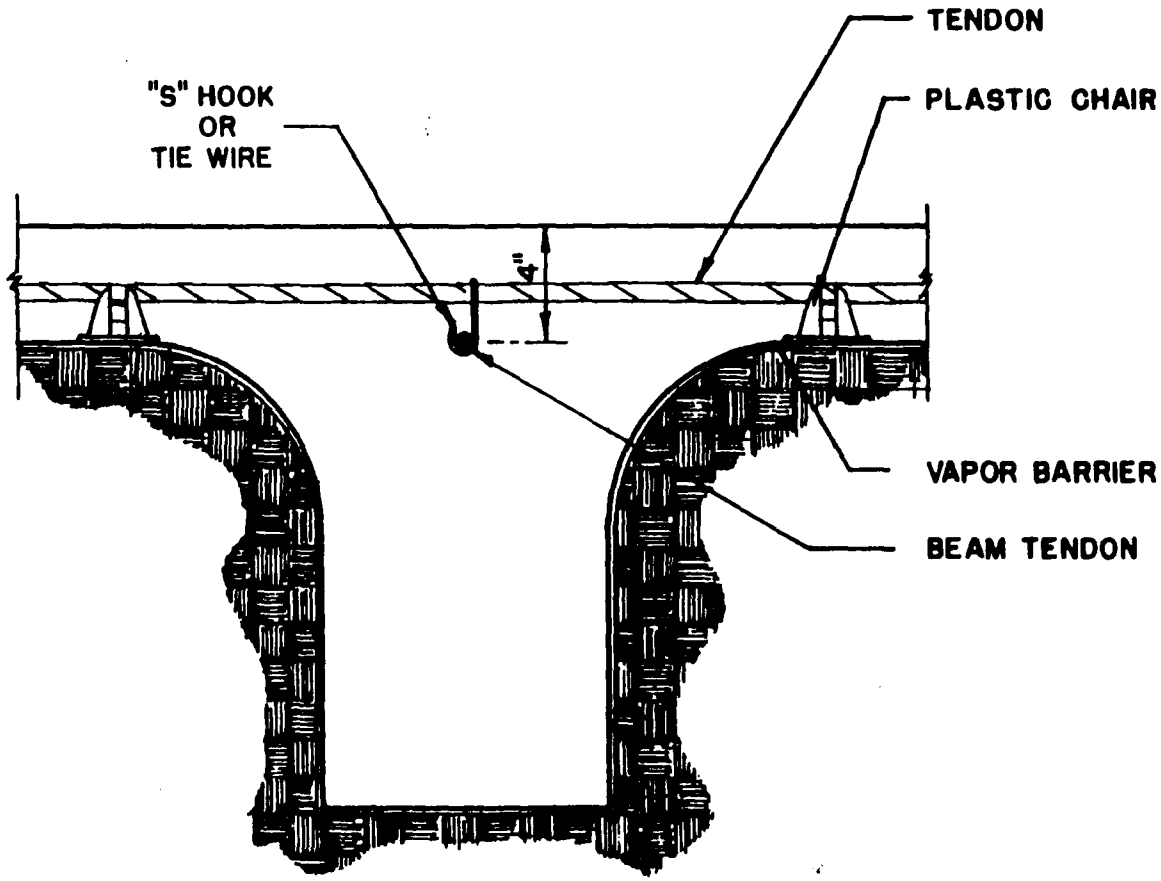
TYPICAL FOUNDATION PLAN AND DETAILS



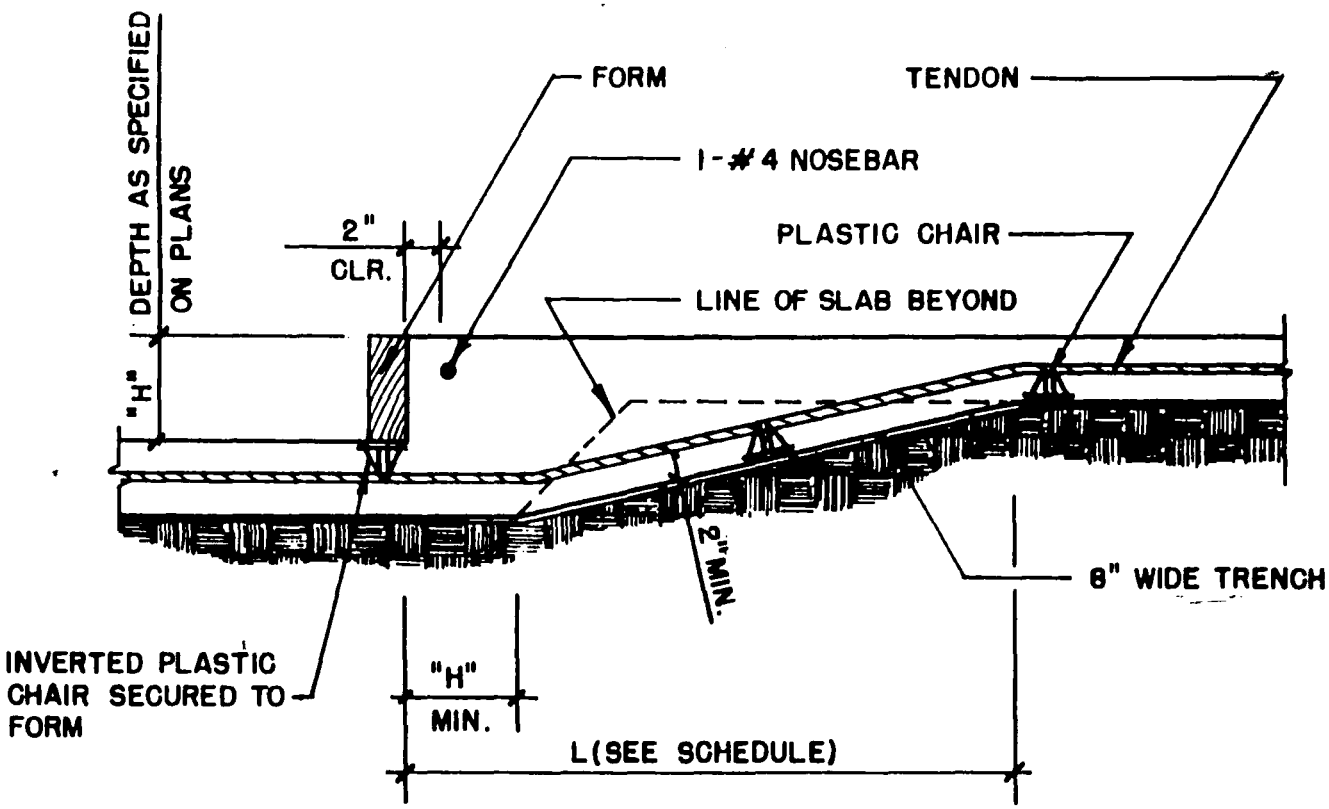
Typical Outside Corner



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

APPENDIX B

TYPICAL GENERAL NOTES

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 2500 PSI.

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	± 1 in. vert., ± 1/2 in. horiz.
Slab tendons	± 1/2 in. vert., ± 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.

FUTURE OF POST-TENSIONED SLABS ON GROUND

by

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Post-Tensioning Institute
American Concrete Institute
American Water Works Association
American Society of Civil Engineers
Structural Engineers Association of Texas
National Society of Professional Engineers
Texas Society of Professional Engineers
American Institute of Steel Construction

Post-Tensioning Institute (PTI) Slab-On-Grade

1. Construction and Maintenance Procedure Manual

The following is a partial list of the areas that will be covered in this document:

- Site conditions
  - effect of old drains, trees
  - impact on design
  - impact on performance
- Soil tests/report
  - why
  - time relevance
  - no. of borings/location
  - soil treatment
  - PTI soil values
  - compaction test
  - recommendations
- Site preparation
  - lot grading
  - drainage plan
  - retaining wall placement
  - piers
  - trenching
  - waterproof membrane
- Construction
  - material - storage & packing
  - forming
  - layout of tendons
  - installation - cover tolerance
    - beam strand profile
    - wobble tolerance
    - back-up bars
    - exposed strand
    - taping
    - embedded items
  - mild reinforcing - grade 40 vs grade 60
    - ungraded steel
  - inspection checklist - before, during and after stressing
  - concrete mix design - cylinders
    - slump tests
    - why
  - form removal - concrete strength
    - grading
  - partial stressing

- stressing
  - power requirement
  - calibration
  - preparation
  - stressing sequence
  - safety procedures
  - elongation
  - cutting of tails
  - grouting of pockets
- Troubleshooting
  - method of crack repair
  - stressing problems
  - elongation problems
- Landscaping
  - proper grade
  - tree type and placement
  - existing trees
  - drought resistant plants
  - placement of sprinkler system
  - planters
- Installation of rain gutters
- Local surface drains
- Maintenance specs for homeowners
  - explanation of expansive soil
  - effects of poor drainage
  - effects of tree or shrub selection on foundation performance
  - effects of location of planters
  - effects of uneven watering
  - effects of failure to water around perimeter
- Repairs
  - causes
  - what can and can't be done
  - cut tendons
- Cracks
  - what is a structural crack
  - causes
  - prevention
  - repair
- Miscellaneous
  - French drain
  - basement
- Detailing standards
- Recommended specifications

- Inspectors
  - qualifications
  - guidelines/what they should do
  - check list
  - legal requirements of different jurisdictions
  - flow chart - pictorial guide of correct procedures

2. Performance Guide for Slab Foundations

This document will include a matrix or combination of distress signs to give assistance in defining foundation problems or failures. It will be used as a performance standard. It will address cracks on interior sheet rock and exterior brick veneer, misalignment of door frames, separations and foundation cracks.

3. Peer Review Program

This will involve the setup of a peer review board, with the goal of providing the public, owners and engineers a resource for confirming structural compliance with the PTI design procedure.

Initially, only preconstruction plans will be reviewed. PTI soils values must be provided. Reviews can only be requested from the design engineer, governmental agency, homeowner or builder/developer. The review will be limited to compliance with PTI Type III slab design procedure and comments shall be limited to any observed deficiencies relating to this procedure.

4. Revision of the PTI S.O.G. Design Manual

This will be done in two phases, with the publication of a revised PTI S.O.G. Design Manual at the completion of each phase. The first phase will cover structural issues and will also incorporate the new PTI documents pertaining to S.O.G construction. Phase 2 will integrate the results of the extensive geotechnical research program into the previous revised Manual.

Phase I

- Design procedures for sloping site conditions
- Shear revisions
- Ultimate strength design provisions
- Guidelines for design of unusual shaped slabs
- Design considerations for track housing, light commercial buildings and unique buildings
- Design details for re-entrant corners
- A section on the use of the PTI design procedure for slabs with reinforced steel and/or wire mesh.
- Incorporation of the slab performance guide
- Incorporation of the Construction and Maintenance Procedures Guidelines

Phase II

- Guidance on what to record in a geotechnical site investigation
- Illustrations on how to determine the depth of the active zone, Z_o and the edge moisture variation distance, e_m for a wide variety of site conditions
- Discussion on the effects of trees, various drainage conditions, vertical and horizontal moisture barriers
- Laboratory testing procedures for measuring suction and cation exchange capacity
- Guidelines and tables for the values of governing suction level, when they are not controlled by the climatic moisture balance.
- Replacement of the tables at the back of our current design guide (differential movement, y_m as a function of the edge moisture variation distance, e_m etc.) with a computer program

5. Computer Programs

- Development of clearly written user's guide with illustrated, worked problems for existing programs - PTISLAB, VOLFLO
- Revisions to the PTISLAB program will include:
  - allowable shear stress dependent on the amount of bonded reinforcement
  - computation of the ultimate strength of the slab in both directions
  - incorporate design rules for moments at re-entrant corners in L-shaped and U-shaped buildings
- Revisions to the VOLFLO program will include:
  - the combination of tree roots and vertical moisture barriers
  - input of initial and final suction profiles in order to calculate the total heave and differential movement
- Development of a new computer program with a clearly written user's guide with illustrations, worked problems
- The new program will be designed to:
  - compute Thoruthwaite Moisture Index from monthly records of rainfall, temperature and latitude
  - calculate the suction-vs-water curve for a soil,given its Attenberg limits and grain size classification
  - calculate the suction at depths below the active zone
  - calculate suction profile corresponding to initially wet, dry or intermediate moisture profiles

NOTE: While every effort will be made to incorporate all of the issues outlined above, time and lack of resources may necessitate some revisions to our intended program.

GUIDLINES FOR GEOTECHNICAL DESIGN, QUALITY CONTROL, AND MAINTENANCE OF RESIDENTIAL PROJECTS IN THE HOUSTON AREA

by

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Biography
David Eastwood, P.E.

David Eastwood is the President of Geotech Engineering and Testing, Inc. Mr. Eastwood has practiced consulting engineering for about 16 years serving in key technical, project management, and administrative roles on both domestic and international assignments. His experience in these functions include a wide range of project types and large capital investments ranging from residential and industrial to commercial buildings. Geotech Engineering and Testing has been a leader in providing soils and foundation engineering services to the Houston area builders, developers, architects, and designers. Mr. Eastwood has conducted soils and foundation explorations and foundation distress studies for a wide variety of projects including a large number of residences, apartment buildings, shopping centers, and office buildings. Mr. Eastwood received his Bachelor and Masters of Science from the University of Houston with specialization in soils engineering. He has several publications on design and construction of foundations on expansive soils. Mr. Eastwood is a member of PTI, GHBA, AIA, ASTM, TSPE, TIBD, ACME, and ASCE. Mr. Eastwood is the Chairman of the geotechnical committee of Post-Tensioning Institute Slab-On-Grade Committee.

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GEOTECHNICAL GUIDELINES

FOR

DESIGN, CONSTRUCTION AND

MAINTENANCE OF

RESIDENTIAL PROJECTS IN THE HOUSTON AREA

By

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Geotech Engineering and Testing, Inc.

Presented at the

Soils-Structure Interaction Seminar

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INTRODUCTION

The variable subsoil conditions in the Gulf Coast area has resulted in very special design requirements for residential foundations. The subsurface conditions should be carefully considered when a subdivision/residence is to be built. Proper planning from the stand point of hazardous waste, subsidence, faulting, soil conditions, design, construction, 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, 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 development. A study like this is 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 hazardous waste 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, and (e) a report of observations and recommendations.

In the event that a Phase I study indicates the potential for the presence of contaminants, a Phase II study is performed. The scope of Phase II study includes: (a) soil and groundwater sampling, (b) chemical testing and analysis, (c) site reconnaissance, (d) contact with state and federal regulatory personnel, (e) and 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. This type of study is not needed for a single lot in an established subdivision or an in fill lot in the city.

GENERAL SOILS AND GROUNDWATER

Geology

The Houston area is located on the Gulf of Mexico Coastal Plain, which is underlain largely by unconsolidated 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 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 soils occur in these areas. The sands are Houston, including generally loose and underlain by relatively impermeable clays and sandy clays. This condition promotes perched water table formation which results in the loss of bearing capacity 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 was beyond the scope of a typical geotechnical report. 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 mentioned in their report. They should then evaluate the affect of any groundwater changes on their design and construction sections of their report.

Some of the groundwater problems areas in Houston include Southside Place, parts of Sugar land, etc. One should not confuse the perched water table with groundwater table. A perched water table occurs when bad drainage exists in areas with a sand or silt layer, about two- to four-feet 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 decisions considering economics, risks, soil type, foundation shape and structural loading. Many times, due to economic considerations, a foundation with greater associated risk may be selected. Most of the time, the foundation types are selected by the owner/builder, etc. It should be noted that some level of risk is associated with all types of foundations and there is no such thing as a zero risk foundation. 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 eight-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. 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 risk 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 Slab	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 Texas are built on this type of foundation and are performing satisfactorily.

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 (such as brick 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 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 four 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, one to two 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.

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 8 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).

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 a lot 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 Active Zone. Active Zone is the zone at which Houston soils experience shrink and swell movements. This depth is about 10-feet. Therefore, we recommend a drilled footing depth of about 10- to 11-feet in the areas where the soils stratigraphy and groundwater depth allow the pier installation. The depth of the active zone should be verified by a geotechnical exploration. 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. Void boxes may be used under the grade beams to separate the expansive soils from the grade beams.

The grade beams for a floating slab foundation should penetrate 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 current post-tensioned slab design manual does not properly model the poor drainage, the effect of the trees, and the depth of the active zone. Revisions to the post-tensioned slab manual is under way to correct some of the short comings. In the mean time, the designer must use experience and judgment when designing post-tensioned slabs. The following design parameters are recommended for design of a post-tensioned slab in the Houston area in parts of the city with highly expansive soils (plasticity indices greater than 40):

Edge Moisture Variation, e_m, feet

Edge Lift	: 5.5
Center Lift	: 4.5

Differential Swell y_m, inches

Edge Lift	: 0.9
Center Lift	: 1.0

Design Plasticity Index : 40 and above

Floor Slabs

The floor slabs for foundations supported on drilled footings should 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 36-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).

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 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 personal experience with void boxes, it is our opinion that they provide an effective feature for reducing swell pressure on the grade beams. However, the validity of void box usage is presently being questioned because of the frequency of observed effects which may outweigh its benefits.

It should be noted that void boxes should be utilized if a structural slab foundation is used in the design.

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.

Vegetation Control

We recommend trees not be planted closer than half the canopy diameter of mature trees from the grade beams, typically a minimum of 20-feet. Alternatively, root barriers must be placed near the exterior grade beams to minimize tree root movements to under the floor slab. This will minimize possible foundation settlement as a result of tree root systems.

In the event that sprinkler systems are used, we recommend that the sprinkler system be placed all around the house to provide a uniform moisture condition throughout the year.

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 height of the bayou.

Any foundation which falls within the hazard zone which extends from the top 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 should 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 should 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 are 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 8 and 20. 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.

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 698-78). This should be confirmed by conducting a minimum of four field density tests per slab, per lift.

3. Minimum concrete strength should be 2,500 psi with a maximum slump of 5-inches. 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.

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

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). 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.

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 four-feet in the building pad area, examining and testing the soils to verify the fill thickness.

Drilled Footing Inspection

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 day 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 floor slab pour. Two concrete cylinders are tested at seven days and two cylinders at 28 days.

HOMEOWNER MAINTENANCE PROGRAM

General

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 and 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 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 is extremely important in minimizing soil-related foundation problems. Sometimes, the homeowners, mount the flowerbed areas, creating a dam, 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 select 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 select fill soils should consist of silty clays and sandy clays with liquid limits less than 40 and plasticity index (PI) between 8 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 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) 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 a 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 9 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. Absence of positive drainage may result in surface water ponding and moisture migration through the slab. This may result in floor slab warping (in the case of wood floors) and tile unsticking.

It is recommended that at least six-inches of clearing be developed between the grading and the wall siding. This will minimize surface water entry between the foundation and the wall material, promoting 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, gutters are recommended all around the roof line. The 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 where poorly compacted backfill fissures and slickensides in the soil mass may allow the water to seep directly into the areas of the foundation and floor slabs.

Landscaping

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.

Planting 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 the 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 North and West Houston, 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 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 North and West 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.

The presence of trees near a residence is considered to be a potential contributing factor to the foundation distress. Our experience shows that large trees in close proximity to residential structures can cause foundation and soil settlements. 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 4-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 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.

Inspections

Every homeowner should conduct a yearly observation of foundations and flatworks 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.

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 jacked precast piles, 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 precast driven piles 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) underneath a 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 jacked precast piers, 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 piers 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 active zone 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/jacked 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 precast concrete jacked 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 jacked piers, 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.

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/driven precast piers 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 geotechnical engineering firm must have a A2LA Laboratory certification in geotechnical engineering.

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

RECOMMENDED QUALITY CONTROL AND INSPECTION

by

PLATT THOMPSON, P.E.
THOMPSON ENGINEERING, INC.

SEQUENCE OF CONSTRUCTION FOR SLABS

CONSTRUCTION TECHNIQUES, DRAINAGE, AND OBSERVATIONS

There are five (5) basic time frames in the proper execution of a structurally sound ground supported concrete slab:

1. Site Preparation
2. Foundation Forms
3. Fill Within the Forms
4. Steel Placement
5. Concrete Placement

1. SITE PREPARATION

A. Prior to Foundation
   - Primary Grading Plan
   - Building Pad or Pads
   - Area Drains
   - Vegetation - Trees

B. Subsequent to Construction
   - Drainage Away From Foundation to Inlets
   - Downspout Drains - Splash Blocks or Underground Air Conditioner Drains, Condensate and Overflow

2. FOUNDATION FORMS

Alignment
Grade
Tightness
Bracing

3. FILL WITHIN THE FORMS

Compaction
Moisture Content
Backfill Around Plumbing Trenches
Grading - Constant Slab Thickness
Beam Shapes, Size and Cleanliness
Membrane Placement

4. STEEL PLACEMENT

Beam Steel, Size, Grade and Location
Corner Bars
Stirrups
Slab Steel, Size, Grade and Location
Compliance with Plans
Supports for Beam and Slab Steel

5. CONCRETE PLACEMENT

Proper Design - Seasonable Requirements - Weather
Mixing Time - "Hot Concrete" - Color
Placement Procedures - Troughs, Pumps, Drops
Slump
Vibration Equipment
Finishing
Curing

SOILS DESCRIPTIONS AND CHARACTERIZATIONS

CLAY

Clays are defined as soils composed of particles less than two microns in dimensions.

The direction and magnitude of the swell potential of clays depends on the manner in which it was deposited; that is by salt or fresh water. Examples are: montmorillonite which was normally deposited by salt water and the molecules are "flocculated" or oriented in every direction and consequently the swell and shrinkage potential is effective in all directions; Illite and kaolinite clays were normally deposited by fresh water and the molecules are basically oriented parallel to the surface of the ground and the swell and shrinkage potential is primarily in the vertical directions. These clays are heavier than montmorillonite.

CLAY MINERALS WHICH MAY BE PRESENT IN ALL FORMATIONS

MONTMORILLONITE	(p.i. range from 40-200) - Montmorillonite is a clay molecule of a ribbon shape and capable of absorbing great quantities of water. Another name often used for this clay is "bentonite."
ILLITE	(p.i. range from 15-40) - Illite is a clay molecule with a basic disc shape and is not usually found in large quantities and considerably less active than montmorillonite in water absorption capacities.
KAOLINITE	(p.i. range from 10-20) - Kaolinite is a clay of rectangular or cubical shape molecule and is the least active of the three basic clays. These clays have very little absorption capacity.

Concrete

Comprised of : Fine & Coarse Aggregate, Cement and Water

Cements - "Portland Cement" - a hydraulic cement consisting essentially of hydraulic calcium silicates usually containing one or more forms of calcium sulfate.

Type I - Most common everyday use
Type II - Used for moderate sulfate resistance or moderate heat of hydration applications
Type III - High early strength requirements
Type IV - Low heat of hydration applications
Type V - High sulfate resistance applications
Type K - Shrinkage compensation cement (Expands first & then shrinks)

Add the letter "A" to type to specify air entrainment. (Always Use)

The "Air" consists of large number of minute air bubbles in the cement paste and have spacings of less than 0.008 inches.

Admixtures:

Accelerators - avoid use of calcium chloride
Water reducing and set controlling - 4 classes
Class 1 & 3 are water reducing and set retarding
Class 2 & 4 are water reducing but usually do not change set time.

Fly Ash:

Three (3) types or class-obtained from burning ground or powdered coal (Class "F" & "C") Class "N" - natural
Class "N" - Calcined natural pozzolans (not usually considered)
Class "F" - Burning anthracite or bituminous coal - contains some pozzolans
Class "C" - Burning lignite or subbituminous coal - contains pozzolans and has cementitious property

Note: Only Class "C" can replace cement in concrete BUT NEVER MORE THAN 20%

FIELD CONTROL OF DEFLECTION

DEFLECTION CONTROL

Simple Span: $\Delta = \frac{5\,w\ell^4}{384EI}$ $M = \frac{w\ell^2}{8}$

Divide Δ by M: $\frac{\Delta}{M} = \frac{5\ell^2}{48EI}$

Cantilever Span: $\Delta = \frac{w\ell^4}{8EI}$ $M = \frac{w\ell^2}{2}$

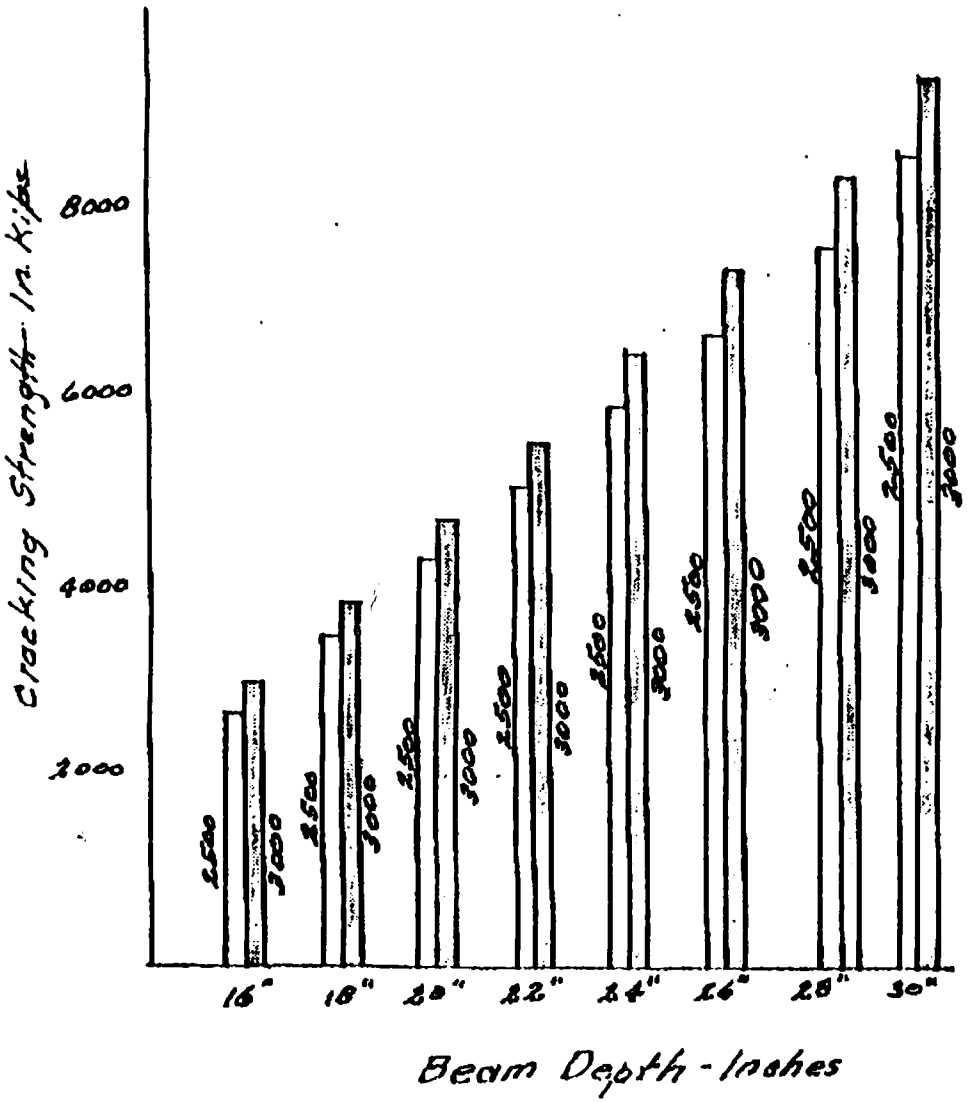
Divide Δ by M: $\frac{\Delta}{M} = \frac{\ell^2}{4EI}$

Field Superintendent cannot change "ℓ", "48", "4", or "E"

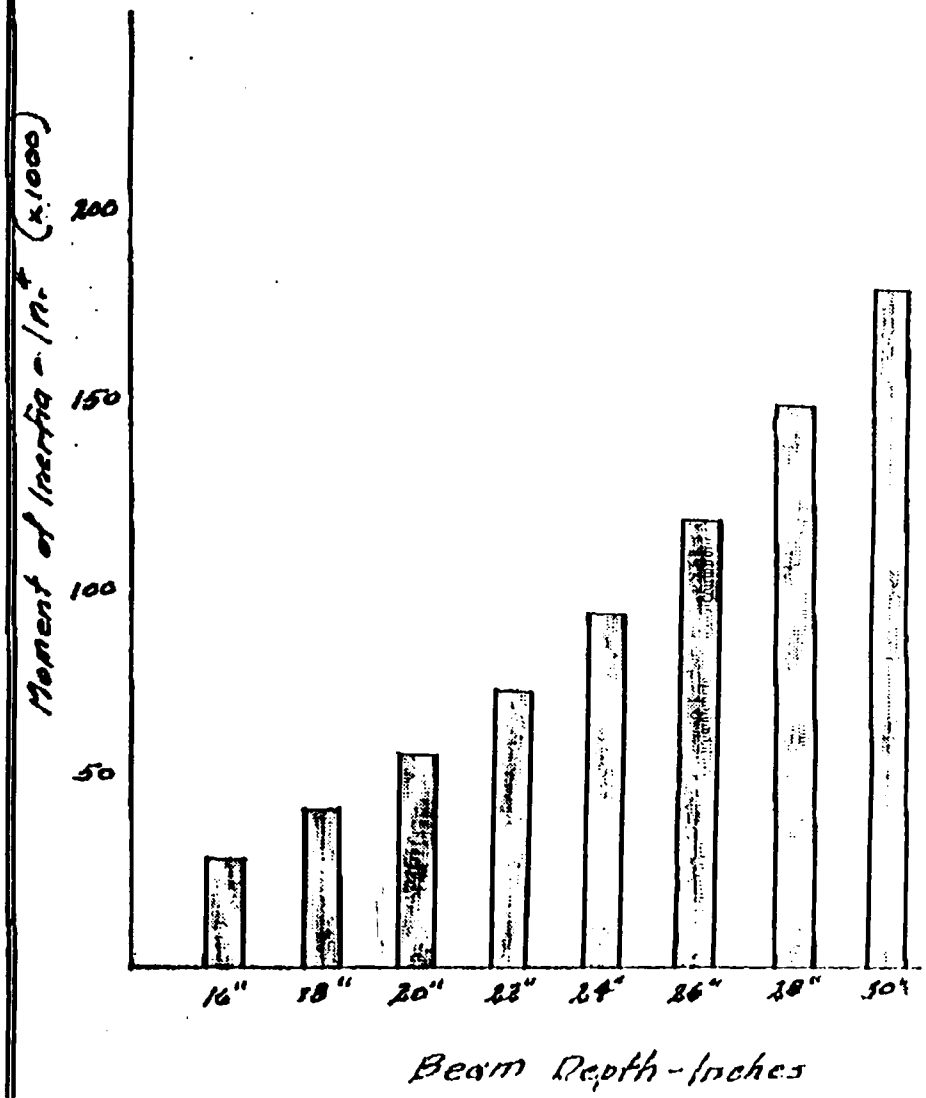
Field Superintendent CAN change "I".

30' Wide Slab with 3-12" Wide Beams

Cracking Strength
2500 psi Conc. Vs. 3000 psi Conc.

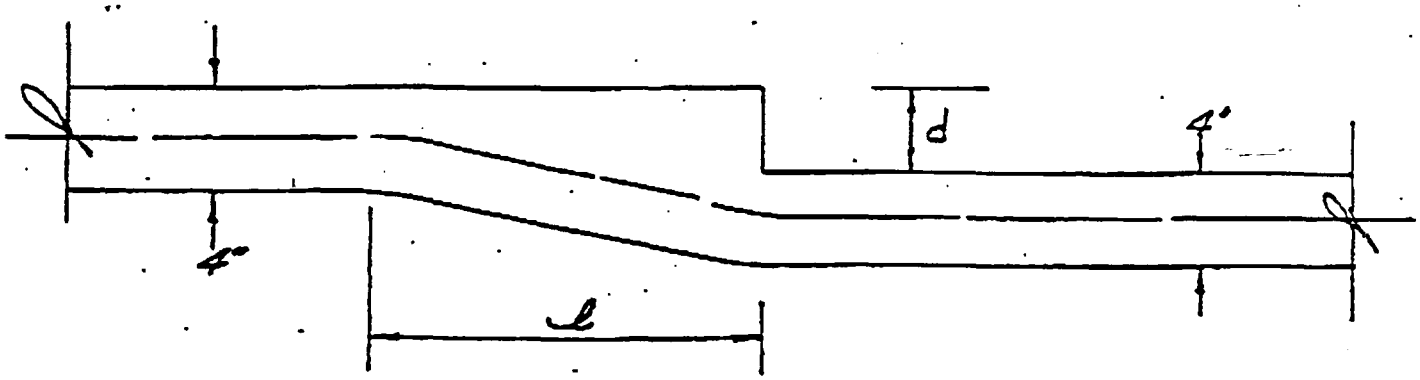


Moment of Inertia Capacity
Resistence to Deflection



THOMPSON ENGINEERING
H. Platt Thompson Engineering Company, Inc.
Consulting Engineers • Surveyors • Environmental Scientists

TRANSITION LENGTHS FOR CHANGE IN SLAB ELEVATIONS



d	l
1"	16"
2"	2'- 0"
3"	3'- 0"
4"	3'- 4"
5"	4'- 0"
6"	4'- 5"
7"	4'-11"
8"	5'- 4"
9"	5'- 7"
10"	6'- 0"

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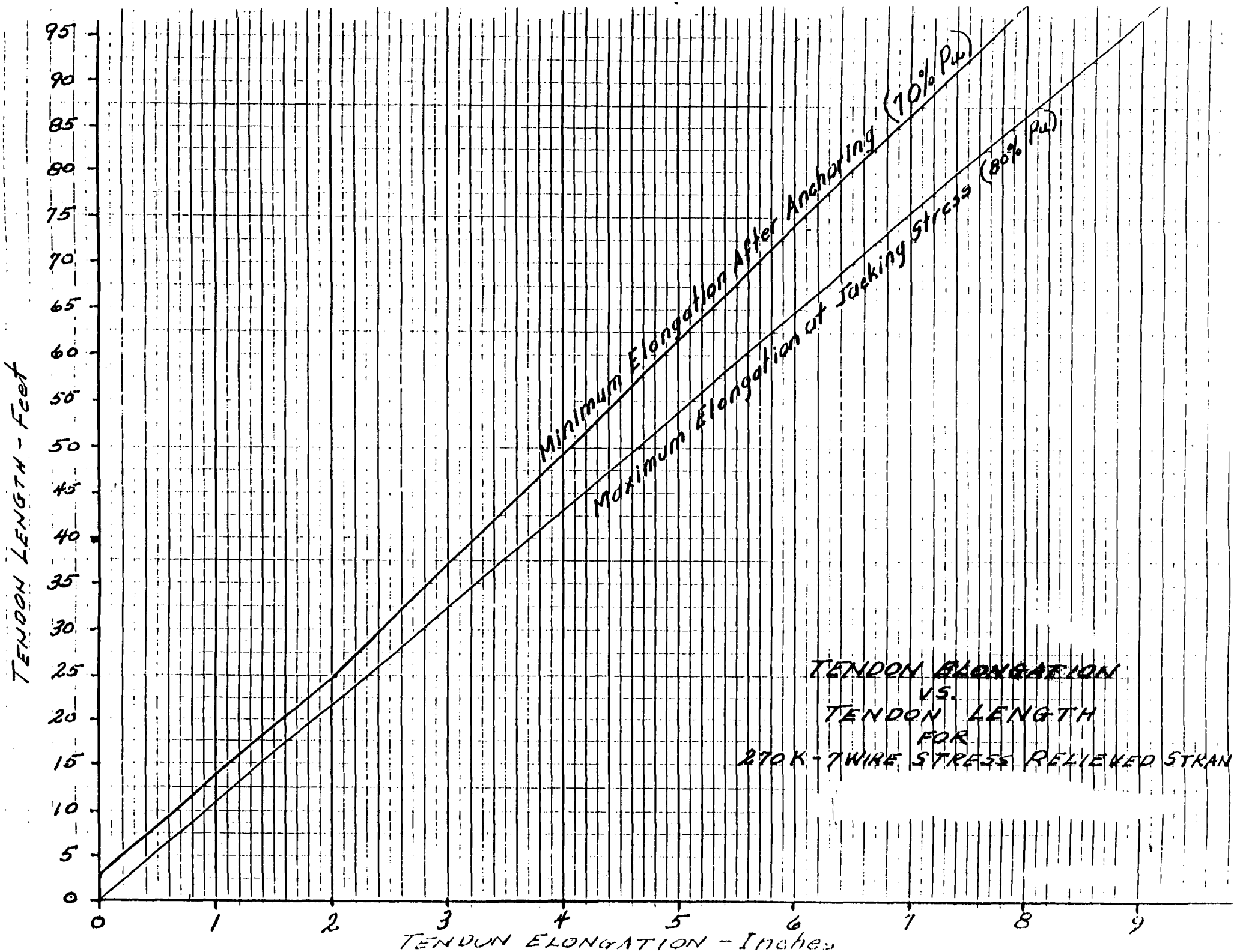
Minimum Elongation After Anchoring (70% P_u)
Maximum Elongation at Jacking Stress (80% P_u)
TENDON ELONGATION
VS.
TENDON LENGTH
FOR
270K - 7WIRE STRESS RELIEVED STRAN
TENDON LENGTH - Feet
TENDON ELONGATION - Inches
0
5
10
15
20
25
30
35
40
45
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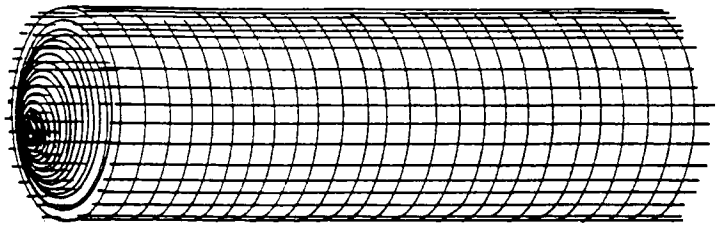

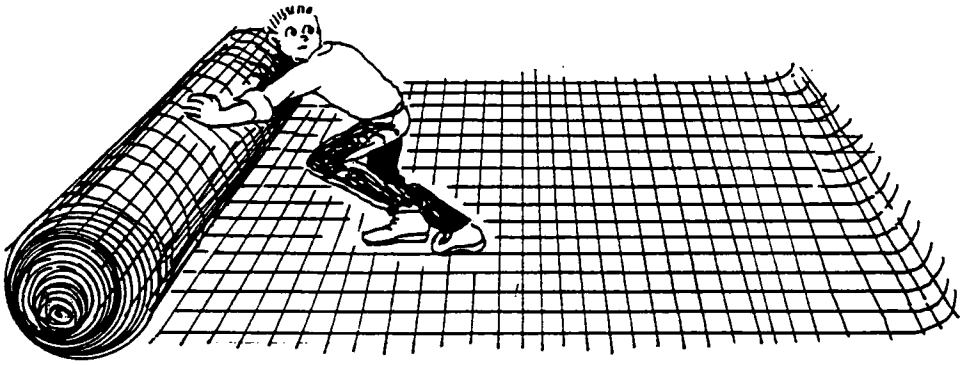

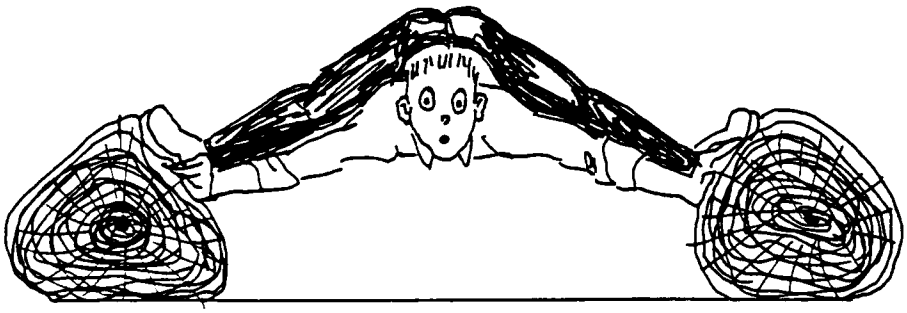

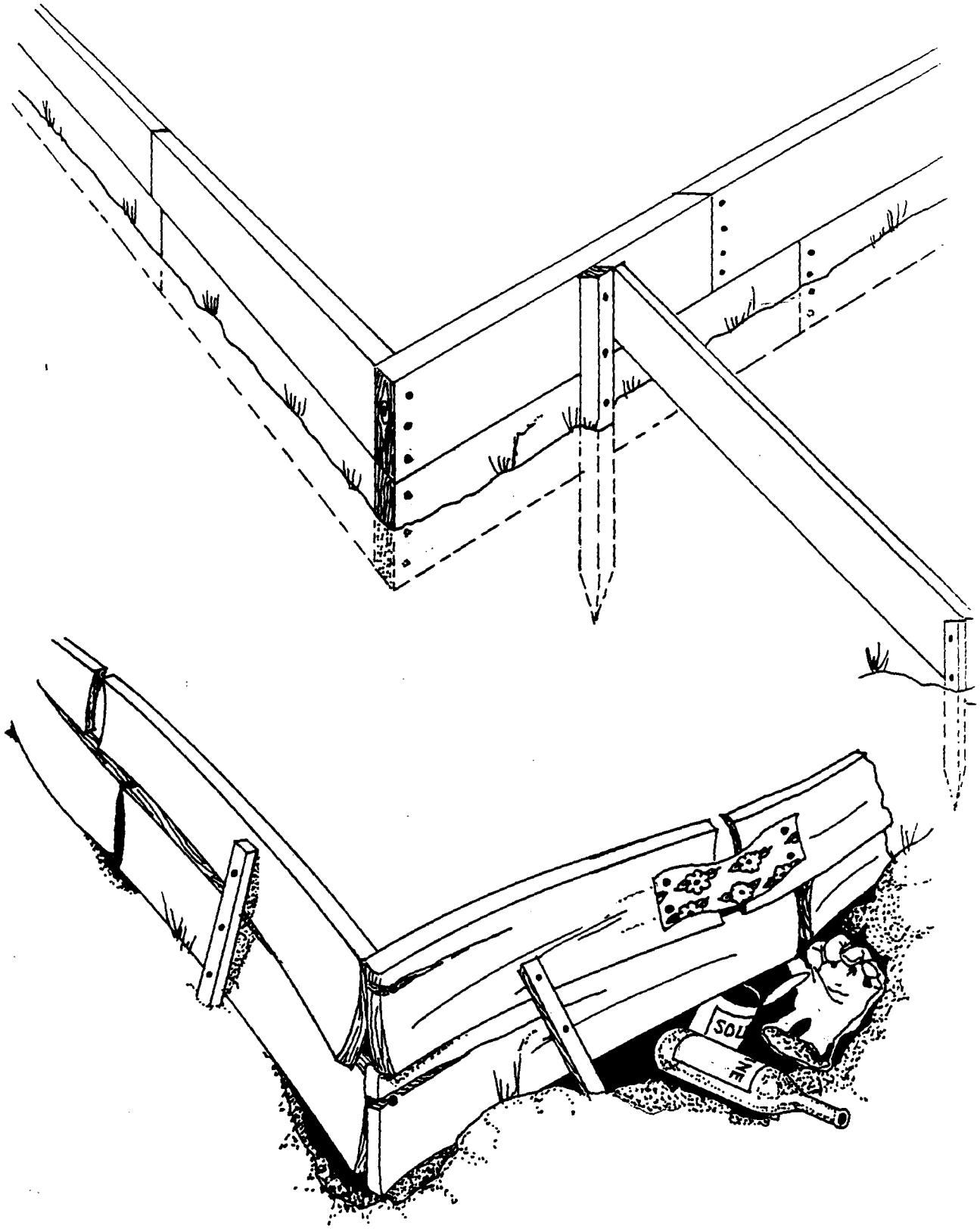
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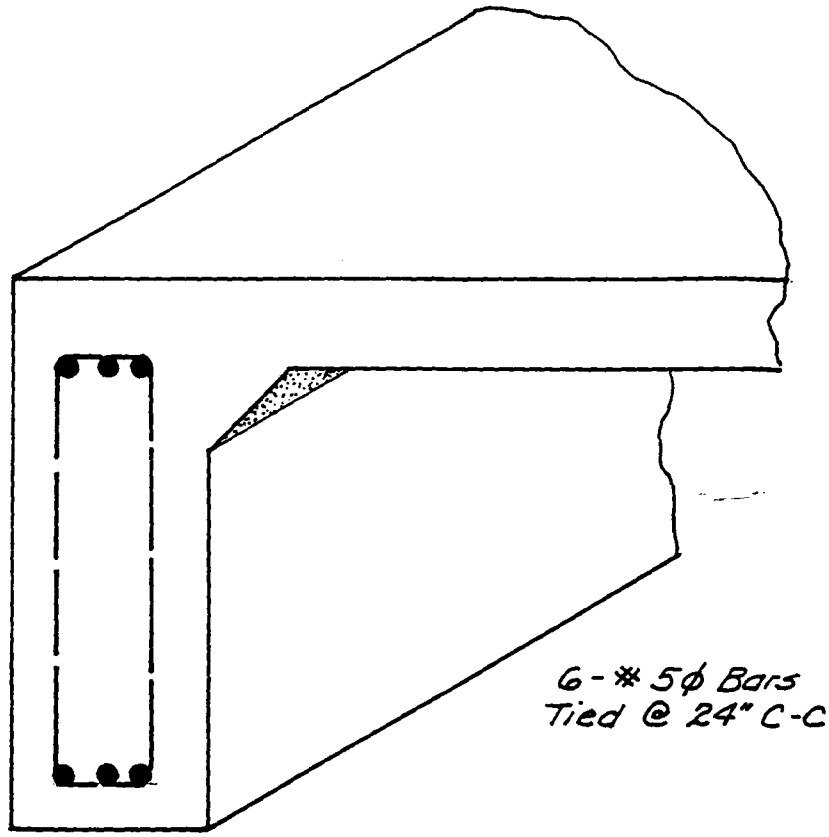
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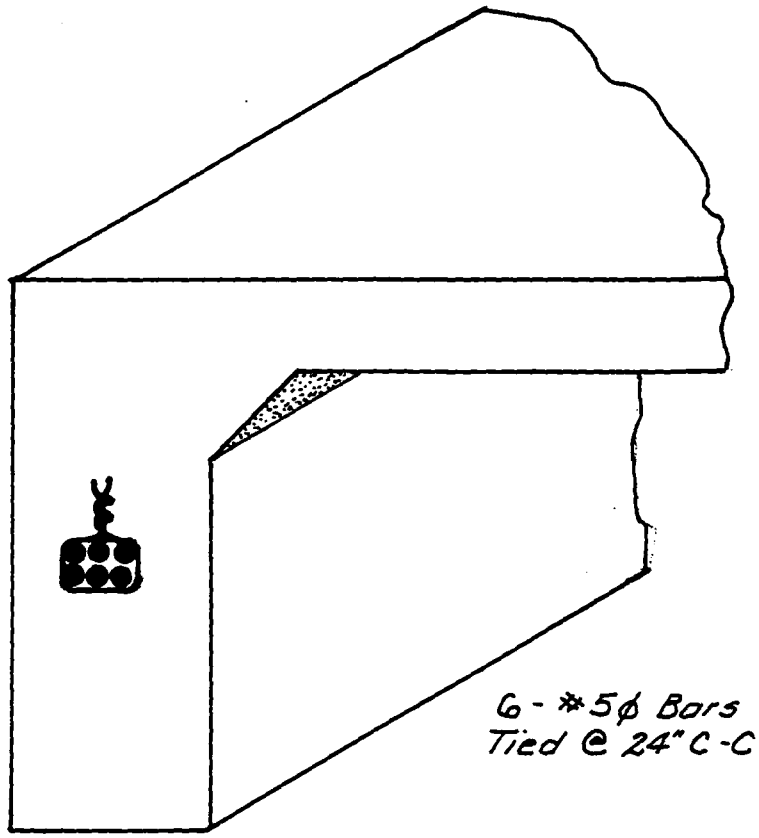
Tied @ 24" C-C


6 - # 5ϕ Bars
Tied @ 24" C-C

FOUNDATION REPAIR TECHNIQUES FOR RESIDENTIAL STRUCTURES

by

DONALD E. LENERT, P.E.
LENERT ENGINEERS, INC.

SOILS-STRUCTURE INTERACTION SEMINAR

SPONSORED BY THE FOUNDATION PERFORMANCE COMMITTEE

July 28, 1994

FOUNDATION REPAIR TECHNIQUES

FOR

RESIDENTIAL STRUCTURES

presented by
Donald E. Lenert, P. E.
Structural Engineer
Lenert Engineers, Inc.
(713) 373-5435
(seminar3.doc LE#31)

PROFESSIONAL RESUME of
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CAREER HIGHLIGHTS

- Founder and President of Associated Engineering Consultants (1966)
  A consulting engineering firm offering structural, civil, mechanical and electrical engineering services. 120 people employed.
- Founder and President of Lenert Engineers (1969)
  A consulting firm offering structural engineering services. This company later became Lenert, Hourani & Assoc. 18 people employed.
- Chief Structural and Civil Engineer for CRS Architects-Abu Dhabi (1979)
  In charge of all structural and civil aspects of a new city for 90,000 people built in the United Arab Emirates. It was the single largest project in the UAE in 1981 (one billion dollars) and included support facilities such as: schools, hotels, apartments, warehouses, shopping centers, road and utility lines and a modern sewage treatment plant. My responsibilities included inspection, contract administration, plan and design checking, quality and schedule control and direction of a team of 7,000 multi-national personnel in the Middle East.
- Present position: Owner, Lenert Engineers, Inc. (1984)
  My position has many duties, such as project engineer, inspector, checker, computer programmer, report writer and general trouble-shooter on various projects all over the United States. I specialize in unusual or special problems on apartment and multi-storied buildings requiring original engineering concepts to solve. I work on projects varying in size from small residences to generating a detailed 400 page architectural and structural report on a $400,000,000 office building and parking garage project in 1990.

PROFESSIONAL EXPERIENCE

My total engineering experience includes over 30 years in responsible charge of projects such as office buildings, schools, shopping centers, churches, hotels and apartments, banks, parking garages, public/governmental projects and industrial facilities. I am qualified as a design structural engineer, expert court witness, field inspector and contract administrator. I am author of many computer programs, engineering reports and technical manuals, a speaker at local seminars and am an experienced general construction estimator.

EDUCATION and BACKGROUND

- Bachelor of Science in Civil Engineering-Texas A&M University 1957
- Registered Professional Engineer - Texas #22916
- Memberships: American Society of Civil Engineers
  International Concrete Repair Institute
  Post-Tensioning Institute

TABLE OF CONTENTS

INTRODUCTION

1) The information in this paper is presented as an introduction to some of the available techniques used in the correction of various structural design and construction problems that exist on residential and light commercial projects, including:

a) Post-tensioned slabs-on-grade
b) Conventionally reinforced "floating" slabs-on-grade
c) Conventionally reinforced "pier-supported" slabs-on-grade
d) "West-University" type foundation systems
e) Light commercial structures

2) As new technologies and techniques are made available, the field of structural failure analysis is expanded. Residential design, investigation and problem solving is more complex than that required for many commercial (shopping centers and multi-storied office) buildings. We recommend that the investigating engineer approach the problems of residential slab-on-grade failure analysis with an open mind and be prepared to revise any initial impressions or preconceived ideas as the actual problems are revealed during the investigative phase.

PROBLEMS

TESTING EXISTING FOUNDATION PERFORMANCE

3) The Building Codes are designed for NEW construction, but do not address standard repair parameters for OLD construction. We recommend that the applicable Building Codes be used as a basis for establishing whether deflections are excessive in any given residential or commercial project. Structural Building Codes nation-wide, including the BRAB Report, the P.T.I. Report, the Uniform Building Code, the BOCA National Building Code, the SBCCI Standard Building Code and the American Concrete Institute *in general* recommend the following criteria:

a) Brick and stucco walls are limited to 1" differential vertical movement in 30' horizontally.
b) Interior surfaces such as roofs, sheetrock walls or wood siding are limited to 1" differential vertical movement in 20' horizontally.

DIFFERENCE BETWEEN "DESIGN" AND "PERFORMANCE"

4) The Building Codes are quite clear that engineers should only use one value (usually the most restrictive) for *design*, but a single deflection value may not be appropriate for *performance*. For example, the Houston Building Code (UBC) stipulates in Table 23-D that a roof or floor member supporting live and dead loading not deflect more than 1:240, while a member supporting masonry is limited to 1:500 to 1:600 maximum deflection. While it may not be possible to design the components of a single building foundation using separate deflection criterias, it may be appropriate to analyze it for performance using applicable criterias for different materials.

MICRO-ELEVATION SURVEY PLAN

5) We recommend the micro-elevation survey as one of the best techniques for assessing an existing structural system. This becomes especially important when the original plans and design are no longer available for review. In such cases, performance may be the best criteria to be used to rationally evaluate a foundation without becoming involved with costly destructive and non-destructive testing.

6) A micro-elevation survey may be made using a surveyor's level instrument, a laser survey instrument or a water level (manometer). We prefer the latter for residential projects since there usually are so many rooms and partitions involved that too much time would be wasted setting up the first two instruments. A water level, properly used, should be able to achieve an accuracy of 1/16" to 1/4", depending on the length, tubing diameter and time allowed to achieve each reading.

PHENOMENA PLAN

7) On a building that is being investigated for potential foundation problems, design and construction are already completed. The performance may be assessed by the use of micro-elevations, but should be compared with existing negative phenomenon, including many of the following items that are shown on the Phenomena Plan:

a) Obviously sloping floors; binding doors or "pie-shaped" gaps above door heads;
b) Diagonal or vertical masonry and sheetrock wall cracks;
c) Brick chimney leaning outwards away from main building;
d) Separations in ceilings and crown moldings;
e) Concrete slab or floor tile cracks;
f) Baseboard separations at the interior walls;

g) Floor tile pulling away from baseboards;
h) Presence of algae at the exterior, indicating super-saturated conditions exist;
j) Existence of earth cracks next to the exterior walls, indicating that an extremely dry condition exists;
k) Separation of wood trim from adjacent wall elements such as at windows, brick, stucco or sheet rock; "Popped" nails in sheetrock walls;
l) Pulling away of roof frieze boards (typically a 1" x 4" below the roof soffit); Roof leaks despite roofing material that is in good condition;
m) Horizontal brick cracks just above the grade beam may indicate a differential settlement, even though there are no corresponding diagonal wall cracks;
n) Poor concrete and reinforcing placement or lack of curing; "Soft" concrete with lack of good durability.
p) Corner sheetrock cracks at the top of windows or doors.

9) The micro-elevation survey plan and phenomena plan should be compared to confirm whether differential foundation movements have occurred. The presence of a change in the slab elevation (slope) does not necessarily mean that differential movement has occurred. We may assume that the construction of slabs-on-grade is seldom closer than 3/4" in vertical elevation control, so it is essential that any measured sloping floor systems be accompanied by some type of negative phenomenon, such as itemized above. If these are not present, then it is quite possible that the building was originally cast out of level.

GEOTECHNICAL AND SITE PROBLEMS

9) Many slab-on-grade problems could be easily avoided if normal geotechnical design criterias were met. In general, a majority of the settlement and/or differential movement problems associated with slab-on-grade foundations are due to factors including:

a) Flat back yards with poor drainage (poor siting).
b) Lack of thick select fill pad to help absorb movements between expansive clays and foundation system.
c) Lack of adequate compaction.
d) Lack of adequate grade beam embedment.
e) Flower bed reverse-drainage problems or presence of deep, water-retaining mulch or sand.
f) Lack of proper (owner) watering techniques.

SOLUTIONS

SITE WORK

10) Poor siting and inadequate grade beam embedment deficiencies are often found. One of the basic techniques used quite frequently in controlling excessive cracking in foundations is to "desensitize" the exterior grade beams from the effects of sudden moisture changes. This technique includes the following:

a) Adding additional slope to divert water away from the building.
b) Extending downspout discharge away from the foundation.
b) Installation of French drain systems that allow surface water collection and dispersal into the street or storm sewer system using either natural drainage or sump pumps.
c) Installation of water barriers underneath the existing grade, such as 3 layers of polyethylene installed 12" below finish grade and 1' to 5' away from the building.
d) "Leaky pipe" installation to help maintain uniform moisture conditions around portions of the building.

LEVELING TECHNIQUES

11) We do not believe that leveling piers are the only solution to level buildings, especially since the appraisers will typically devalue a property if foundation leveling has been done. We do believe that leveling piers should be the <u>last</u> option used to correct a foundation, only after other methods have been ruled out or are not deemed adequate. The following methods are always used to <u>raise</u> a foundation system:

a) Drilled bell-bottom leveling piers
b) Pressed piles (concrete or steel)
c) Helical anchors
d) Polyurethane injection
e) "Leaky pipe" watering systems

CONTROL JOINTS

12) Overall foundation movements sometimes are not sufficient to violate the Building Code parameters, yet excessive cracking phenomena exists. Rather than use drilled piers, with the accompanying loss of property value due to major foundation repairs, control joints may be strategically placed to divide long masonry walls into smaller panels so that the walls can better absorb normal thermal or differential foundation movements without cracking. This is a well known "secret" in the apartment industry, where knowledgeable builders often install alternate panels of brick and wood wall construction as a technique for reducing visible distortions in minimal foundation systems.

13) Often thermal cracks in brick walls are interpreted as differential movement problems. One method to control minor differential movement problems in buildings having brick veneer is to install vertical control joints at 10' to 20' spacings where large areas of window and/or door openings occur.

FRENCH DRAINS

14) A popular technique for correcting poor site drainage is the use of perforated PVC pipe in a gravel bed. Bank sand is installed above the gravel, and sod is placed on the top to match the adjacent ground. The sand and gravel pipe should be separated by a screen, or the pipe should be wrapped with a screen (to prevent infiltration of sand into the gravel). The PVC pipe slope varies between 1/16" to 1/8" per foot, and the end of the pipe discharges through the concrete curb in the front of the street.

ADDITIONAL CONSIDERATIONS

15) What is the correct elevation to raise a foundation? If 50% of foundation needs to be raised, should piers be placed at the portion that is presently performing satisfactorily? If builder's piers are present, should they be used to level or reused with new piers? The engineer should consider these questions when evaluating the final corrective design system to be used.

16) When a foundation system is raised, a void space is created under the grade beams and slab areas. A 4" thick concrete slab will span not more than 8' before the deflection becomes too large. When are these voids to be filled? Here are a few general ideas:

a) Fill if the P. I. (plasticity index) is less than 40% and the voids are greater than 1/2".
b) Fill if the P. I. is between 40% and 60% and the voids are greater than 1".
c) Voids under grade beams are not needed on sands and silts, but may be desired on highly expansive clays (or if clays on site are very dry at the time of leveling). Use grade beam voids ONLY if there is good drainage away from the building.

POLYURETHANE INJECTION

17) Polyurethane injection uses chemicals that react with water as the primary force to lift and relevel a foundation system. Holes are drilled through the floor slab at approximately 5' centers, and polyurethane foam is pressure injected. The proportions are determined by the applicator, and vary depending upon factors such as amount of lift, partition or wall loads present and degree of accuracy required. The advantage is that the polyurethane can be designed to set up very rapidly, and thus a very high degree of accuracy in the area and amount of lift can be controlled.

MUD-JACKING

18) In mud-jacking, a fine dirt is mixed with cement to form a slurry that is injected under pressure under the slab. Either a long pipe is used to penetrate below the grade beam and fill the voids, or holes are drilled at approximately 5' to 10' centers in the slab. Mud-jacking is usually okay on sands and silts, but should be used with care on expansive clays. The amount of control on the amount of vertical lift varies from very poor to marginally satisfactory.

MOISTURE BARRIERS

19) Sometimes one end of the foundation is found to have heaved upwards from swelling clays next to deeply mulched flower beds or flat back yards. Does this mean that the part of the foundation that is functioning without problems must now be raised to the level of the foundation that is not? If a foundation needs to be lowered, one method is the polyethylene moisture barrier. It is important that a route be left for trapped water to get out of the interior of a foundation system on the "downhill" side. If this is not provided, a future leaking sewer line may inflict more damage than the original problems!

PIPE TRENCHES

20) Pipe trenches have traditionally been installed using bank sand as the backfill material. Sand is dumped into the open trench and water-compacted to achieve the necessary Proctor density. Sometimes these same pipe trenches become a channel for transmitting excessive water into the interior of the building, and heaving of the clay soils and cracking of the interior finish materials may result. Although seldom addressed by structural engineers, care should be taken to specify that the ends of pipe trenches be sealed off at the perimeter of the foundations. Suggested corrections to existing problems of this type include: water barriers, clay fill, French drains, polyurethane injection or other methods.

CHECKING TENDON STRESSES

21) How do we know if post-tensioned tendons have been stressed? A clue to stressing not having been done is an unusually high incidence of shrinkage cracks, especially on the sides of the exterior grade beams. Chipping out at least two live end anchorages on each side and inspecting the shims, one can see if they are properly seated or not. If it is not clear whether or not they are, then the tendons should be tensioned to 25 to 50% of the maximum design stress using a calibrated hydraulic ram jack. If the shims do become not unseated, it may be assumed that at least a percentage of the design tensile force is present. We do not recommend that the full design force be applied, since should the gripping shims slip, the cable end may end up several inches inside the grade beam and very costly structural repairs would then be required.

SOIL STABILIZATION

22) Available soil stabilization methods for existing buildings:

a) Condor SS injection (an ionization treatment for clays)
b) Lime injection
c) Water barriers
d) Polyurethane injection
e) Improved moisture control using deep sump pumps
f) Barriers at pipe trench entrances to foundation system
g) "Leaky pipe" systems

---------- end ----------

APPENDIX- DRAWINGS

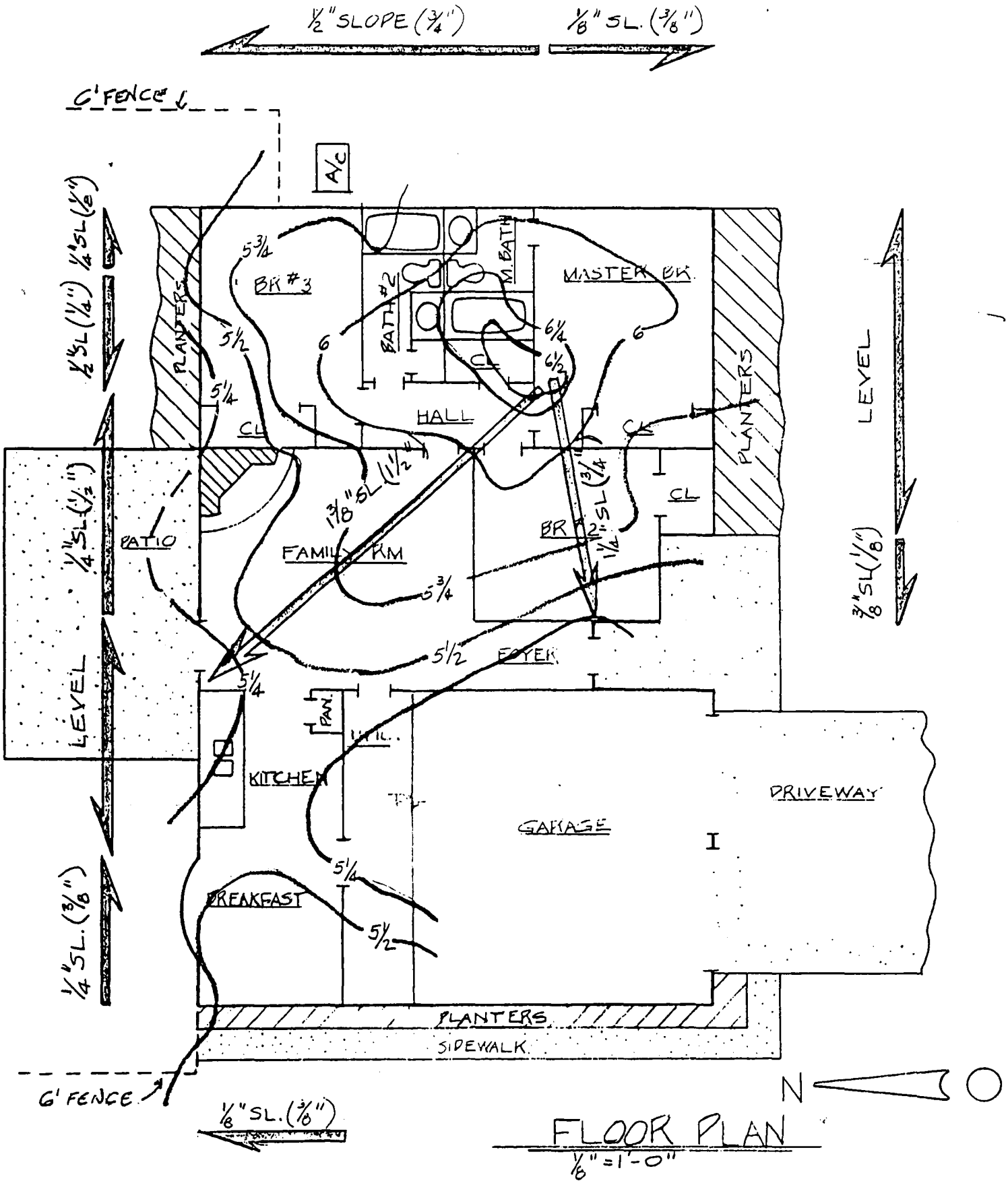
SAMPLE FOUNDATION PROBLEMS & CORRECTION TECHNIQUES

SK-1, 2 & 3	Plumbing line leak - claim accepted
SK-4	Normal center lift - condominium
SK-5,6 & 7	Edge lift requiring leveling piers and polyurethane injection
SK-8	Extreme tilting
SK-9 & 10	Localized drilled footing failure
SK-11	Plumbing leak - claim denied
SK-12	French drain and polyethylene moisture barrier
SK-13	MIRAFI impervious moisture barrier
SK-14	Concrete leveling pier
SK-15	Concrete pressed pile
SK-16	Description - Perma-jack system
SK-17	View - Perma-jack system
SK-18	Detail - steel pipe piling
SK-19	Helical anchors
SK-20	URETEK (polyurethane injection) method
SK-21	HYDROPIER method - (modified "leaky pipe")

EXAMPLE OF PLUMBING LINE LEAK - CLAIM ACCEPTED

SK-1

MICRO-ELEVATION CONTOUR PLAN



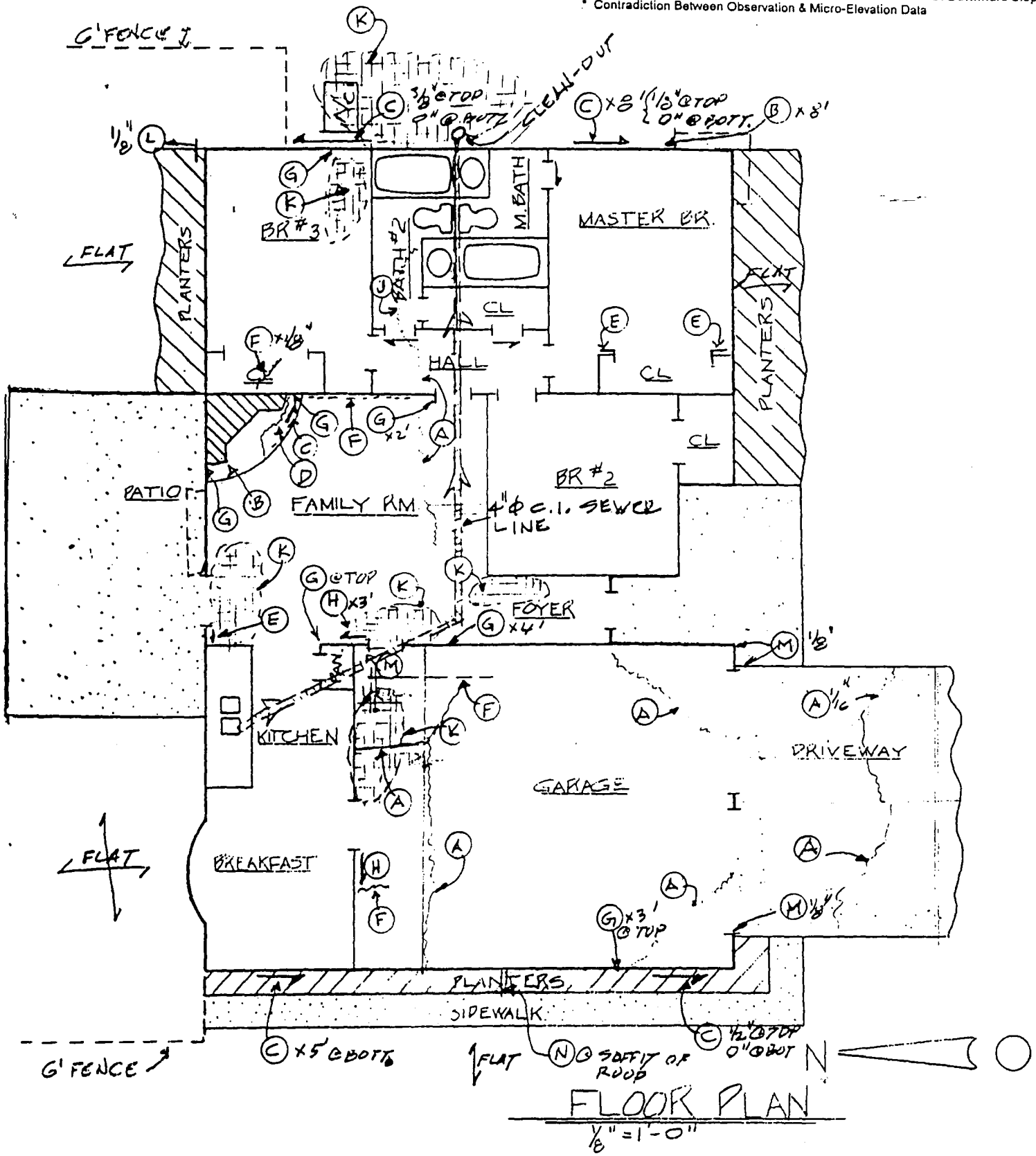
EXAMPLE OF PLUMBING LINE LEAK - CLAIM ACCEPTED

SK-2

PHENOMENA PLAN

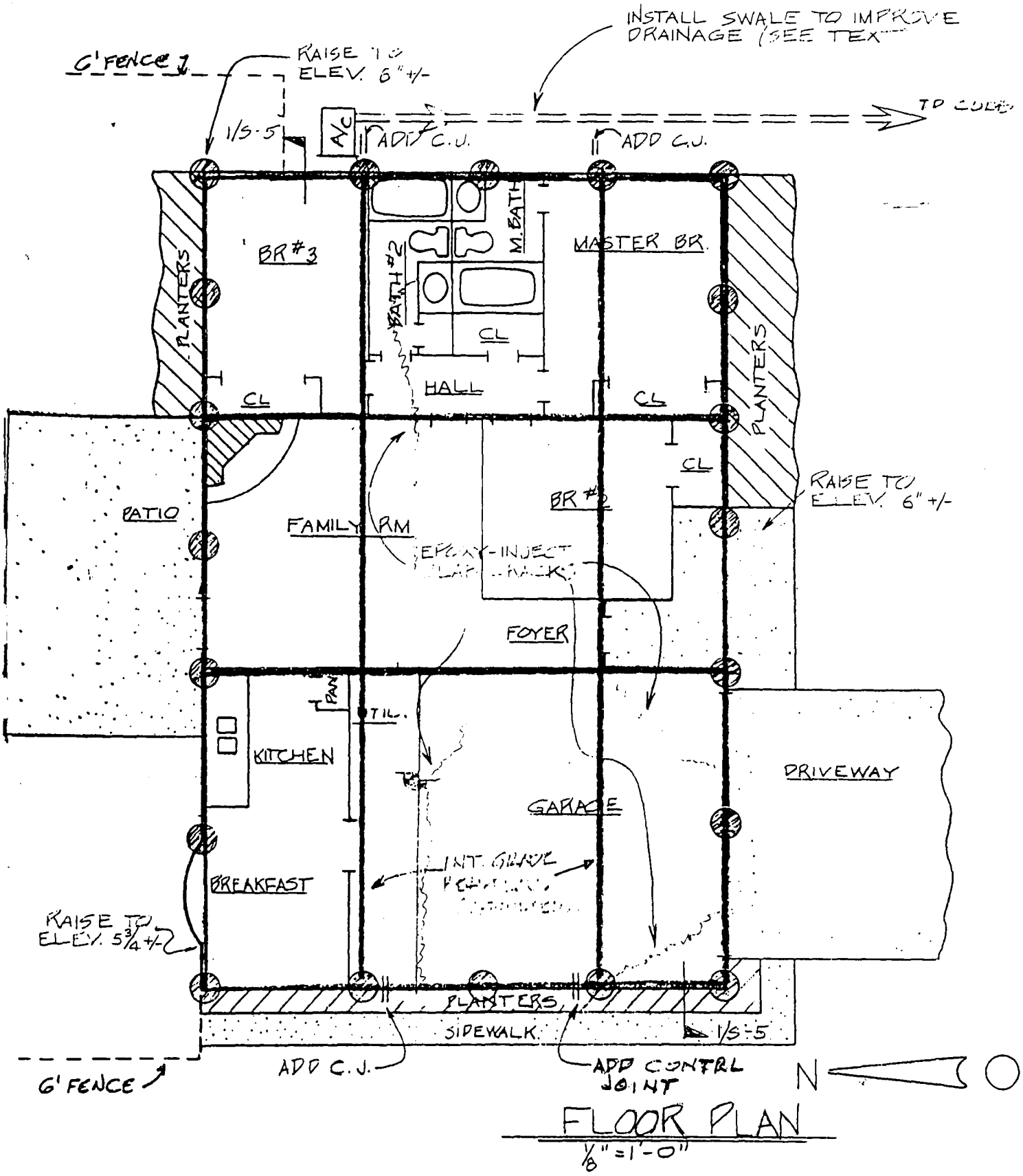
Notes:
A) Concrete Crack
B) Vertical Brick Crack
C) Diagonal Brick Crack
D) Horizontal Brick Crack
E) Horiz. Shtrock Crack @ Window/Door Head
F) Ceiling Sheetrock Crack
G) Vertical Sheetrock Crack
* Contradiction Between Observation & Micro-Elevation Data

H) Diagonal Sheetrock Crack
J) Horizontal Tile Crack
K) Wet Area
L) Roof Frieze Board Movement
M) Vertical Trim Separation
N) Horizontal Trim Separation
<------ Direction Of Downward Slope



FLOOR PLAN
1/8" = 1'-0"

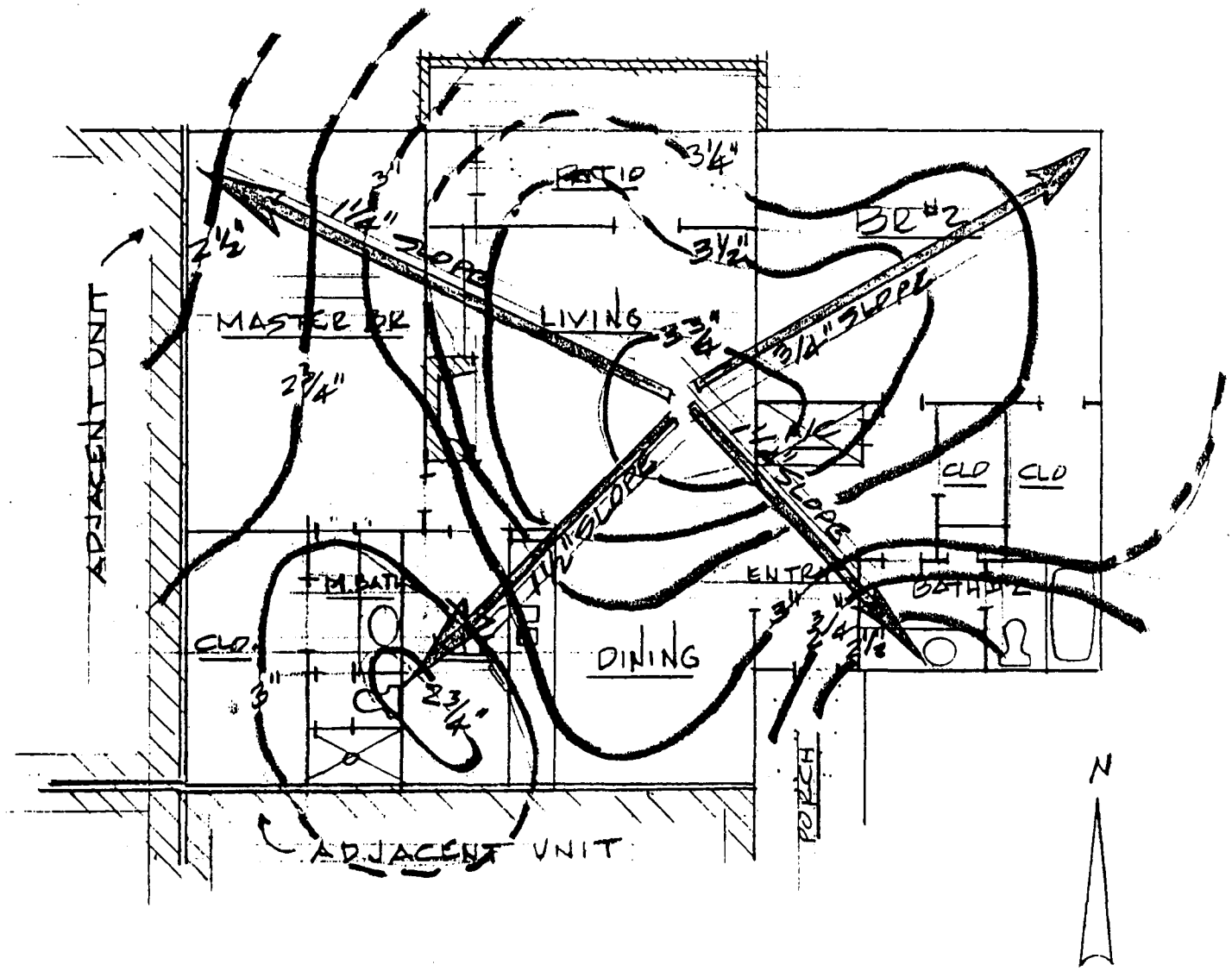
REPAIR PLAN



FLOOR PLAN

1/8" = 1'-0"

MICRO-ELEVATION CONTOUR PLAN



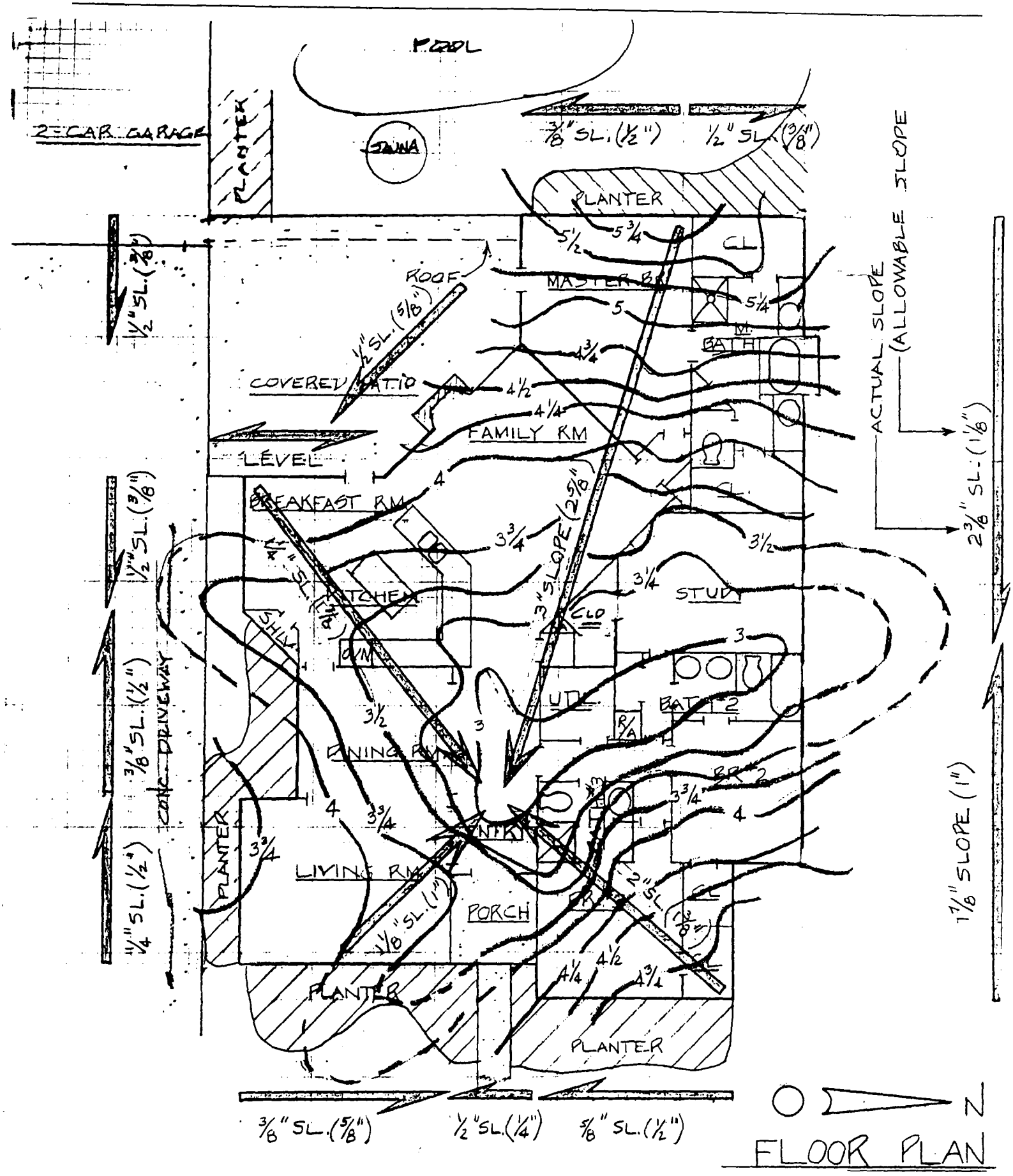
FLOOR PLAN

APPROX SCALE: 1/8" = 1'-0"

EXAMPLE OF EDGE LIFT REQUIRING
LEVELING PIERS AND POLYURETHANE INJECTION

SK-5

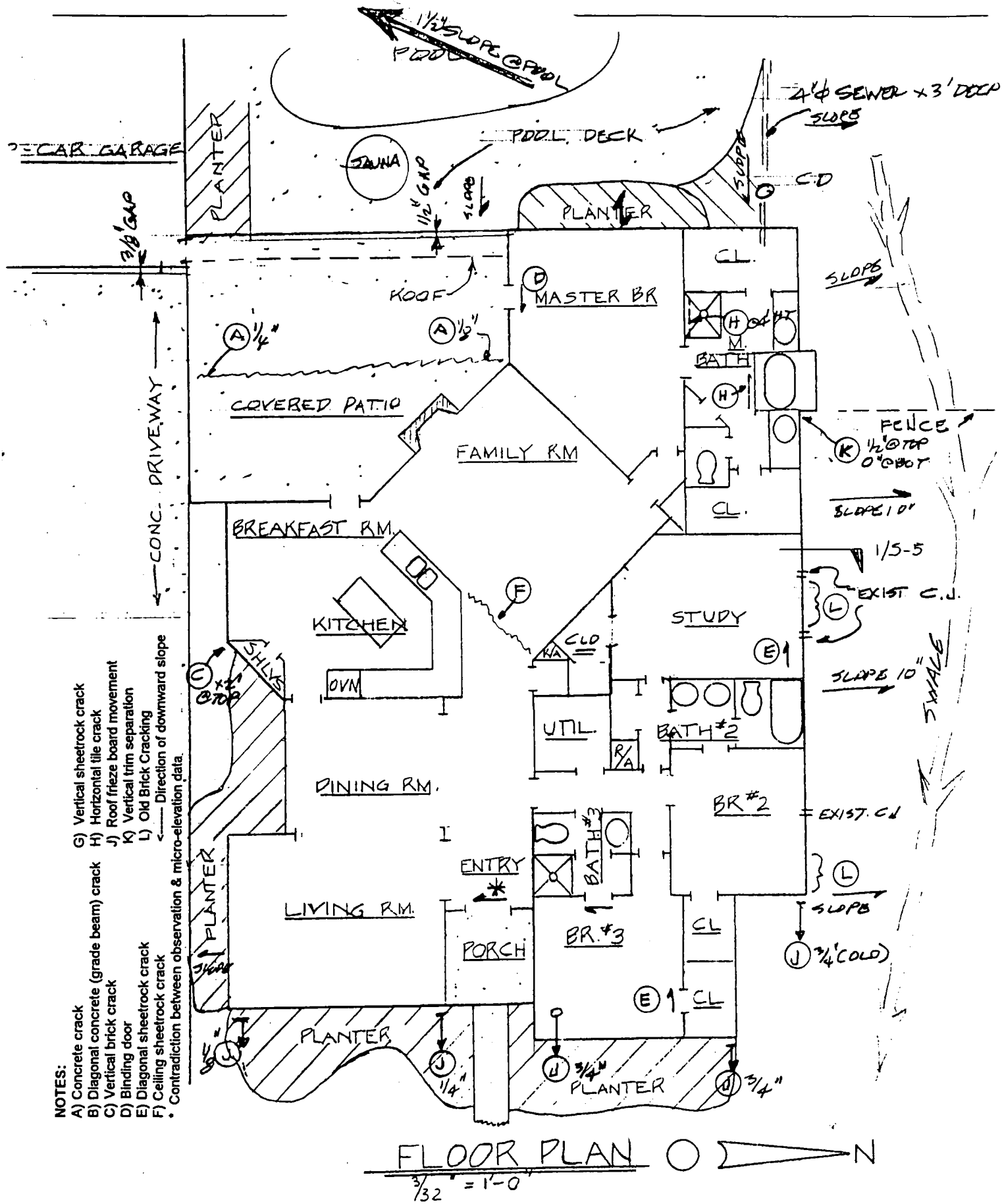
MICRO-ELEVATION CONTOUR PLAN



EXAMPLE OF EDGE LIFT REQUIRING LEVELING PIERS AND POLYURETHANE INJECTION

SK-6

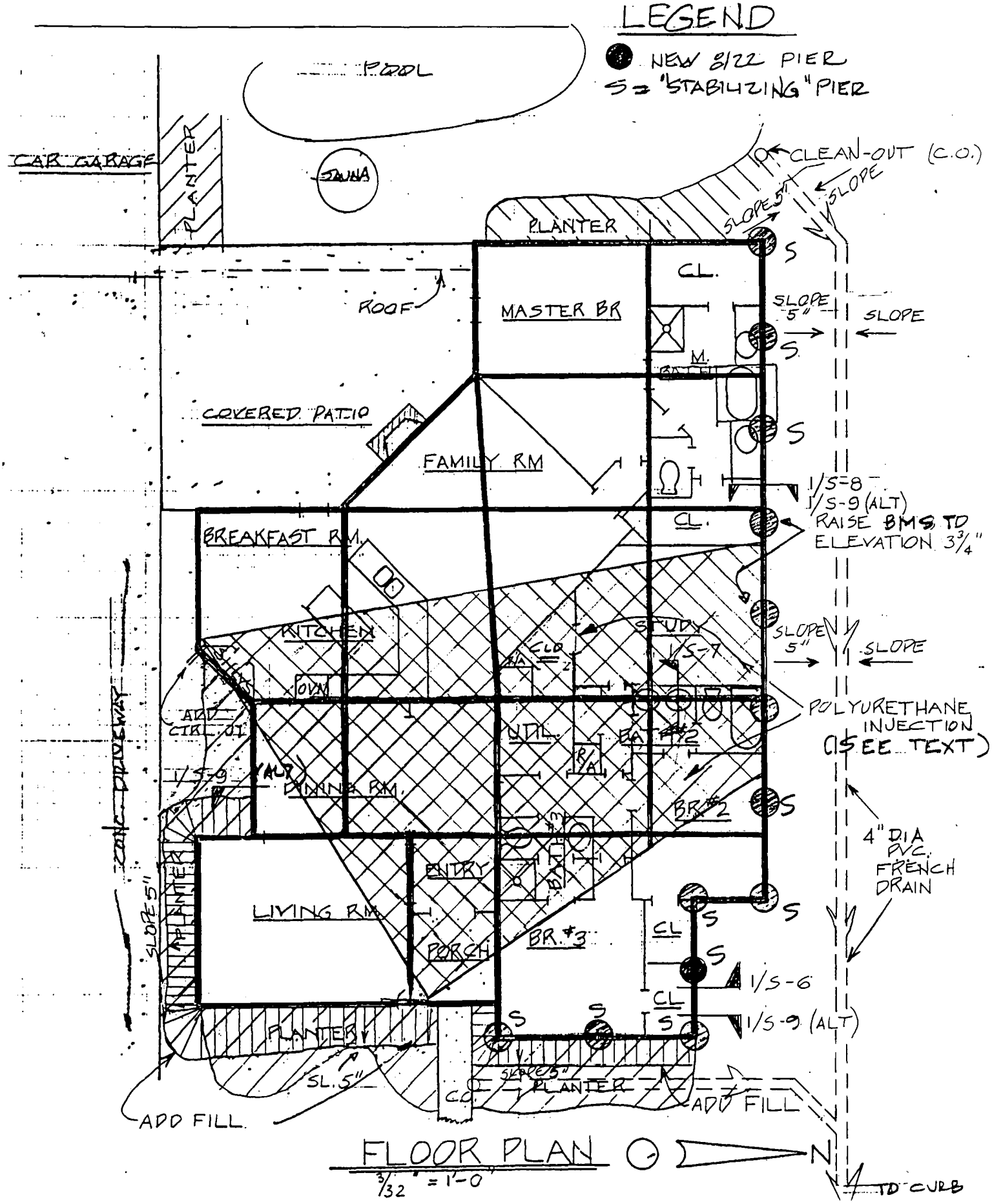
PHENOMENA PLAN



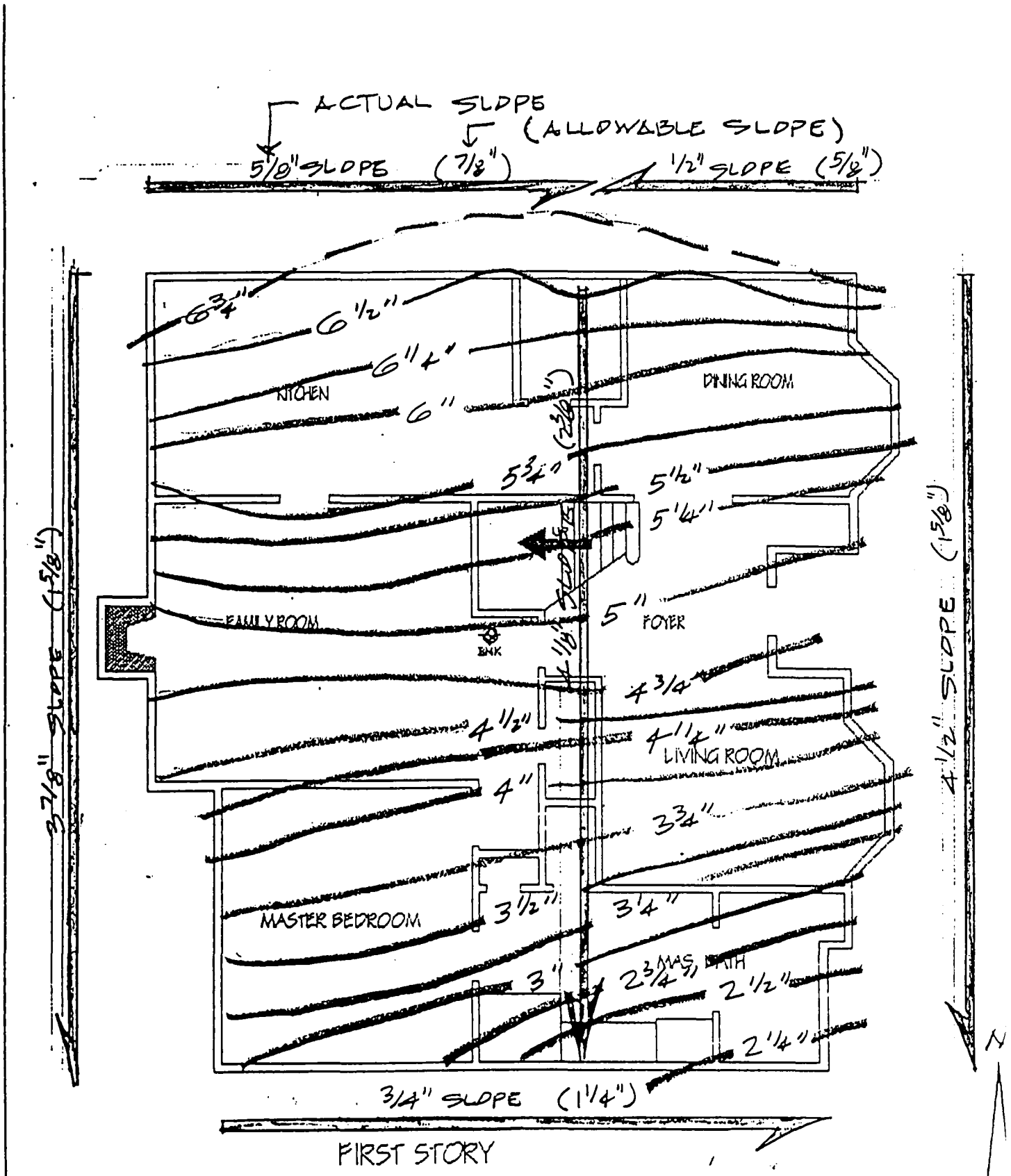
NOTES:
A) Concrete crack
B) Diagonal concrete (grade beam) crack
C) Vertical brick crack
D) Binding door
E) Diagonal sheetrock crack
F) Ceiling sheetrock crack
* Contradiction between observation & micro-elevation data.
G) Vertical sheetrock crack
H) Horizontal tile crack
J) Roof frieze board movement
K) Vertical trim separation
L) Old Brick Cracking
<—— Direction of downward slope

EXAMPLE OF EDGE LIFT REQUIRING
LEVELING PIERS AND POLYURETHANE INJECTION

REPAIR PLAN

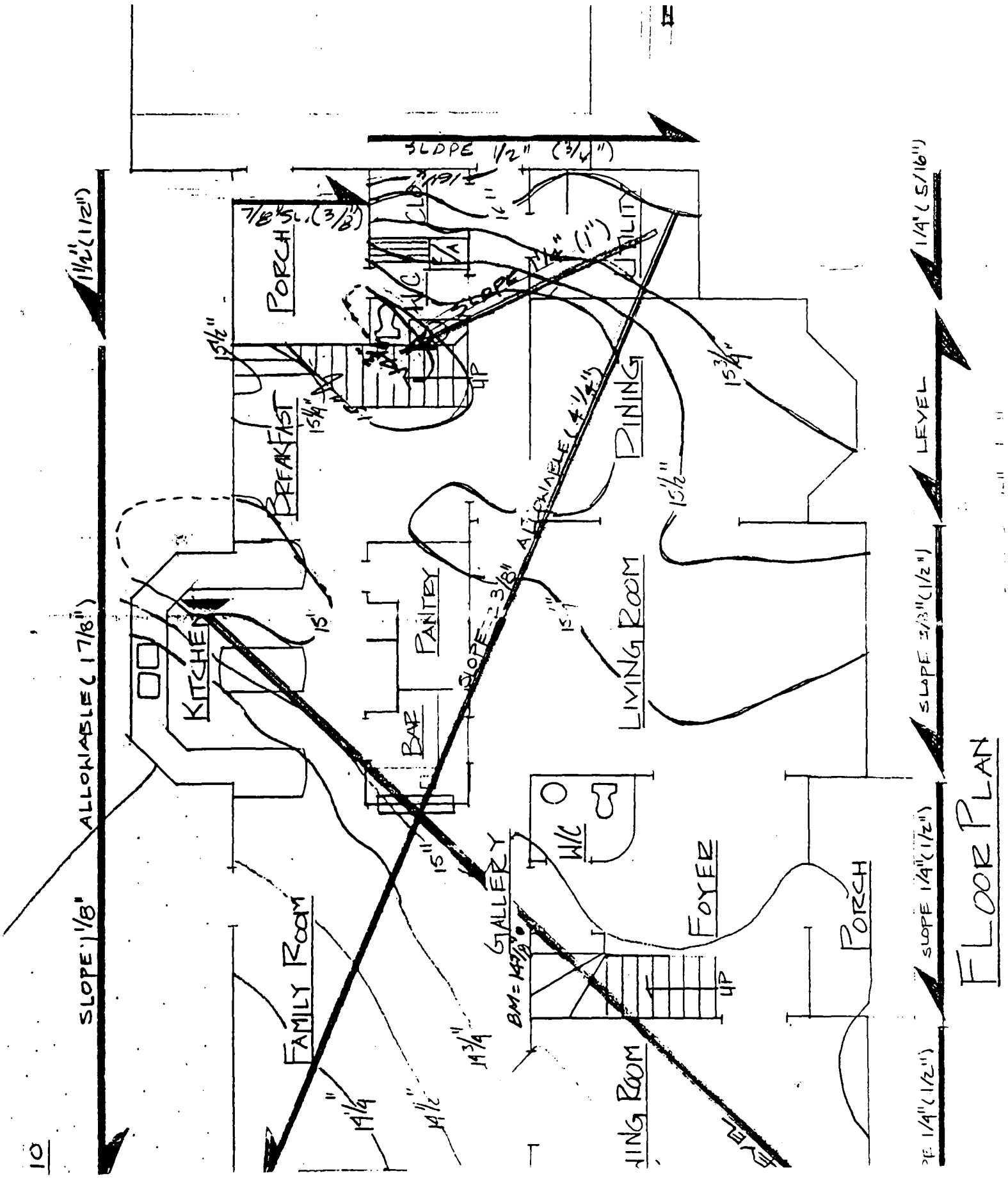


MICRO-ELEVATION CONTOUR PLAN



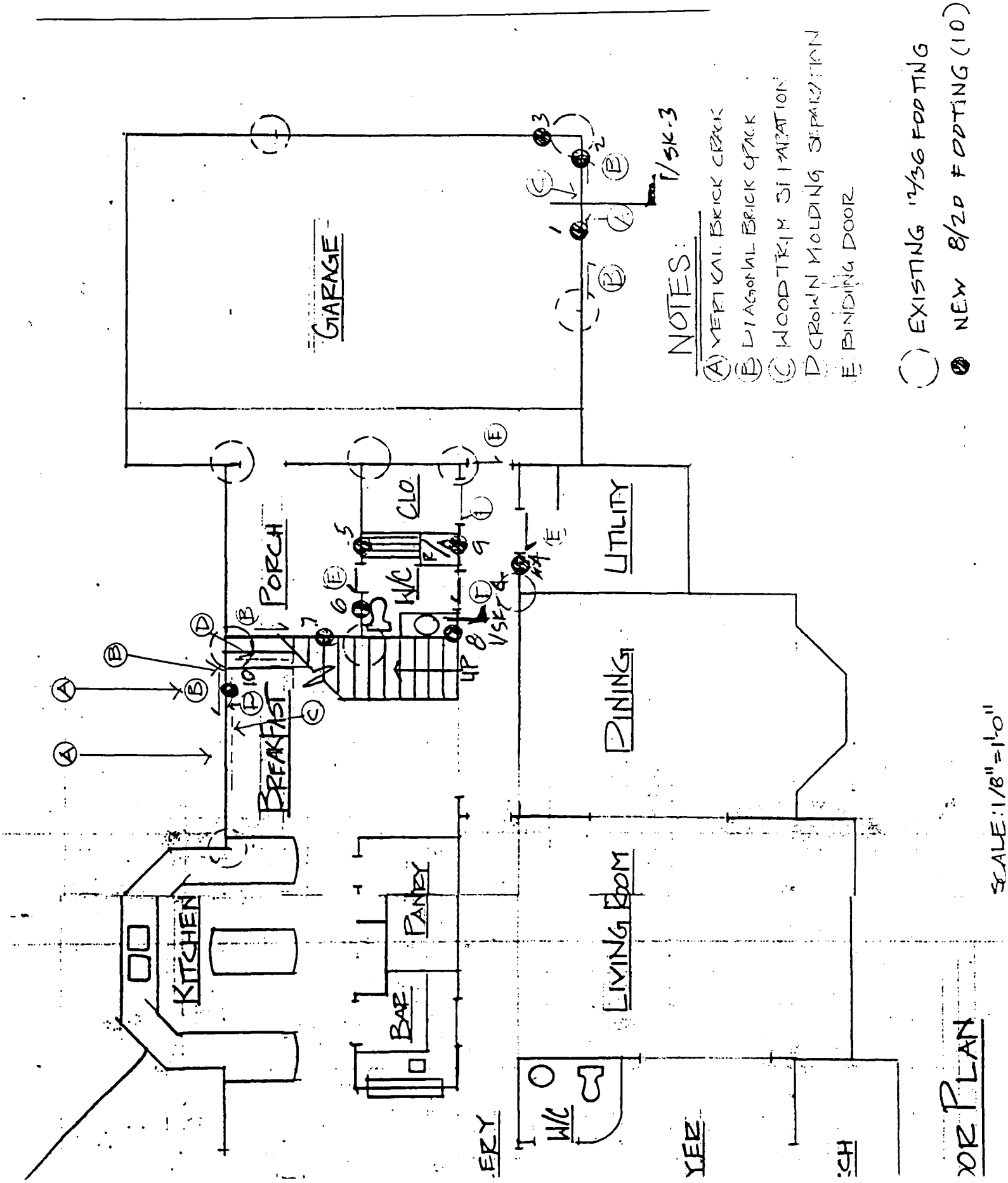
SCALE 1/8"=1'

MICRO-ELEVATION CONTOUR PLAN



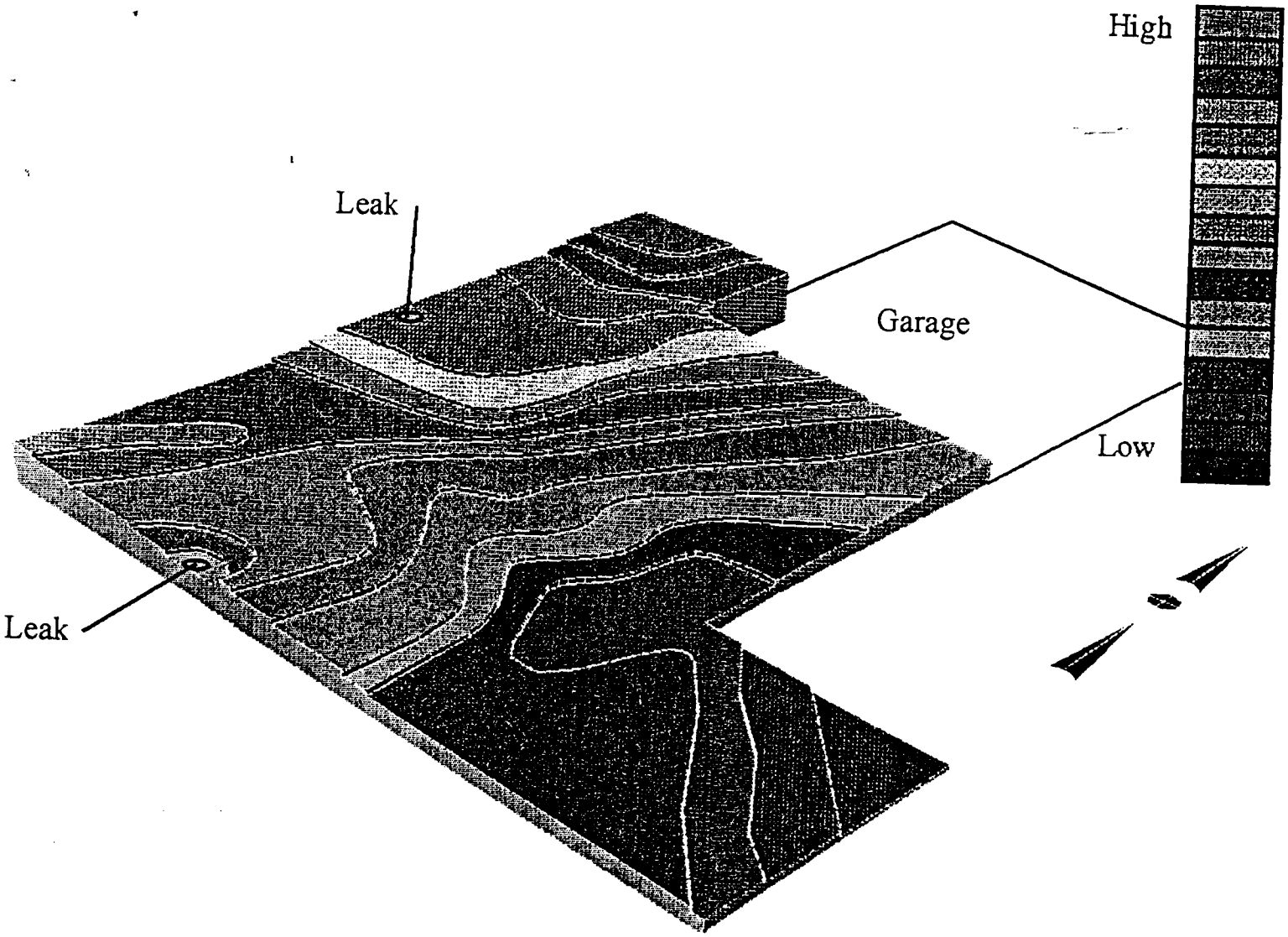
EXAMPLE OF LOCALIZED DRILLED FOOTING FAILURE

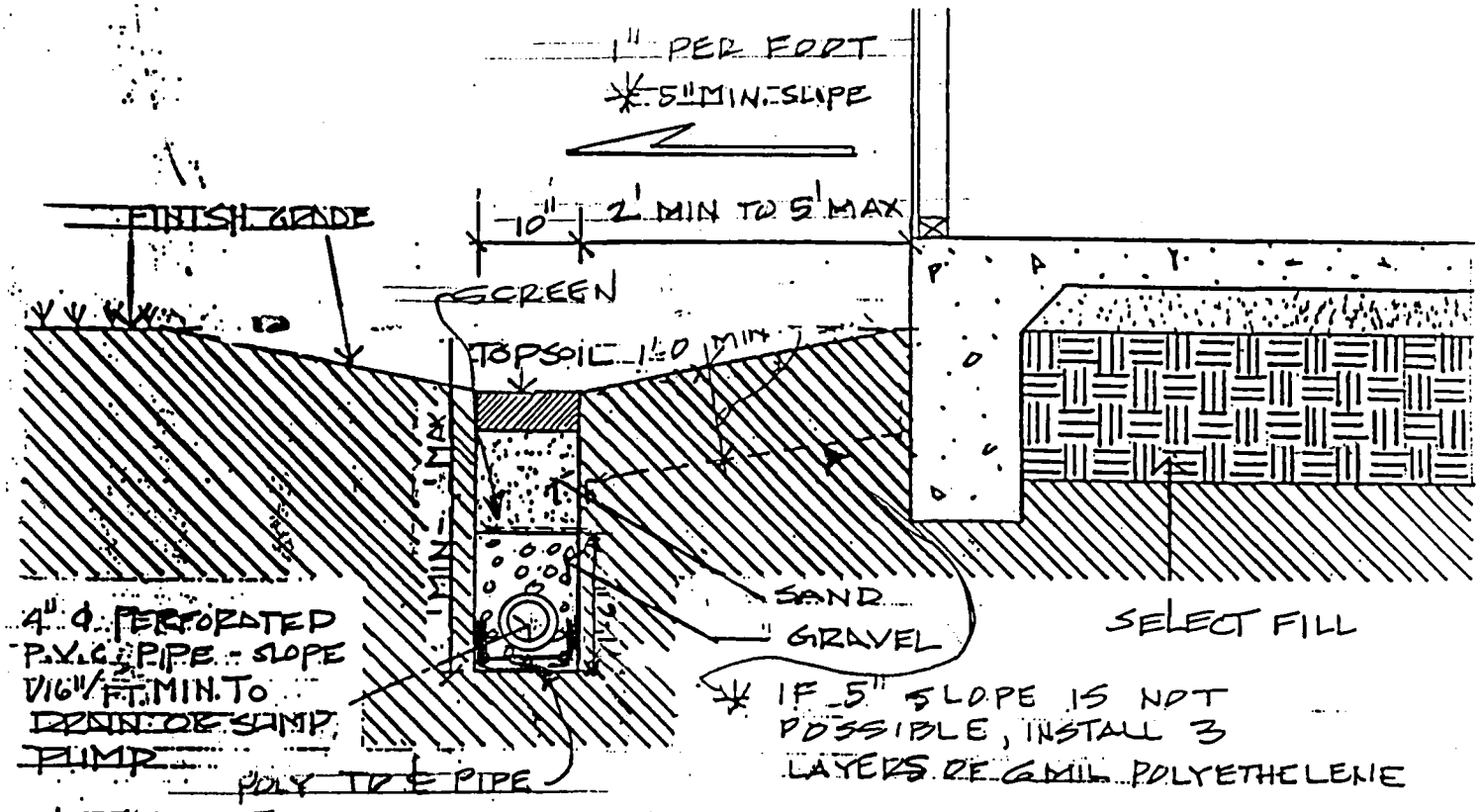
REPAIR PLAN



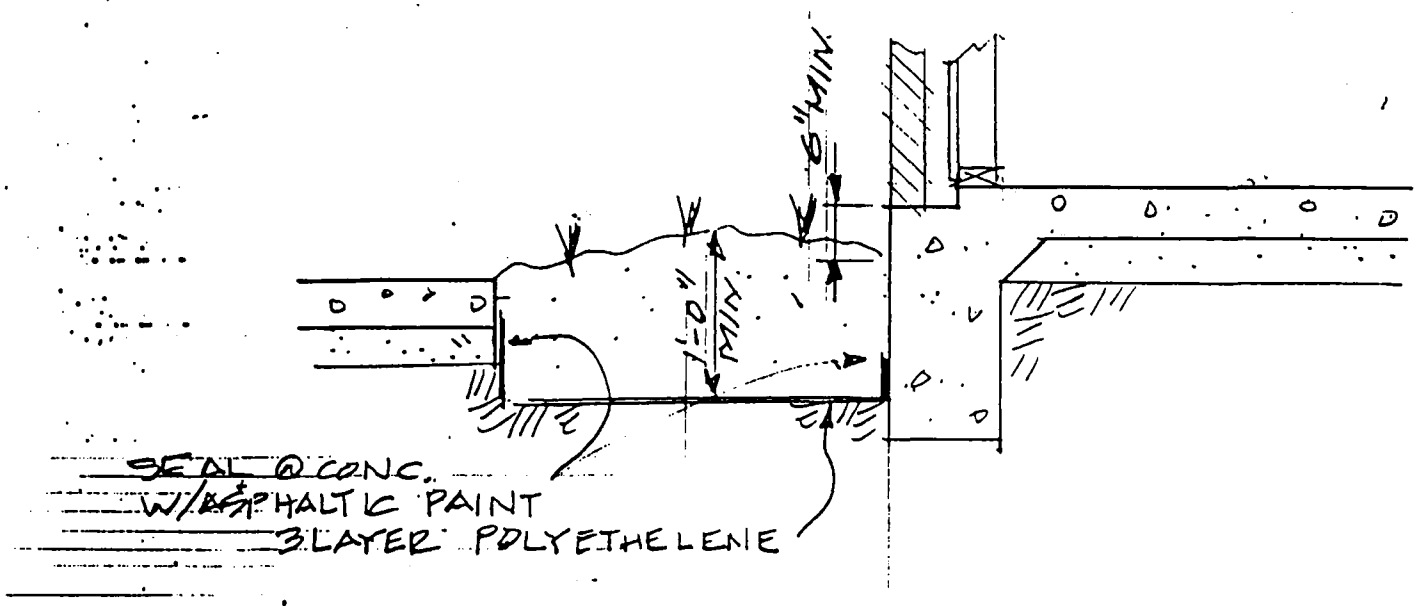
EXAMPLE OF PLUMBING LEAK - CLAIM DENIED SK-11

PERSPECTIVE VIEW

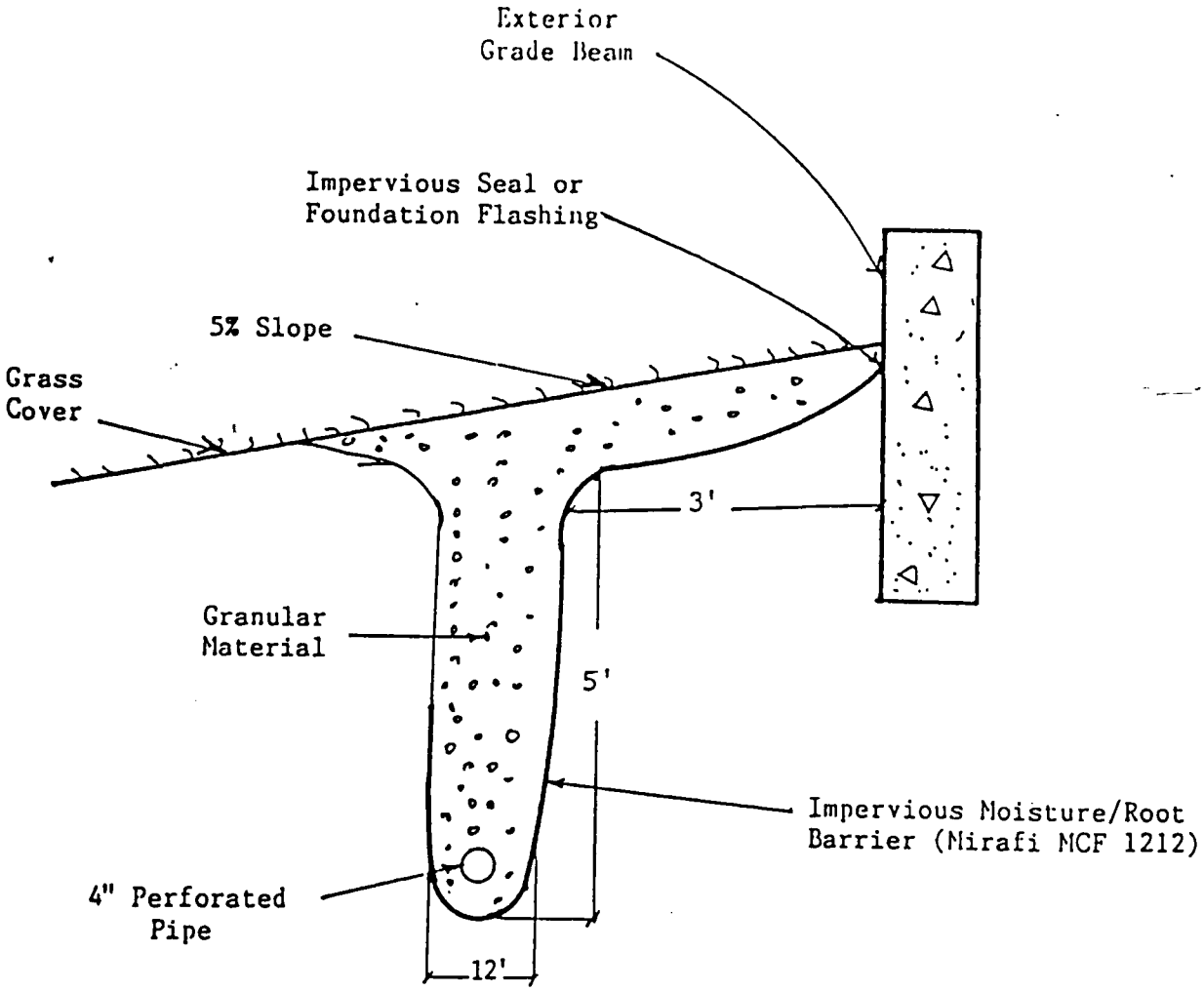




1/S-12 FRENCH DRAIN - PROPERLY INSTALLED



2/S-12 POLYETHELENE SEAL



VERTICAL AND HORIZONTAL
MOISTURE BARRIERS

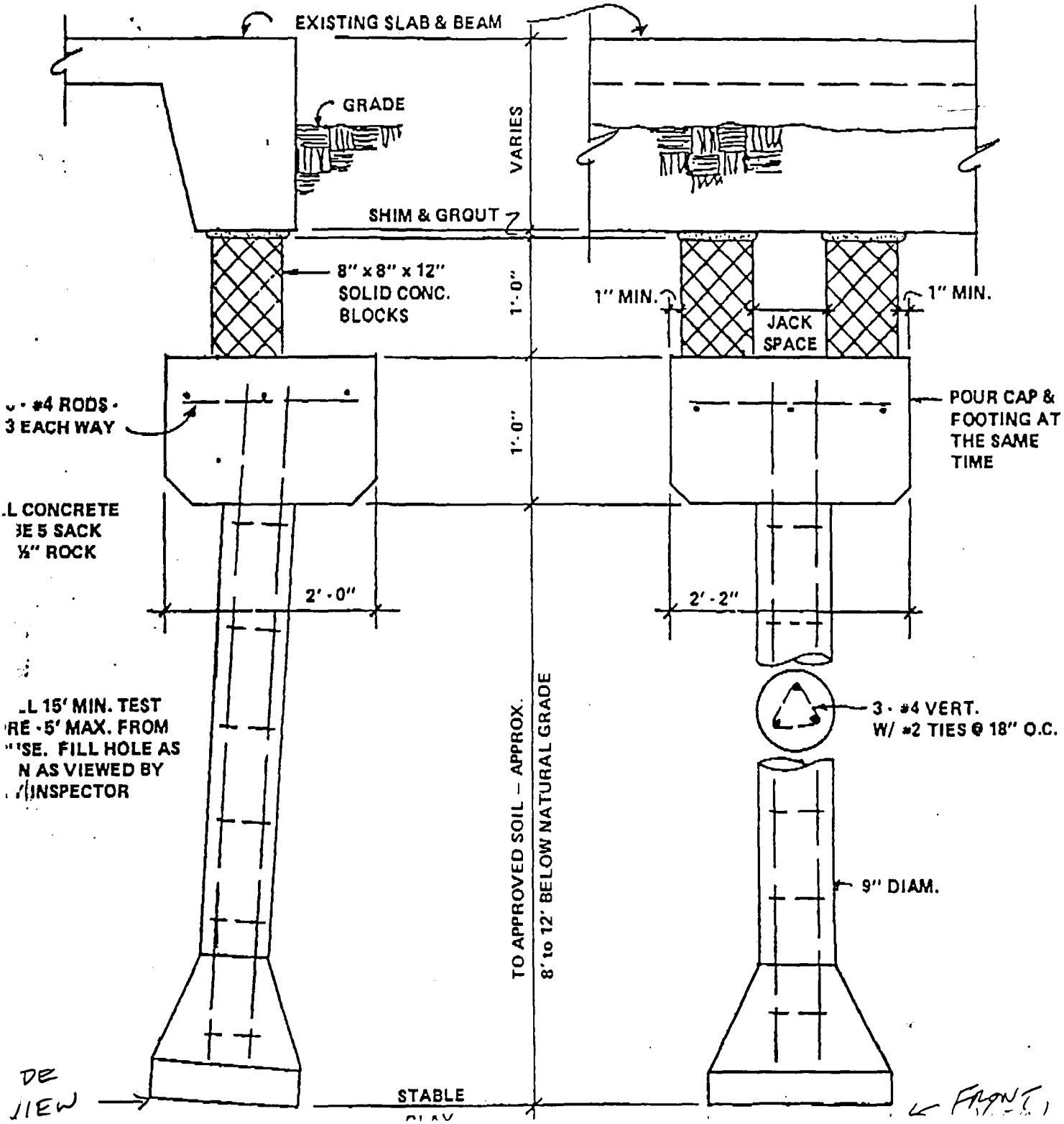
AAA

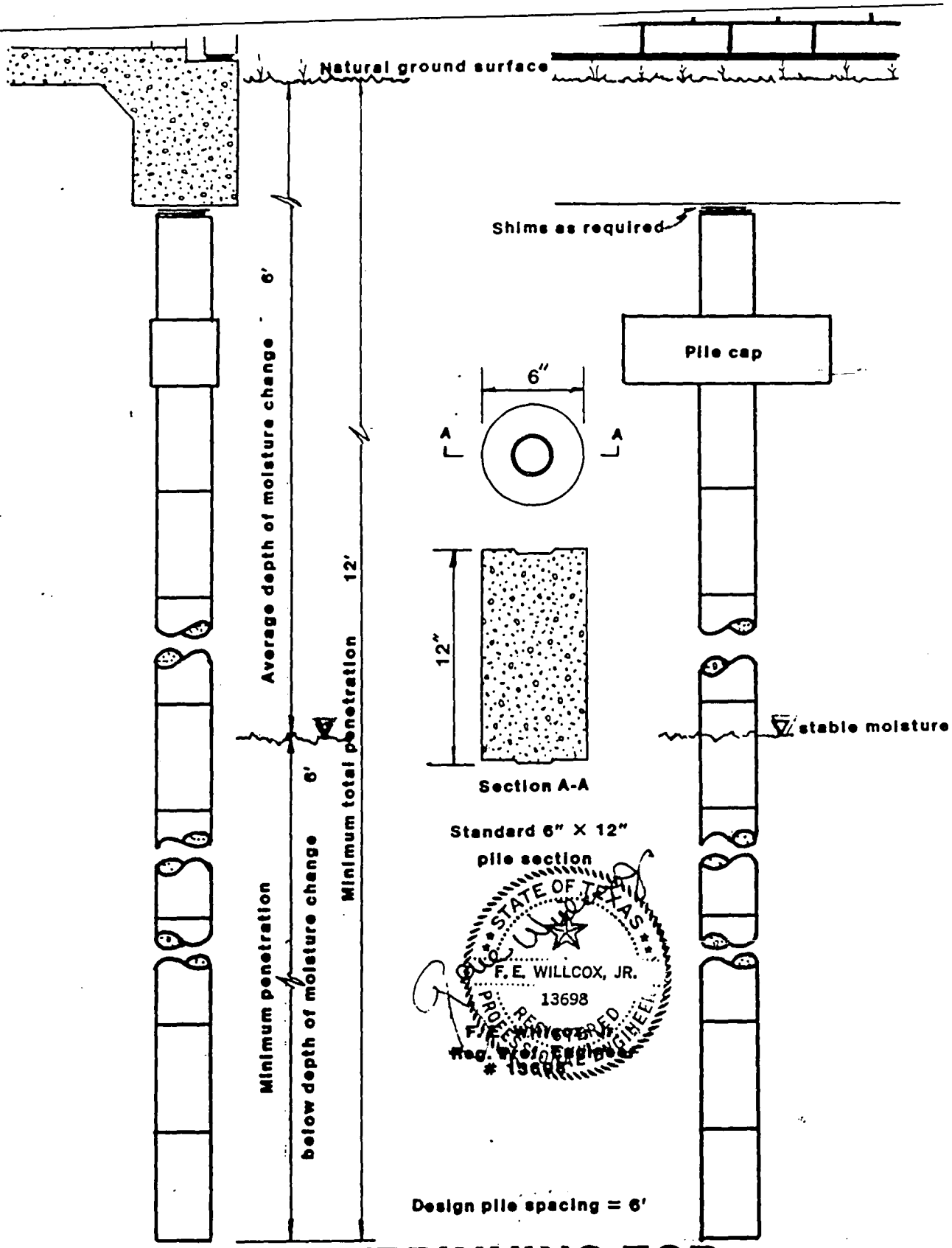
FOUNDATION SERVICE, INC.
HOUSTON, TEXAS

AAA FOUNDATION INC. SINCE 1955

PIER FOUNDATION DETAILS

FOR REPAIR ONLY TO DAMAGED FOUNDATIONS WHERE SPECIFICALLY DESIGNATED





UNDERPINNING FOR 1—STORY BRICK VENEER HOMES

Perma-Pile™ System for use on the Beaumont Formation, Texas

Std. 1—686

THE PermaJack® SYSTEM

④ How the Perma-Jack System works

An opening is made adjacent to the foundation and enough dirt is removed so that the Perma-Jack Bracket (A) (the core element of the system . . . see diagram at right) can be placed under the foundation. This bracket, a strong structural steel weldment weighing 44 pounds, is permanently installed under the foundation. Next, a pier formed from one or more pipe support columns (B) is placed through the bracket. These pipe support columns are aligned and tightly connected by a column connector (C).

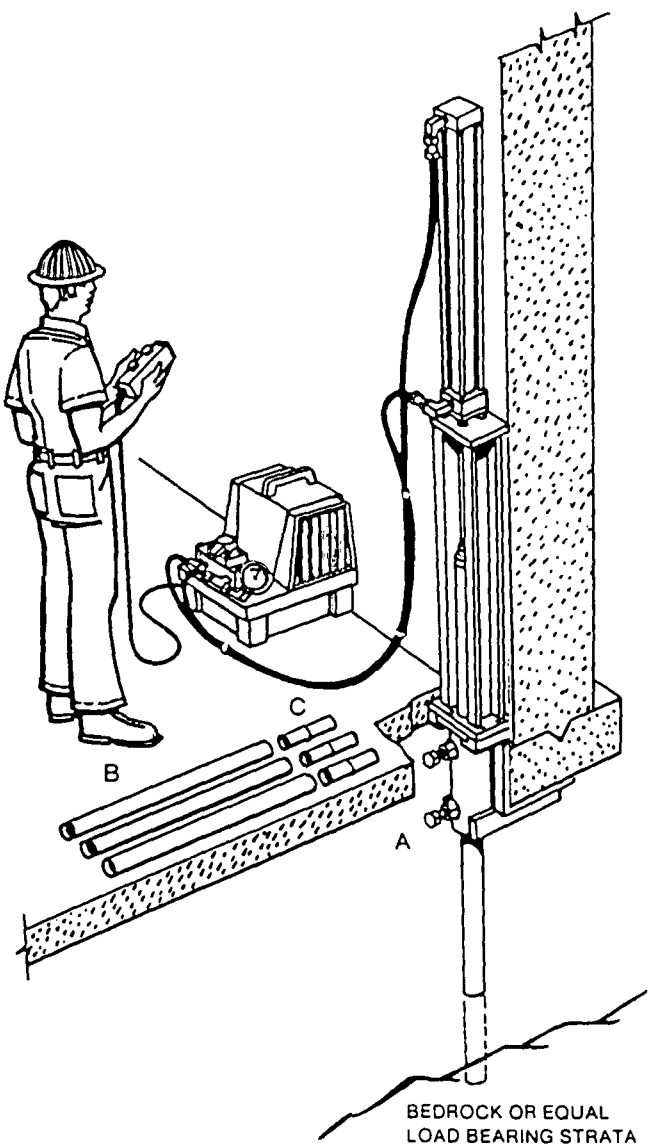
Hydraulic pressure is then applied to force the pier into the ground until it hits bedrock or equal load bearing strata. The hydraulic equipment that is used for this operation is capable of testing each pier at 24,000 LBS. of force—yet it operates quietly . . . at a maximum of 80 decibels.

Pipe support columns are connected and forced into the soil to form a sturdy, stable steel pier reaching as deep as is necessary to contact solid bedrock or equal load bearing strata. The competency of the bearing material is verified by the very nature of the tested installation. Steel piers are positioned wherever needed under the structure to effectively support the foundation.

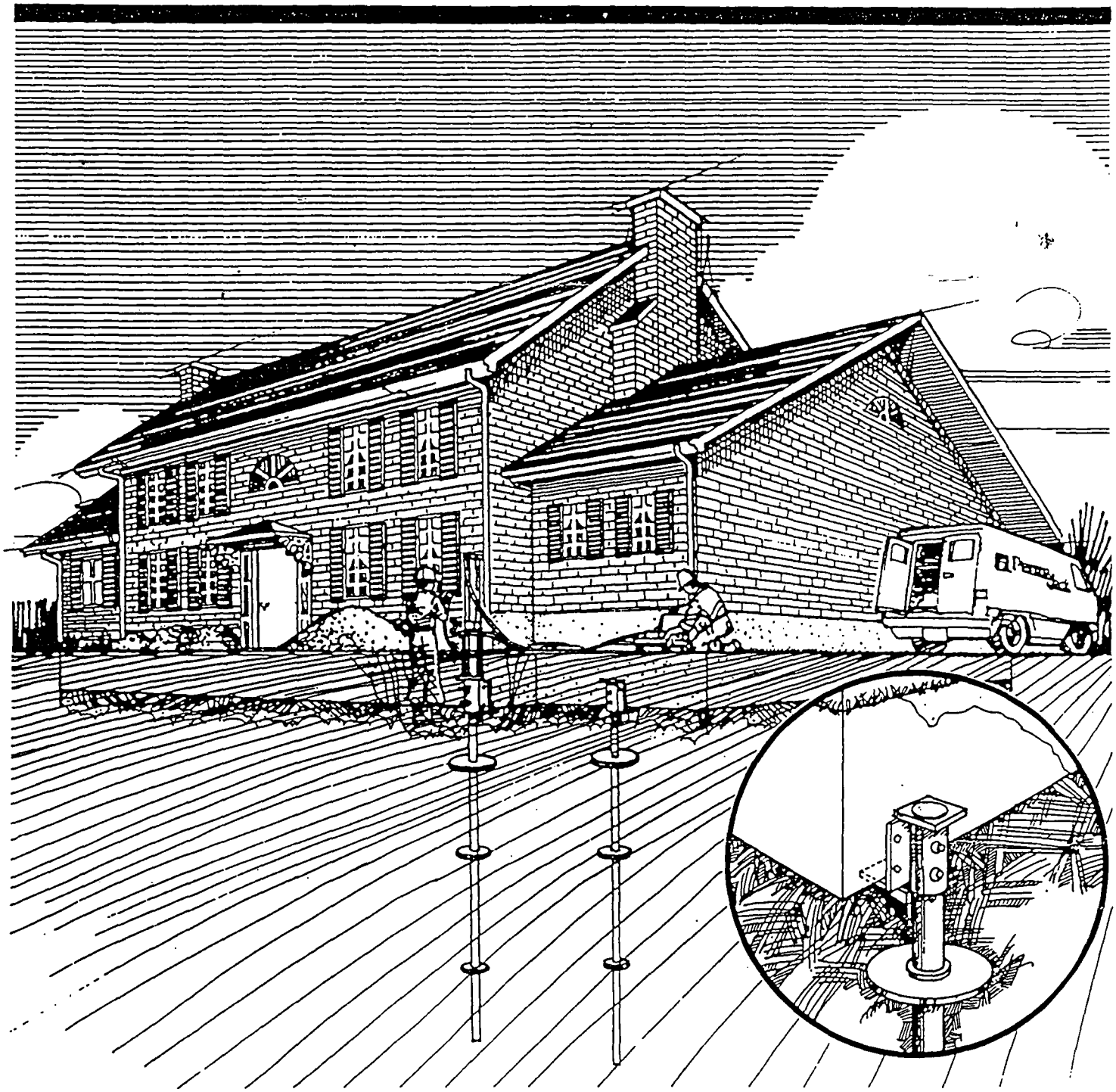
Why is Perma-Jack better?

The Perma-Jack System is a patented process that actually forces steel piers all the way down to bedrock or equal load bearing strata . . . so your house or commercial building is as solid as rock. Most other systems simply reinforce the foundation by using concrete as a spacer between it and the earth at a slightly lower level.

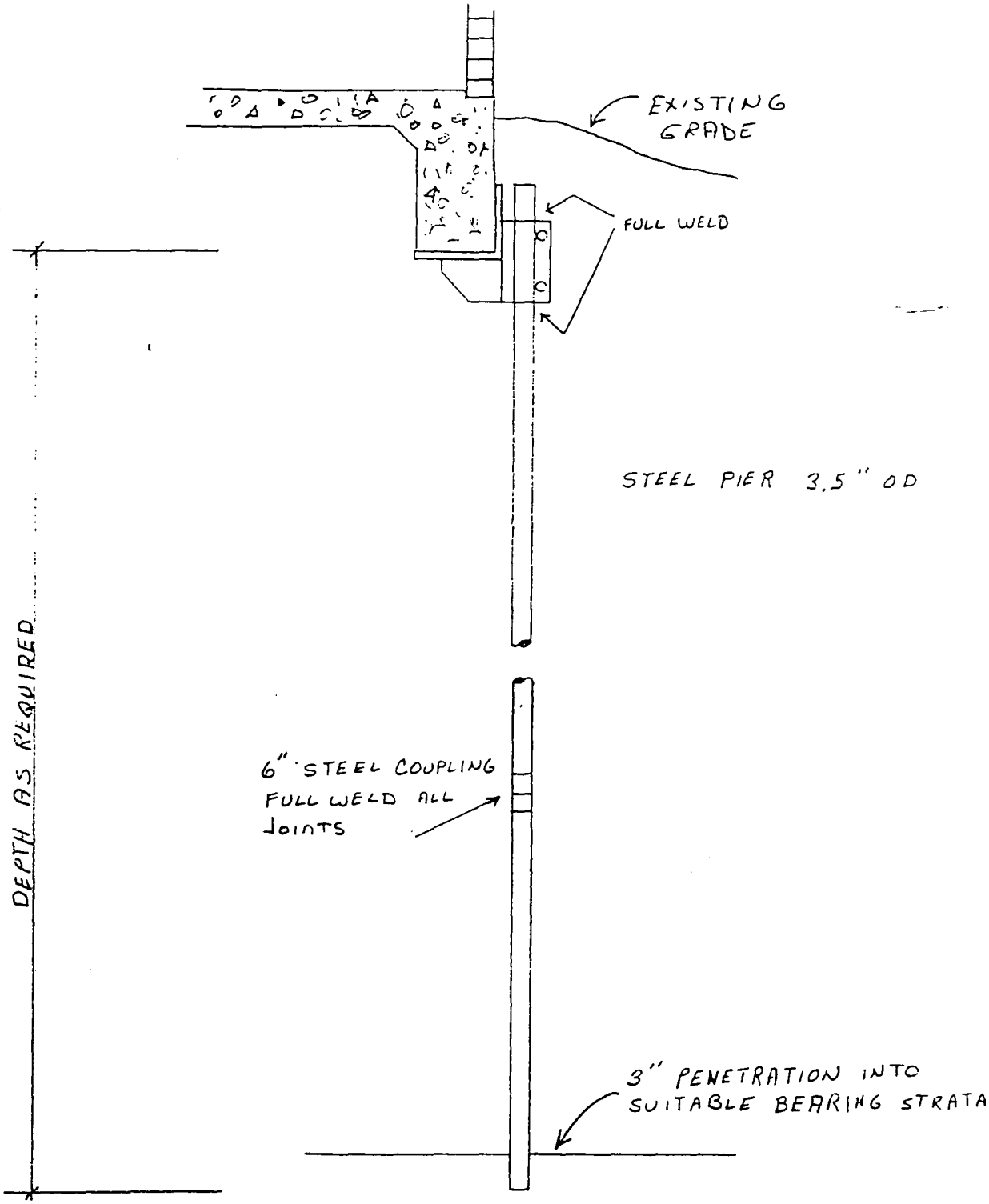
Concrete can shift and settle again. Perma-Jack's steel piers securely support your house or building to **rock or equal load bearing strata** . . . assuring that there will be **no** future vertical settlement of your foundation in the area where the Perma-



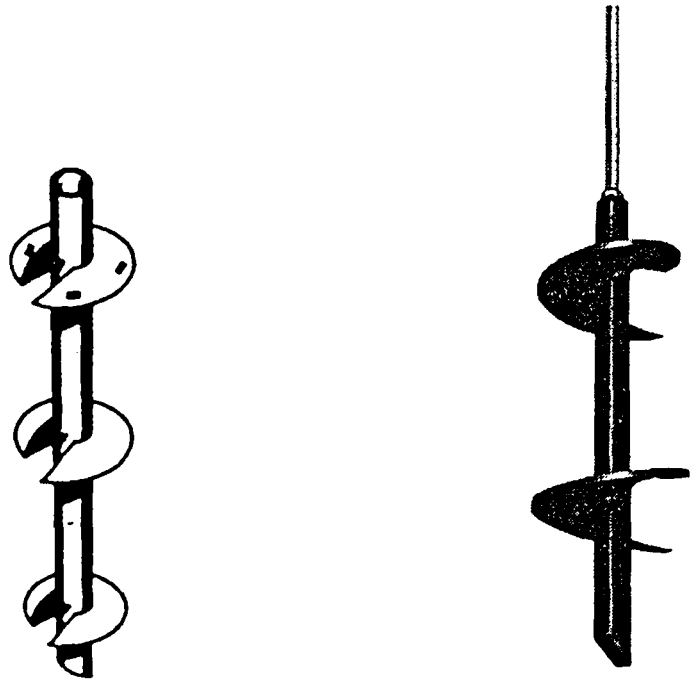
Perma-Jack System being installed



THE PermaJack® SYSTEM



STEEL PIER DETAIL



Pipe Type Shaft Square Shaft

Figure 1. Illustration of Foundation Anchor

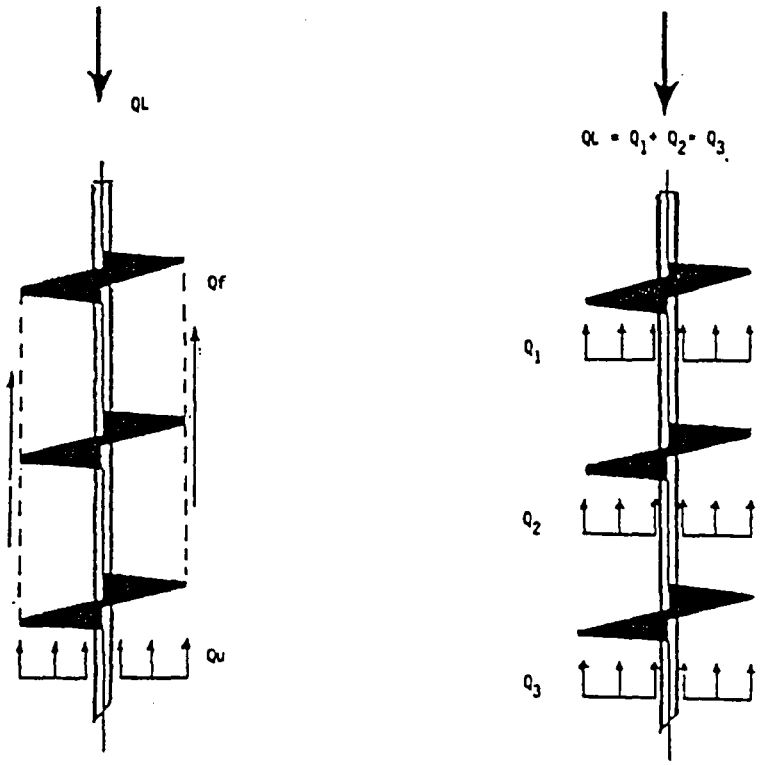


Illustration of Bearing & Cylindrical Shear Soil Reaction

Illustration of Bearing Soil Reaction

The URETEK Method®

URETEK USA, INC.

Dear Prospective Client:

We understand skepticism and recognize that one of the paramount duties of an engineer, when evaluating a product or repair technology is to gather pertinent and accurate data. We seek to provide that information so that a professional engineer seeking solutions can properly evaluate **The URETEK Method®** of lifting and undersealing concrete.

This booklet is not a sales brochure, but rather an assembly of data and documents for your careful review.

The Uretek service policy is simple. We may not be able to meet your requirements for every repair project, but if one of our representatives makes a commitment to you and our product or service does not perform according to the commitment, you will not be billed.

We are a service organization, dedicated to serving our clients with the highest degree of professionalism. If you favor us with an order, you will invite us back. That's our promise to you.

Thank you for your interest in **The URETEK Method®.**

Sincerely,

Brent Barron
President

DESCRIPTION OF URETEK METHOD

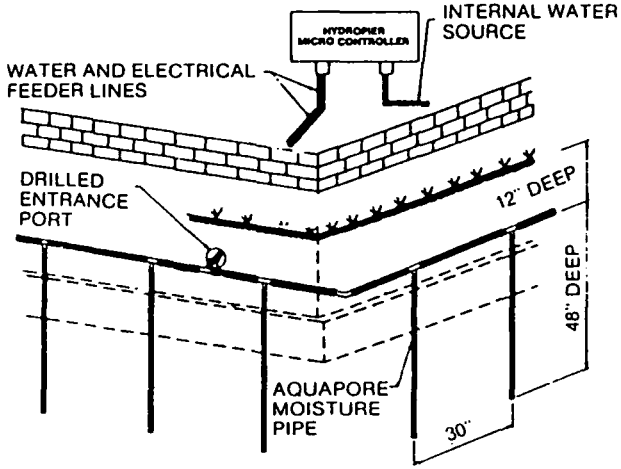
SK-20

9422 Hammerly • Houston, Texas 77080 • Tel: 713/973-0125 • Fax: 713/973-2168

SK-21

DESCRIPTION OF HYDROPIER METHOD
MODIFIED "LEAKY PIPE"

ENGINEERING SPECIFICATIONS



PATENTED HYDROPIER SYSTEM

The Problem: In many parts of the country, the content of the soil is greatly affected by the amount of moisture it can hold. Given too little rainfall, the soil can dry and crack . . . too much, and it can wash away. Soil is rarely "solid".

Generally speaking, our soil is a mixture of topsoil, sand, clay and rocks. The consistency of the soil (the material it is actually made of), determines just how well it retains moisture. Moisture-content determines how stable, or how "solid" the soil is.

During long, dry summers, the soil can shrink as much as 40% in some areas . . . enough to crack and break a home's foundation. As the soil loses its moisture and drys out, it shrinks away from the foundation, removing the support, allowing the weight of loadbearing walls to crack and break the foundation.

The Solution: HydroPier is the cost effective, long term solution to this problem. The key was an extensive research project conducted by the University of Texas Research Center. This 15-year study looked closely as the problems of residential foundation settlement. Based on the results of this study, they concluded that further development of a moisture-injection system was the most viable solution for the majority of the residential foundation problems.

The result of their work is the HydroPier System. HydroPier is a low pressure water-injection system that controls the moisture content of the soil around and under the foundation of a home, and stabilizes it, eliminating shrinkage and providing a firm, solid base for the structure.

How HydroPier Does It . . . The HydroPier System is a patented System that can repair existing problems and prevent future damage. The System utilizes a unique fabricated porous pipe. This pipe allows water to seep or "sweat" through the pipe's surface into the surrounding soil. A high-grade polyethylene pipe is placed in a small excavation around the perimeter of the house, below the frost line, and acts as a feeder line to the porous vertical pipes or piers.

The porous vertical piers are closed at the bottom end and are installed approximately 2 1 /2-feet apart. The vertical piers are then attached to the feeder line with PVC connectors designed for HydroPier.

On /off valves and check valves are then installed and the water source is attached. Water flow is controlled by a wall-mounted, fool-proof micro-controller (computer). This controller was tested and developed to state-of-the-art. It is the "brain" of the HydroPier System with water injected at intervals as determined by the results of soil analysis. The patented HydroPier System is now complete.

Consumer Care Program

RESIDENTIAL SOIL AND FOUNDATION REQUIREMENTS DEFINED BY THE CITY OF BELLAIRE

by

JOE EDWARDS
BUILDING OFFICIAL
CITY OF BELLAIRE

CITY OF
Bellaire
7008 South Rice Avenue • Bellaire, Texas 77401-4495 • (713) 662-8222 • Fax (713) 668-4211

RESIDENTIAL SOIL AND FOUNDATION REQUIRMENTS
GOVERNMENT AGENCY POINT OF VIEW

BY JOE EDWARDS
BUILDING OFFICIAL
CITY OF BELLAIRE, TEXAS
JULY 28, 1994

I. Listed is an overview of soil and foundation requirements from the following cities.

A. City of Bellaire
The soil engineer and structural engineer shall certify separately by letter to the City of Bellaire that inspections have been made and based upon the review of data, that the foundation as constructed and poured substantially conforms to the design and the intent of the soil exploration and foundation plan which has been submitted for a permit under other provisions of the Code of Ordinances of the City of Bellaire.

B. City of Missouri City
Soil reports are required. Structural Engineer to design foundation or use City's minimum requirements for one and two family dwellings.

C. City of West University
See attached.

D. City of Houston
See attached.

II. Foundation Repair

III. The Trend

CITY OF
Bellaire

7008 South Rice Avenue • Bellaire, Texas 77401-4495 • (713) 662-8222 • Fax (713) 668-4211

JOSEPH A. EDWARDS
CHIEF BUILDING OFFICIAL
CITY OF BELLAIRE, TEXAS
713-662-8230

Activities and Experience: Chief Building Official for the City of Bellaire, Texas for the past 19 years.

For the past 17 years I have taught Building Codes, Building Construction, and Housing Real Estate Inspections. This has been in conjunction with the Building Officials Association of Texas, Texas A&M Engineering Extension Services, and the Texas Association of Real Estate Inspectors. I have been a consultant to various cities pertaining to construction techniques, code interpretations and plan checking. Qualified as Building, Electrical, Plumbing, Mechanical, Housing and Health Inspector. Expertise in all types of construction including high-rise structures. Recognized as an expert in soil mechanics and cement and concrete technology. Have been associated with Building Codes, general construction and related areas for the past thirty-seven years.

Professional Association Memberships:

Past Vice President-Board of Directors-Greater Houston Builders Association
Outstanding Associate Member-1970-Greater Houston Builders Association
Past President-Building Officials Association of Texas
Founding President-Gulf Coast Association of Building Officials
Honorary Member-Texas Association of Real Estate Inspectors
International Conference of Building Officials
International Association of Plumbing and Mechanical Officials
Licensed Plumbing Inspector-State of Texas
Texas State Association of Plumbing Inspectors
International Association of Plumbing Inspectors
Construction Specifications Institute
Southern Building Code Congress International
Texas Public Health Association
Texas Environmental Health Association
Past Chairman - International Conference of Building Officials Technical Engineering Evaluation Committee. This committee deals with building codes, building systems and material usage across the United States and its possessions.
Member of ICBO ES Committe since 1985
Recognized and listed in Who's Who in Government in America.
Presently serving on ICBO Evaluation Service, Inc. Board of Directors
State of Texas Building Official of the Year - 1992

CITY OF HOUSTON
Building Inspection Division

CODE WORD 91

INTERPRETATIONS & APPLICATIONS OF THE UNIFORM CODES, '91 EDITION

PUBLICATION: August 20, 1992

SUBJECT: Policy - Minimum Required Foundation

CODE: Building

SECTION: 2907

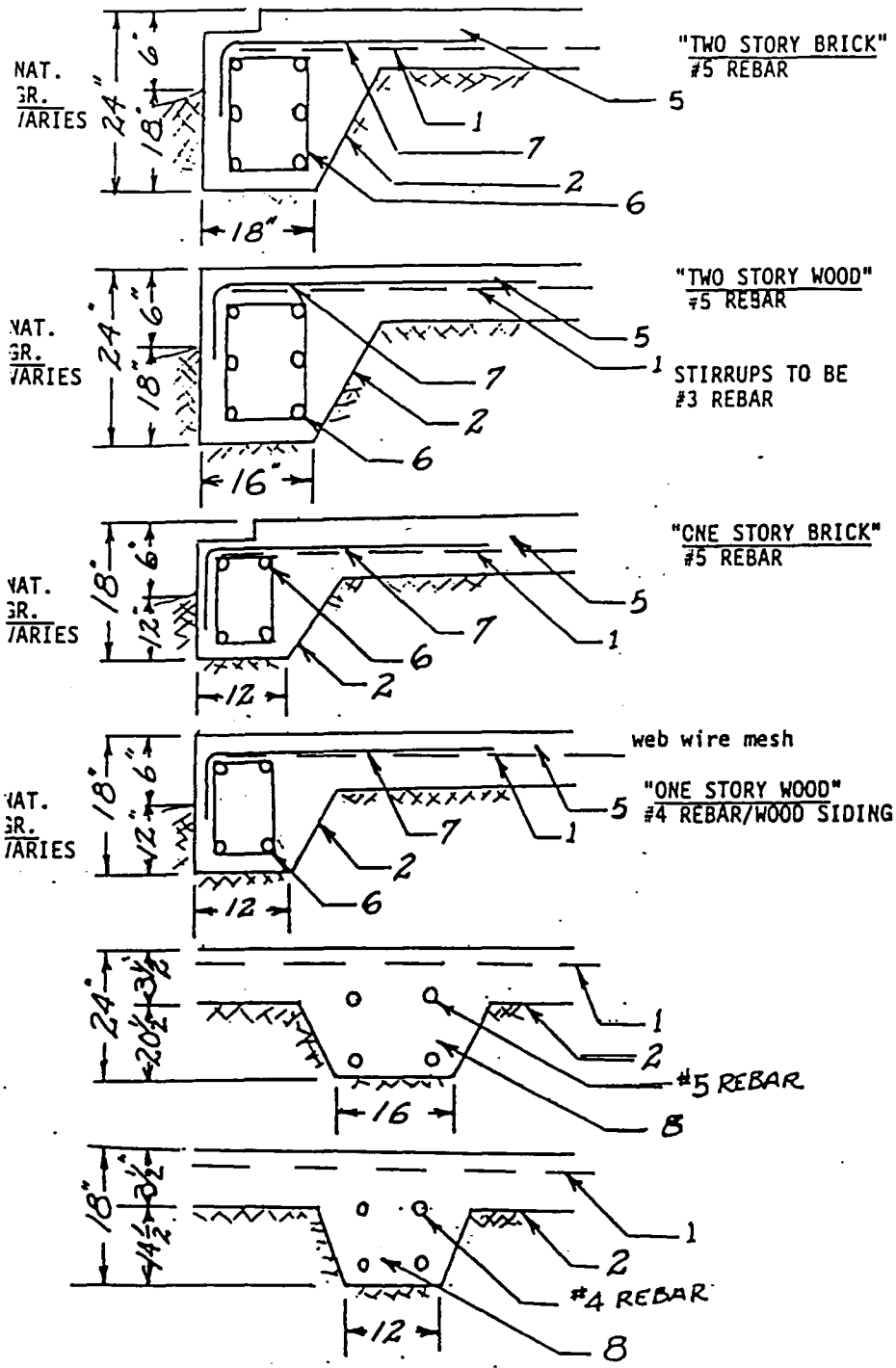
A foundation which 1) is classified as exempt by Section 20 (f) of the Texas Engineering Practice Act and 2) meets or exceeds the specifications contained on page 2 of this policy shall be considered to comply with the Houston Building Code. No engineer's seal and no soils report is required. Other designs, including post-tension designs, must bear the seal of a Texas registered engineer.

J. Hal Caton
Chief Building Official

CITY OF HOUSTON
Building Inspection Division

CODE WORD 91

INTERPRETATIONS & APPLICATIONS OF THE UNIFORM CODES, '91 EDITION



CONSTRUCTION NOTES

1. All slabs shall be reinforced. #6 web wire mesh 6" x 6" minimum.
2. All house slabs shall have a 6 mil vapor barrier using poly or approved material.
3. All slabs shall be a min. of 3½" thick with a 4" sand cushion.
4. Concrete shall have a minimum of 2000 PSF in 28 days.
5. Steel shall be covered with a 2" of concrete.
6. Stirrups to be #3 rebar.
7. Shear reinforcing at intersection of slab and all beams to be #4 rebar, 5' long, 5' on center.
8. Steel shall be covered with 3" of concrete.
9. Interior beams every 20 linear ft. or under bearing partition.

CITY OF HOUSTON
Building Inspection Division

CODE WORD 91

INTERPRETATIONS & APPLICATIONS OF THE UNIFORM CODES, '91 EDITION

AMENDED: August 19, 1992

INITIAL PUBLICATION: November 21, 1991

SUBJECT: Policy - Requirements for Classification of Soil

CODE: Building

SECTION: 2904 & 2905

This section required the classification of the soil at each building site to be determined by an engineer or architect licensed by the State or by an approved agency. The classification shall be based on observation and any necessary tests of the materials disclosed by borings or excavation made in appropriate locations. A written report of the investigation shall be submitted with construction drawings for a building permit.

EXCEPTIONS:

1. Buildings constructed with joists and sills supported on blocks and bases.

2. Group M1 and M2 occupancies that are not required to be designed by a professional engineer.

3. Buildings or additions not exceeding two stories and not exceeding 600 square feet of foundation area, provided the spacing of beams do not exceed 12 feet.

4. Repairs to foundations that are performed in accordance to policy issued November 8, 1985, by the Code Enforcement Division.

5. Foundation designed and sealed by a structural engineer licensed by the State. (Soil classification and design bearing capacity shall be noted on plans.)

CW NO: 91-51

6. Foundations for structures which are exempted from the State Engineering Practice Act by Section 20 (f) of that Act, provided the foundation meets the City of Houston's Minimum Required Foundation policy.

J. Hal Caton
Chief Building Official

CITY OF WEST UNIVERSITY

Section 6-52. Exceptions

The following exceptions are made from the Standard Building Code adopted by this article:

(1) All roofs must have a Class C or better fire resistance as determined in accordance with Section 706 of the Building Code.

(2) All foundations for new construction of buildings, and foundation repairs involving the installation of piers, shall be signed and inspected by a registered professional engineer and shall meet all of the following requirements:

(a) The foundation or foundation repair shall be illustrated in complete plans and specifications signed and sealed by the registered professional engineer.

(b) The foundation design for new construction of buildings shall be based upon a soils "SR" report prepared by a recognized and reputable soils investigation agency or firm.

EXCEPTION: Foundations for single story accessory structures containing less than four hundred fifty feet (450) square feet of gross floor area do not require a soils report.

(c) The foundation or foundation repair shall be inspected by the registered professional engineer and the engineer's report certifying the proper construction shall be submitted to the Building Official prior to additional work being done.

THE PLAINTIFF'S PERSPECTIVE

by

JAMES R. MORIARTY
MORIARTY & ASSOCIATES

JIM MORIARTY
JAMES R. MORIARTY & ASSOCIATES

The firm of James R. Moriarty & Associates specializes in non-class action, mass tort cases involving large numbers of individual plaintiffs. In the past few years, the firm has recovered several hundred million dollars for its clients in consumer fraud, construction, and securities litigations.

Moriarty & Associates, along with co-counsel, has represented the owners of more than 12,000 homes in 26 states damaged by polybutylene (PB) plumbing systems.

The Washington-based Trial Lawyers for Public Justice described the firms' success in the PB litigation as "fairly extraordinary. It is hard to overstate the enormity of the task that faces a small...firm attempting to take on a major corporate giant."

In 1992, the firm, in a joint-venture with Bristow Hackerman Wilson & Peterson, of Houston, recovered a $52 million settlement in one of the fastest out-of-court resolutions of a national securities fraud case in recent history. More than 5,800 clients throughout the United States were represented.

The firm's approach to mass tort litigation and its cases have been reported by *CBS Radio, The New York Times, the Wall Street Journal, Good Morning America, The Boston Globe, 60 Minutes, The Dallas Morning News, CNBC-TV, The Houston Chronicle, and The Houston Post,* among others.

A frequent lecturer at law conferences, Mr. Moriarty is an advocate of using computers as a means of improving efficiency in the legal industry. He has spoken throughout North America before various bar organizations and law schools.

The firm, featured in a Macintosh video introducing new technologies, also was profiled in the recent book, "Winning With Computers - Trial Practice in the 21st Century." Mr. Moriarty served as a voluntary member of the Houston Independent School District Technology Task Force, a coalition of business and academic leaders charged with recommending innovative applications of new technologies.

A decorated veteran, Mr. Moriarty served two tours of duty in Vietnam for the U.S. Marines. A resident of Houston, he is married with three children.

***

***James R. Moriarty**, born Schenectady N.Y., September 10, 1946; admitted to bar, 1976, Texas; U.S. Supreme Court, U.S. District Court, Southern District of Texas, U.S. Court of Appeals, Fifth and Eleventh Circuits. Education: University of Houston (B.B.A., 1973; J.D., 1981). Order of the Barristers. Judge, Hedwig Village, Texas, 1978-1979. Member: Texas Trial Lawyers Association; Association of Trial Lawyers of America. **Concentration:** Consumer Litigation.*

(5/94)

Soil Structure Interaction Seminar 1994

The Plaintiff's Perspective

by

James R. Moriarty

July 28, 1994

Deceptive Trade Practices Act
(DTPA)

I. Texas Business & Commerce Code §17.01 *et seq.*

A. General Provisions

1. Effective date of the act and amendments: The effective date of the act or practice giving rise to the cause of action under the DTPA determines which version of the DTPA applies. Woods v. Littleton, 554 S.W.2d 662 (Tex. 1977). i.e. pre 1979 version of act calls for mandatory trebling, whereas post 1979 version call for discretionary trebling upon finding of knowingly.

2. Cumulative remedies: Remedies under DTPA are cumulative, not exclusive. Consumers may avail themselves of common law or other statutory remedies in addition to the remedies available under the DTPA. Berry Property Management Co. Inc. v. Bliskey, 850 S.W.2d 644.

3. Construction of the act: §17.44 provides the act shall be liberally construed and applied to promote its underlying purpose; to protect consumers against false, misleading and deceptive acts and practices, unconscionable acts and breaches of warranty.

4. Who can sue? §17.50(a) Consumers-defined as an individual, partnership, corporation, this state, or a subdivision or agency of this state who *seeks or acquires by purchase or lease*, any goods or services..." Note, consumer need **not** have sought or acquired the goods or services from the defendants; i.e., no requirement of privity. Cameron v. Terrell & Garrett, Inc., 618 S.W.2d 535 (Tex. 1981), Flenniken v. Longview Bank & Trust Co., 661 S.W.2d 705 (Tex. 1983).

5. Pre-suit notice: Consumer must give 60 days written notice to the person against whom the consumer is filing suit. Failure to give notice, results in abatement, not dismissal. Hines v. Hash, 843 S.W.2d 464 (Tex. 1992).

6. Who may be sued? Any "person" who commits a violation of DTPA or Art. 21.21 of Insurance Code. "Person" is defined as an individual, partnership,

corporation, association, or other group, however organized. Only exception is advertising media.

7. Waiver of DTPA: General rule is waivers are void. Two exceptions: (1) business consumers with assets of $5 million or more can contractually waive the DTPA; (2) consumers who are not in a significantly disparate bargaining position who are represented by counsel on transactions other than home purchases on deals greater than $500,000 when both consumer and attorney sign written waiver.

B. Prohibited conduct: False, Misleading or Deceptive Acts

1. Laundry list: Of the 23 listed items, the most frequently used are items (5) &(7) which deal with representations about the quality of goods and services and (23) failing to disclose know facts.

2. Breach of warranties

a) Express: Any representation of fact or promise as to the title, condition or quality of goods or services will constitute an express warranty. For instance, a representations that a product is in "good working order" or "top quality" constitute an express warranty. Woods v. Littleton 554 S.W.2d 662 (Tex.1977), Chrysler Plymouth City v. Guerrero, 620 S.W.2d 700 (Tex.Civ.App.-San Antonio, 1981, no writ).

b) Implied

i) Merchantability: New goods are fit for their intended use. Does not apply to used goods, but has been extended to the sale of used homes. Thornton Homes, Inc. v. Greiner, 619 S.W.2d 8 (Tex.Civ.App.-Eastland 1981, no writ).

ii) Good workmanship: New homes must be constructed in a good and workmanlike manner. Evans v. J. Stiles, Inc., 689 S.W.2d 399 (Tex. 1985)

iii) Good and workmanlike repair: Services involving the repair or modification of existing tangible goods or property. Melody Home Mfg. v. Barnes, 741 S.W.2d 349 (Tex. 1987)

iv) Good and workmanlike development: Developer has implied duty to develop in a good and workmanlike manner. Luker v. Arnold, 843 S.W.2d 108 (Tex.App.-Fort Worth 1993, no writ)

c) Disclaimers: Must be conspicuous. "As is" effectively disclaims all implied warranties.

3. Unconscionable acts: Taking advantage of a consumer to a grossly unfair degree; it need not occur at time of sale and consumer does not have to show the defendant acted with intent, knowledge or conscious indifference. In one case, court held failure to install patio using good and workmanlike manner deemed unconscionable. Thrall v. Renno, 695 S.W. 2d 84 (Tex. App.-San Antonio 1985, writ ref'd n.r.e.)

C. Producing cause: A consumer can only maintain an action if the false, misleading or deceptive act or practice is the producing cause of actual damages. This is a lesser standard than proximate cause and has been liberally interpreted to mean any conduct by the defendant which factually causes any of the consumer's harm is sufficient to establish producing cause.

D. Damages

1. Recovery of actual damages: Defined as those damages recoverable at common law. Consumer can recover the greatest amount of actual damages he alleges and proves. Farrell v. Hunt, 714 S.W.2d 298, 300 (Tex.1986).

2. Elements/Measures of damages available

a) Direct losses: all pecuniary losses caused by the wrongful conduct can be recovered.

Examples

i) Out of pocket: difference between what the consumer gave versus what he got in return.

ii) Lost benefit of the bargain: difference between what was promised and what was actually received.

iii) Amount of consideration paid: upon recission, the amount of consideration paid for the goods or services.

iv) Cost of repair

b) Consequential losses

i) Lost profits-typically used in breach of contract cases.

ii) Interest-when a consumer purchases goods on credit and the purchase is induced by a deceptive trade practice, the finance charges which the consumer becomes bound to pay can be recovered.

iii) Expenses-any expense directly or indirectly caused by the defendant's misconduct is recoverable; i.e., car rental expenses.

c) Personal injury/mental anguish: mere worry, anxiety, vexation or anger are not enough; although, humiliation, embarrassment, indignity, grief, severe disappointment, wounded pride, shame and/or despair are recoverable.

d) Treble damages: Upon finding of knowingly, court can award up to two times the amount of actual damages that exceed $1,000 while first $1,000 is doubled automatically; net effect is a trebling of damages.

e) Multiple recoveries: Consumer can recover both DTPA trebled damages and common-law punitive damages when separate findings under different theories of recovery are obtained.

3. Attorney's fees

Residential Construction Liability Act
(RCLA)

II. Residential Construction Liability Act (RCLA) -- Texas Property Code § 27.001 *et seq.*

A. Applicability §27.002

1. Applies only to residential construction.

2. Applies to "construction defects" -- but does not apply to actions for damages for personal injury (excluding mental anguish), survival, wrongful death or for damage to goods.

3. "Construction defect": any matter concerning the design, construction or repair of a new residence, or the remodeling of an existing residence. Includes any "appurtenances" to a residence i.e., swimming pool, detached garage, etc.

B. Who is a proper Defendant? "Contractors" §27.003

a) Defined as: a person contracting with an owner for the construction or sale of a new residence constructed by that person or of an alteration or addition to an existing residence, repair of a new residence, or construction, sale, alteration, addition, or repair of an appurtenance to a new or existing residence.

b) The definition also includes a risk retention group that insures any part of a contractor's liability for the cost of repairing residential construction defects.

C. Who is a proper Plaintiff? §27.001 et seq

Anyone who suffers damages from a construction defect, i.e., anyone who seeks or acquires a contractor's services to design, build or repair a new home or to remodel or add to an existing home. A subsequent purchaser of the home is required to follow the RCLA procedures.

D. Proceeding under the act -- § 27.004

1. Homeowner provides written notice of the construction defect in *reasonable detail,* by certified mail, return receipt requested at least 60 days before filing suit.

2. Inspection: contractor, upon written request, must be given opportunity to inspect within 35 days after receiving the notice.

3. Offer: contractor may make written offer to repair or pay money within 45 days after receiving notice.

4. If homeowner accepts offer, repairs must be completed within 45 days (unless delayed by the claimant or events beyond the contractor's control).

5. If claimant unreasonably rejects offer or doesn't permit contractor reasonable opportunity to repair, claimant may not recover an amount in excess of the reasonable cost of repairs which are necessary to cure the construction defect and may only recover the amount of reasonable and necessary attorney's fees and costs incurred before the offer was rejected.

6. If a contractor fails to make a reasonable offer, or fails to make a reasonable attempt to complete the repairs, or fails to complete the repairs in a good and workmanlike manner, the limitations on damages and defenses to liability do not apply.

7. If procedures followed, claimant may only recover the following damages

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

   b) reasonable expenses of temporary housing necessitated by the repairs;

   c) reduction in market value of the residence, if any, due to structural failure; and

   d) reasonable and necessary attorneys' fees.

8. Damages may not exceed the claimant's purchase price for the residence.

E. Defenses to Liability §27.003 & §27.004

1. Normal wear, tear, and deterioration.

2. Normal shrinking due to the drying or settlement of construction components within the tolerance of building standards.

3. Damages not *proximately caused* by the alleged construction defect.

4. Damages proportionally reduced by percentage due to failure by anyone other than contractor-- including the Owner-- to take reasonable action to maintain the residence.

5. Contractor reasonably relied upon written government information that was false or inaccurate.

6. Unlike the DTPA, all common law defenses apply to RCLA claims.

7. Abatement, if no or insufficient notice given

TEXAS DECEPTIVE TRADE PRACTICES ACT AND FOUNDATION CASES: THE DEFENDANT'S PERSPECTIVE
OR
ARE FOUNDATION CASES ALL THEY ARE CRACKED UP TO BE?

by

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June 24, 1994

David A. Eastwood
5889 W. 34th Street
Houston TX 77092

Dear David:

Here is the udpated DTPA article you requested. Please note the recent case of Parkway v. Woodruff and its impact on developer liability, as well as the change in expert testimony admissibility brought about by the U.S. Supreme Court case Daukert v. Merrell Dow Pharmaceuticals.

If you have any questions or need anything further please do not hesitate to contact me.

Sincerely,

Daniel F. Shank

DFS:ce

Enclosure

dfs:\eastwood.ltr

The Texas Deceptive Trade Practices Act and Foundation Cases: The Defendant's Perspective or Are Foundation Cases All They Are Cracked Up to Be?

Presented by Daniel F. Shank
Davis & Shank, P.C.
1415 Louisiana, Suite 4200
Houston, Texas 77002
(713) 659-1010

I. Who Steals My Purse Steals Trash

IAGO: Good name in man and woman, dear my lord,
Is the immediate jewel of their souls.
Who steals my purse steals trash; 'tis something, nothing;
But he that filches from me my good name
Robs me of that which not enriches him,
And makes me poor indeed.

WILLIAM SHAKESPEARE, OTHELLO act 3, sc. 3.

II. Texas Deceptive Trade Practices-Consumer Protection Act (DTPA) -- Texas Business & Commerce Code §§ 17.41 *et seq.* (Vernon 1987 & Supp. 1994).

III.

A. Who Can Sue? Plaintiff must be a "consumer"

1. Consumer:

a. Must "seek *or* acquire"

b. "goods *or* services"

(1) DTPA covers mixed purchases of goods and services, such as construction of a house. *Norwood Builders, Inc. v. Toler*, 609 S.W.2d 861 (Tex. Civ. App.--Houston [14th Dist.] 1980, writ ref'd n.r.e.) (holding that a contract for the construction of a new home is a "sale of goods").

c. by purchase *or* lease.

(1) Purchase need not be consummated if Plaintiff in good faith sought to complete it and had the capability of completing it. *Anderson v. Havins*, 595 S.W.2d 147 (Tex. Civ. App.--Amarillo 1980, writ dism'd) (finding that the DTPA applies, even though the Plaintiff did not complete the purchase of real property).

(2) Person who seeks or acquires the goods or services does not have to be same as person who pays for them. (Subsequent purchasers may qualify as consumers).

2. "Consumer" does *not* include a business consumer with assets of more than $25 million or one that is controlled by a corporation or entity with assets of $25 million or more. DTPA § 17.45(4).

a. "Business consumer" defined: Individual, partnership or corporation that seeks or acquires, by purchase or lease, any goods or services *for commercial or business use*.

B. Who May Be Sued? Privity not required.

1. Coverage of DTPA extends to *any* deceptive practice made in connection with the purchase of goods or services. The courts have not required privity, instead allowing as Defendants all those "inextricably intertwined" in the sale or lease transaction. *Flenniken v. Longview Bank & Trust Co.*, 661 S.W.2d 705 (Tex. 1983).

2. A test that has been used by the courts to determine if a defendant is "connected with" a transaction is whether he "sought to enjoy the benefits of the sale." *Luker v. Arnold*, 843 S.W.2d 108, 111 (Tex. App.--Fort Worth 1992, n.w.h.); *Parkway v. Woodruff*, 857 S.W.2d 903, 908 (Tex. App.--Houston [1st Dist.] 1993, n.w.h.).

In *Parkway*, the court held that homeowners could recover from the developer of their "master-planned" community, though they purchased their home from a separate builder, because the developers sought to enjoy the benefits of the transaction.

a. A consumer can therefore sue an engineer, contractor, subcontractor, supplier, real estate broker, inspector, real estate appraiser, that commits a deceptive practice related to the

construction or design of the house. Even more far reaching cases have involved developers and lenders.

C. Liability for a defective foundation might be premised on one or more of the following grounds:

1. "Laundry List" of 24 False, Misleading, or Deceptive Acts or Practices DTPA §§ 17.50(a)(1) & 17.46(b)(1)-(24)

a. Representing that a built project in general, or its foundation in particular, has "characteristics, ingredients, uses, benefits or quantities" that it does not actually have. (e.g., misrepresenting that a foundation will not shift or crack, or the amount of steel that the foundation contains).

b. Representing that a project or its foundation is "of a particular standard, quality, or grade." *Jim Walter Homes, Inc. v. Chapa*, 614 S.W.2d 838 (Tex. Civ. App.--Corpus Christi 1981, writ ref'd n.r.e.) (builder's misrepresentation that home would be built in a "good, substantial, and workmanlike manner" created grounds for DTPA liability).

c. Failing to meet contractual obligations to supervise and inspect the work of employees. *Building Concepts, Inc. v. Duncan*, 667 S.W.2d 897 (Tex. App.--Houston [14th Dist.] 1984, writ ref'd n.r.e.).

d. Passing off goods and services as those of another.

e. Representing that an agreement confers rights, remedies or obligations which it does not have.

f. Representing that a guarantee or warranty confers rights or remedies which it does not have.

g. Representing that work or services have been performed on goods when the work or services were not actually performed.

2. Breach of an Express or Implied Warranty -- DTPA § 17.50(a)(2)

a. Express warranties

(1) Example: Representation that all defects in house will be repaired by builder. *Moore Bros. Lumber Co. v. Toombs*,

614 S.W.2d 858 (Tex. Civ. App.--Texarkana 1981, no writ).

b. Implied warranties -- implied by law

(1) 4 types applicable to construction and development:

(a) Implied warranty of good and workmanlike construction: House was constructed in a good and workmanlike manner. *March v. Thiery*, 729 S.W.2d 889 (Tex. App.--Corpus Christi 1987, no writ).

"Good and workmanlike manner:" that quality of work performed by one who has the knowledge, training, or experience necessary for the successful practice of a trade or occupation performed in a manner generally considered proficient by those capable of judging such work. *Melody Home v. Barnes*, 741 S.W.2d 349, 354 (Tex. 1987).

(b) Implied warranty of habitability: House is suitable for human habitation. *Humber v. Morton*, 426 S.W.2d 554 (Tex. 1968).

(c) Implied warranty of good and workmanlike development. *Luker v. Arnold*, 843 S.W.2d 108 (Tex. App.-- Fort Worth 1992, n.w.h.).

(d) Implied warranty to repair or modify existing tangible goods or property in a good and workmanlike manner. *Melody Home Mfg. Co. v. Barnes*, 741 S.W.2d 349 (Tex. 1987).

(2) These implied warranties may not be waived.

(3) Implied warranties arise by operation of law, when public policy so mandates. *Parkway v. Woodruff*, 857 S.W.2d 903, 911 (Tex. App.--Houston [1st Dist.] 1993, n.w.h.).

(4) Implied warranties exist at time services are sought or acquired. *Parkway v. Woodruff*, 857 S.W.2d 903, 911 (Tex. App.--Houston [1st Dist.] 1993, n.w.h.)

(5) Arguably, implied warranties do not apply to professional services. *Dennis v. Allison*, 698 S.W.2d 94 (Tex. App.--Fort Worth 1988, writ denied).

(6) These implied warranties are automatically extended to subsequent purchasers to cover latent defects not discoverable by a reasonably prudent inspection at the time of the later sale.

3. Unconscionable Action -- DTPA § 17.50(a)(3)

a. An "unconscionable action" is one that takes advantage of a lack of knowledge, expertise, ability, or capacity to a grossly unfair degree *or* results in a gross disparity between the value received and that paid. DTPA § 17.45(5)

b. *Pfeiffer v. Ebby Halliday Real Estate, Inc.*, 747 S.W.2d 887 (Tex. App.--Dallas 1988, no writ) (finding no showing of unconscionability in case involving foundation problems due to soils).

4. An independent inspection may constitute a new and independent intervening cause of a purchase and sale transaction with a subsequent purchaser.

D. Proceeding With a DTPA Action: Statutory Notice & Offer of Settlement requirements -- DTPA § 17.505

1. Consumer must give defendant 60 days written notice before filing suit.

2. Notice must reasonably detail the specific complaint and the amount of actual damages and expenses incurred, including attorneys' fees.

Rather than dismissing action when notice requirements are not followed, a court will merely abate action. *Hines v. Hash*, 843 S.W.2d 464 (Tex. 1992).

3. Upon written request, a consumer must allow an opportunity to inspect. Unreasonable refusal to inspect results in loss of automatic doubling of actual damages under $1,000.

4. Within 60 days after receipt of the notice letter, the Defendant may tender a written settlement offer. If no notice received, the settlement offer may be tendered within 60 days after the suit has been filed.

5. The consumer has 30 days after receipt of the offer to accept the settlement; if not accepted, it is considered rejected.

6. If the consumer rejects the offer, it may be filed with the court along with an affidavit certifying its rejection. If the court finds actual damages to be the same, substantially the same, or less than the settlement offer, the consumer cannot recover an amount in excess of the settlement offer or the actual damages found by the court, whichever is *less*.

   This provision allows a Defendant to offer "substantially the same as" the amount of actual damages, which may be less than actual damages. However, the offer must include an offer of attorneys' fees. *Cail v. Service Motors, Inc.*, 660 S.W.2d 814 (Tex. 1983).

E. Damages Available -- DTPA § 17.50(b)

1. Purchaser could seek rescission -- to "undo" the sale. *Coyter v. MCR Const. Co.*, 673 S.W.2d 938, 941 (Tex. Civ. App.--Dallas 1984, writ ref'd n.r.e.). However, the seller is entitled to the fair market rental value of the house for the time the Plaintiffs occupied the home. This offset will reduce damages and in some cases could actually require the Plaintiff to pay the seller to rescind the sale. An argument also exists that the purchaser's tax benefits (tax deductions taken for points, mortgage interest, and taxes paid) should be accounted for, but this has not yet been addressed by a Texas court.

   a. Example:

Plaintiff's Expenses	
Past Repairs	500
Mortgage Interest	30,000
Closing Costs & Down Payment	10,000
Subtotal I	40,500
Less: FMV Rental for 42 Months @ $1,000	(42,000)
Subtotal II	(1,500)
Less: Tax Benefits -- Interest payments @ 28%	(9,000)
Net Amount Due Seller	$10,500

b. Another downside of rescission, from purchasers' perspective, is requirement that Buyer tender real estate back to Seller.

c. Arguably, mental anguish is not recoverable in a rescission claim.

2. Actual Damages

a. "Benefit of the Bargain" Measure of Damages

(1) Comprises difference between amount actually paid for a home constructed without defects and the value of a home with the alleged defects. *Jim Walter Homes, Inc. v. Gonzalez*, 686 S.W.2d 715 (Tex. Civ. App.--San Antonio 1985, writ dism'd).

(2) Applicable when builder has not substantially complied with building contract. *Turner, Collie & Braden, Inc. v. Brookhollow, Inc.*, 642 S.W.2d 160 (Tex. 1984).

(3) May result in further damages due to reduced market value of "damaged goods," because of the homeowner's own legal obligation to inform a subsequent buyer of the repaired defect, and the "stigma" related to such repair.

(4) Election to seek benefit-of-the-bargain damages may preclude Plaintiff from obtaining damages for mental anguish. *Keith v. Stoelting, Inc.*, 915 F.2d 996, 999 (5th Cir. 1990).

b. Remedial or Out-of-Pocket Measure of Damages

(1) Includes all of Plaintiff's expenses necessitated by the defect, including all necessary repairs, as well as all future repair costs due to continuing breaches resulting from a failure to make promised repairs. *Brighton Homes, Inc. v. McAdams*, 737 S.W.2d 340 (Tex. App.--Houston [14th Dist.] 1987, writ ref'd n.r.e.); *Jim Walter Homes, Inc. v. Gonzalez, supra*. Typical compensable repair costs would include soils analysis, foundation stabilization, cosmetic repairs, and landscaping repairs).

(2) Includes consequential damages (e.g., loss of use of property, temporary housing costs during repairs).

(3) Includes any diminution in market value of the property after repairs are made. *Ludt v. McCullom*, 762 S.W.2d 575 (Tex. 1988).

(4) Includes damages for mental anguish ("soft" damages). *J.B. Custom Design & Bldg. v. Clawson*, 794 S.W.2d 38 (Tex. App.--Houston [1st Dist.] 1990, no writ); *HOW Ins. Co. v. Patriot Fin. Servs. of Texas, Inc.*, 786 S.W.2d 533 (Tex. App.--Austin 1990, writ denied).

3. Automatic doubling of any actual damages of $1,000 or less.

4. "Additional damages:" up to 3 times the amount of actuals in excess of $1,000 may be awarded <u>if</u> Plaintiff shows that the Defendant committed the deceptive acts or practices "knowingly." *See March v. Thiery*, 729 S.W.2d 889 (Tex. App.--Corpus Christi 1987, no writ) ("knowingly" means actual awareness of defects in construction of house); *Jim Walter Homes, Inc. v. Valencia*, 690 S.W.2d 239 (Tex. 1985).

   a. Treble damages may not be recovered from the Real Estate Recovery Fund, which was established in order to pay back monetary damages suffered by victims of unscrupulous real estate agents. *Pace v. Texas*, 650 S.W.2d 64 (Tex. 1983).

5. Court costs and reasonable and necessary attorneys' fees are recoverable.

F. Remedies Provided by DTPA Are Not Exclusive -- DTPA § 17.53

1. However, a Plaintiff may not obtain double recovery of actual or punitive damages where the same acts comprise the DTPA violation and the basis for the other cause of action.

G. Defenses to Liability under the DTPA

1. Plaintiff is not a "consumer"

   a. Burden of proof is upon claimant to establish his status as a consumer. *Farmers & Merchants Bank v. Ferguson*, 617 S.W.2d 918 (Tex. 1981).

2. Plaintiff has more than $25 million in assets, or is owned or controlled by an entity with $25 million or more in assets, and thus does not qualify as a "business consumer." *Eckman v. Centennial Savings Bank*, 784 S.W.2d 672 (Tex. 1990).

3. Lapse of statute of limitations

   a. Lawsuit brought more than 2 years after consumer discovered or should have discovered deceptive act or practice or breach of warranty is barred.

4. Defendant was merely "puffing"

   a. Mere expressions of opinion by a seller not made as a representation of fact are not actionable under the DTPA. *Dowling v. NADW Marketing, Inc.*, 631 S.W.2d 726 (Tex. 1982).

   b. A court of appeals has recognized puffery as a defense to the DTPA; though the Texas Supreme Court denied the writ of error, they did not agree or disagree with the defense. *Prudential Ins. Co. v. Jefferson Associates*, 839 S.W.2d 822, (Tex. App.--Austin), *writ denied per curiam*, 843 S.W.2d 476 (Tex. 1992).

5. An offer to settle in full was made by the Defendant and refused by the Plaintiff. DTPA § 17.506(d)

6. The Defendant gave reasonable and timely written notice of his reliance on information from the government or another source, and he did not know, nor could have known, of the falsity or inaccuracy of the information. DTPA § 17.506.

7. Note: If the court finds that Plaintiff's action is groundless, or was brought in bad faith or for the purpose of harassment, Defendant may recover its attorneys' fees and court costs as a counterclaim. DTPA § 17.50(c).

IV. Residential Construction Liability Act (RCLA) -- Tex. Prop. Code Ann. §§ 27.001 *et seq.* (Vernon Supp. 1994) [Added by Acts 1989, 71st Leg., ch. 1072, § 1, eff. Sept. 1, 1989; Amended by Acts 1993, 73rd Leg., ch. 797, §§ 1, 2 eff. Aug. 30, 1993].

A. Applicability

1. Applies only to residential construction: Single-family, duplexes, triplexes, quadruplexes, and condominium and cooperative apartment units. RCLA § 27.001.

2. Applies only to "construction defects" -- but does not include claims for damages for personal injury (including mental anguish) or death, or for damages to goods. RCLA § 27.002.

3. "Construction defect": matter concerning design, construction or repair of a new residence, or the remodeling of an existing residence.

   a. Also extends to any "appurtenances" to a residence (e.g., swimming pool, detached garage, etc.), or the real property on which the residence or appurtenance are built.

   b. Appurtenances do not include furnishings or other personalty. Damages for these items must be sought through the DTPA or some other legal remedy.

B. Who May Invoke RCLA?

1. Only a "contractor" responsible for design, construction or repair ~~of a~~ new residence, or remodeling of or addition to an existing residence. Thus, if a Plaintiff sends a DTPA notice letter to a contractor, the contractor should invoke RCLA and inform the Plaintiff that he must comply with the RCLA requirements.

   a. "Contractor" includes a risk retention group that insures any part of a contractor's liability for the cost of repairing residential construction defects.

C. RCLA vs. DTPA

1. RCLA preempts the DTPA only where the two statutes conflict. Thus, the DTPA continues to apply to (a) engineers, architects, home designers, and other non-contractors involved in the design, construction and repair of homes or structural foundations; (b) appraisers; (c) real estate brokers; (d) lenders; (e) developers; and (f) the involvement of any of these persons with non-residential projects.

2. RCLA will not preempt the DTPA if:

   a. a claimant reasonably rejects a RCLA settlement offer; or

   b. the contractor fails to repair the defects within the allowed time in a good and workmanlike manner.

D. Who Is a Proper Plaintiff?

1. Anyone who suffers damages from a construction defect, i.e., anyone who seeks or acquires a contractor's services to design, build or repair a new home or to remodel or add to an existing home. A subsequent purchaser of the home is required to follow the RCLA procedures.

E. Proceeding Under RCLA -- RCLA § 27.004

1. As with the DTPA, claimant must first give the contractor written notice of the construction defect claim, by certified mail--return receipt requested, at least 60 days before filing suit.

2. After contractor has given written request, he must be given a reasonable opportunity to inspect the residence within 35 days after receiving the written notice of claim.

3. Within 45 days after receiving notice of claim, the contractor may make a written offer for money damages or to repair. The contractor may offer either:

   a. to repair the construction defect or to have the defect repaired by an independent contractor

   OR

   b. monetary settlement -- typically the cost to repair plus attorneys' fees reasonably and necessarily incurred by Plaintiff.

   c. The claimant and the contractor may agree in writing to extend the statutory period for notice, offer, and repair.

   Overall Rationale of RCLA: Promotion of reasonable settlements. Proceeding under RCLA minimizes the involvement of the lawyers and keeps the courts free of cases involving only minor damages. RCLA capitalizes on the fact that the contractor is often in the best position to quickly and inexpensively repair any defects. *See Hines v. Hash*, 843 S.W.2d 464 (Tex. 1992).

4. If the contractor's offer to repair is accepted, the repairs must be completed within 45 days (unless delayed by the claimant or events beyond the contractor's control -- e.g., weather, materials shortages).

   a. contractor must make a "good faith" effort to repair defect;

b. the repairs must cure the defect;

c. the repairs must be accomplished in a good and workmanlike manner.

5. If an offer of settlement is not accepted within 25 days after Plaintiff receives it, the offer is deemed rejected and Plaintiff may file suit under the DTPA or any other law. RCLA S 27.004(h).

a. If offer of settlement is unreasonably rejected, claimant's damages are limited to reasonable cost to repair defect plus attorneys' fees reasonably and necessarily incurred up until the time of rejection.

F. Defenses to Liability

1. Alleged defect is merely normal wear, tear, and deterioration. RCLA § 27.003.

2. Alleged defect is merely "normal shrinking due to the drying or settlement of construction components within the tolerance of building standards." RCLA § 27.003.

3. Damages were not proximately caused by the alleged construction defect. RCLA § 27.006

4. Damages against contractor will be proportionally reduced by percentage due to negligence of a subcontractor or his employee or agent. RCLA § 27.003(a)(1).

5. Damages will be proportionally reduced by percentage due to failure by anyone other than contractor--including the Owner--to take reasonable action to mitigate the damages. RCLA § 27.003(a)(2)(A).

6. Damages will be proportionally reduced by percentage due to failure by anyone other than contractor--including the Owner--to take reasonable action to maintain the residence. RCLA § 27.003(a)(2)(B).

7. Contractor reasonably relied upon written government information that was false or inaccurate. RCLA § 27.003(a)(5).

8. Contractor offered a reasonable settlement, which the fact finder determines to have been unreasonably rejected.

a. claimant's damages are limited to reasonable cost to repair the construction defect plus any attorneys' fees reasonably and necessarily incurred up until the time of rejection. RCLA § 27.004(d)

9. Statute of limitations.

10. Unlike the DTPA, all common law defenses apply to RCLA claims. RCLA § 27.003(b).

If Plaintiff did not give written notice of the complaint at least 60 days before filing a lawsuit, the court will probably follow DTPA abatement procedures (discussed above).

G. Damages

1. If contractor follows RCLA procedures, the new RCLA amendments provide that the claimant may recover only the following damages, if proximately caused by a construction defect:

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

   b. reasonable expenses of temporary housing necessitated by the repairs;

   c. reduction in market value of the residence, if any, due to structural failure; and

   d. reasonable and necessary attorneys' fees.

   RCLA § 27.004(g)

2. The new amendments limit damages in a proceeding under RCLA to the claimant's purchase price for the residence.

3. *However*, if contractor fails to make an offer of settlement, or its offer is reasonably rejected, a claimant may file suit under DTPA or any other legal remedy. Damages under such a proceeding may include:

a. Actual Damages

(1) "Benefit of the Bargain" Damages;

(2) Remedial or Out-of-Pocket Measure of Damages, which may include damages for Mental Anguish; or

(3) Rescission damages.

b. Doubling of any actual damages of $1,000 or less.

c. "Additional damages" of up to three times any actual damages over $1,000, upon showing that the contractor knowingly committed one of the prohibited deceptive practices.

d. Court costs and attorneys' fees.

4. Failure to meet provisions of RCLA may thus expose contractor to the DTPA's "additional" damages provisions (see above).

5. Contractors therefore have a great incentive to make a reasonable settlement offer under RCLA, and to meet the deadlines for carrying out the settlement.

6. Unreasonable rejection of a contractor's reasonable settlement offer limits damages to the cost of repairing the defect and attorneys' fees.

V. Practical Tips in Defending Foundation Defect Cases

A. The DTPA

1. As drafted, the DTPA is often vague.

2. Lower liability threshold required under the DTPA.

   a. Deceptive practice need only be a "producing" cause of damages, compared with "proximate" cause required to show negligence.

3. Juries practically apply instructions and usually require Plaintiffs to show violation of industry standards and causation.

4. Potential exists for runaway verdict in egregious situations.

5. The prospect of treble damages and attorneys' fees makes cases dangerous and creates settlement value.

6. Potential to preempt the DTPA: A clause in the sale agreement that requires resolution of any disputes through binding arbitration under the Federal Arbitration Act preempts the DTPA. *Jack B. Anglin Co. v. Tipps*, 842 S.W.2d 266 (Tex. 1992).

B. Problems with cases involving multiple Plaintiffs

1. "Slop-over" evidence.

2. Transactional defense costs create incentive for insurance carriers to settle on a nuisance or non-liability basis.

3. "Subdivision falling into the sea" hysteria -- aggravates damage claims.

4. Efforts by Plaintiffs to create their own stigma in a marketplace.

C. Problems with "Junk Science"

1. Expert witnesses are the key to trying foundation cases. Principal testimony of experts relates to whether the homes or foundations were:

   a. not constructed according to industry standards;

   b. defectively designed; or

   c. misrepresented by the seller or broker involved.

2. No industry standards exist as a basis for defining what comprises a "defective" foundation.

3. Problems exist with individuals making the judgment calls in vague areas -- a growth industry involving the biased "independent" expert.

4. Beauty truly is in the eye of the beholder: When is "too much movement too much?"

5. Subjective versus objective findings.

6. Expert witnesses must be qualified

7. Factual basis must exist for expert testimony.

a. May be based on hearsay (i.e., evidence not otherwise admissible at trial).

8. The expert testimony must assist the trier of fact:

a. to understand the evidence, or

b. to determine a fact in issue.

9. Expert may give an opinion on an ultimate issue.

10. Reliability versus Credibility: Issues relating to the admissibility of expert opinion testimony.

a. Trial court has wide discretion regarding admissibility and scope of expert opinions.

b. Most judges leave credibility to jury.

c. May attack expert opinion admissibility and reliability.

d. Preparation is the key in both direct and cross-examination.

e. Standard for admissibility of expert testimony. *Daubert v. Merrell Dow Pharmaceuticals*, 113 S.Ct. 2786 (1993). The Court held that the Federal Rules of Evidence (Fed. R. Evid. 702-705) superseded the *Frye* Test (holding "general acceptance" as the standard).

(1) The trial judge has the task of ensuring that an expert's testimony: (a) rests on a reliable foundation, and (b) is relevant to the task on hand.

(2) To rest on a reliable foundation, the testimony's subject must be "scientific, technical, or specialized knowledge," which connotes more than subjective beliefs or unsupported speculations.

(3) To be relevant, the testimony must "assist the trier of fact to understand the evidence or to determine a fact in issue." This helpfulness standard requires a valid, scientific connection to the pertinent inquiry as a precondition to admissibility.

(4) The proffered evidence may be challenged by cross-examination, the presentation of contrary evidence, and careful instruction on the burden of proof.

f. It is the trial judge's duty to ensure that any and all scientific testimony or evidence admitted is both relevant and reliable. *Daubert*, 113 S.Ct. at 2795.

g. The trial court may exclude relevant expert testimony if its probative value is outweighed by the danger of unfair prejudice, confusion of the issues, or misleading the jury, or by considerations of undue delay, or needless presentation of cumulative evidence. (Tex. R. Civ. Evid. 403).

11. The "Symposium Problem"

a. Experts gathering to create a standard of care in their industry.

b. Recognize conflict of interest involved

(1) Plaintiffs
(2) Defendants
(3) Experts
(4) Practitioners

c. Professional witnesses seeking to create peer review and accepted industry standards: Who pays in the short term versus the long run?

12. Failure to Keep Documents

a. Problems for the small contractor, design, or brokerage firm -- failure to keep records.

b. Documenting construction and design generally undercuts the Plaintiff's case.

13. Problems with Punch-lists and Homeowner Hysteria

a. Did the Plaintiffs get the home they bargained for?

b. Are the Plaintiffs looking for a windfall for a defect that doesn't exist?

c. Ask jurors to use their common sense -- they ordinarily do.

14. The Role of the Professionals

a. The Architect/Engineer

(1) Proper design

(2) Proper factual basis for design

(3) Proper construction inspections

b. The Developer

(1) Depends on scope of information provided to builder

(2) Developers give an implied warranty to develop in a good and workmanlike manner, creating a potential basis for DTPA liability. *Luker v. Arnold*, 843 S.W.2d 108 (Tex. App.--Fort Worth 1992, n.w.h.).

(3) How "connected" with a transaction is the developer? Is he seeking to enjoy the benefit of the transaction? *See, Parkway v. Woodruff*, 857 S.W.2d 903 (Tex. App.--Houston [1st Dist.] 1993, n.w.h.)

c. The Lender

(1) Depends on scope of involvement with construction inspections and draw reports. Generally speaking, a lender owes no duty of care to a prospective purchaser. *Baskin v. Mortgage & Trust, Inc.*, 837 S.W.2d 743 (Tex. App.--Houston [14th Dist.] 1992, writ denied)(lender made *no* representations, promises, guarantees, warranties, or statements to homeowners in connection with the home purchase).

(2) With foreclosure and resale, lender becomes the seller.

(3) If builder went bankrupt, did lender finish construction?

(4) Otherwise, lender liability is an extreme reach. There is no cause of action for negligent lending to a developer or a builder. *Baskin, supra.*

d. The Broker

(1) Absent specific agreement, broker is generally the agent of the seller.

(2) Broker is a "limited" or "special" agent.

(3) Potential arguments regarding this limited authority, based on ratification by seller's benefitting from the broker's affirmative misrepresentations, may require seller to rescind sale to protect itself from broker's conduct.

(4) Real estate agents and brokers have no legal duty to inspect property for defects beyond asking sellers if such defects exist. *Kubinsky v. Van Zandt Realtors*, 811 S.W.2d 711 (Tex. App.--Fort Worth 1991, writ denied).

(5) There is an implied warranty of good and workmanlike performance with respect to services of real estate brokers.

D. Regarding subcontractors, engineers, architects and brokers, you really want to require proof of insurance (review certificates of insurance/policies)

E. Indemnities/Disclaimers

1. Disclaimers of DTPA liability are normally void. DTPA § 17.42.

2. Under "fair notice requirements," any contractual disclaimer must be conspicuous. *Dresser Industries, Inc. v. Page Petroleum, Inc.*, 853 S.W.2d 505 (Tex. 1993).

3. Texas follows the "express negligence" test regarding disclaimers of negligence. *Ethyl Corp. v. Daniel Construction Co.*, 725 S.W.2d 705 (Tex. 1987).

F. Warranty Companies

1. "Benefit of the Bargain" Argument

2. Standard: Uninhabitable, unsanitary, or unsafe ("UUU")

   a. Differs from industry standards of construction and design-build.

   b. Example: "Sticking doors" are not defective construction or design, in and of themselves, but may trigger the HOW Warranty.

   c. Provide home purchasers with maintenance manual at closing.

G. "L/360" -- ACI 318

1. Allowable post-tension movement < L/360

2. Slope vs. deflection

3. Disagreements as to how measured in industry -- Is it misapplied?

4. Really begs the question: Is the balance of the structure failing due to the foundation's movement?

5. Everyone knows that minor foundation movement is normal in Houston, Texas;

6. Everyone knows that cure cracks occur in concrete;

7. Everyone knows that sheetrock cracks occur on a new home.

8. References to BRAB, Snowden & Littleton, and Post-Tension Institute.

9. No foundation is ever poured absolutely level.

10. L/360 is really a standard for *new* construction -- technically, it only applies at construction.

VI. More Honored in the Breach

HORATIO: What does this mean, my Lord?

HAMLET: The King doth wake to-night and takes his rouse,
Keeps wassail, and the swagg'rig up-spring reels;
And as he drains his draughts of Rhenish down,
The kettle-drum and trumpet thus bray out
The triumph of his pledge.

HORATIO: Is it a custom?

HAMLET: Ay, marry, is't,
But to my mind, though I am native here
And to the manner born, it is a custom
More honor'd in the breach than in the observance.

WILLIAM SHAKESPEARE, HAMLET act 1, sc. 4.