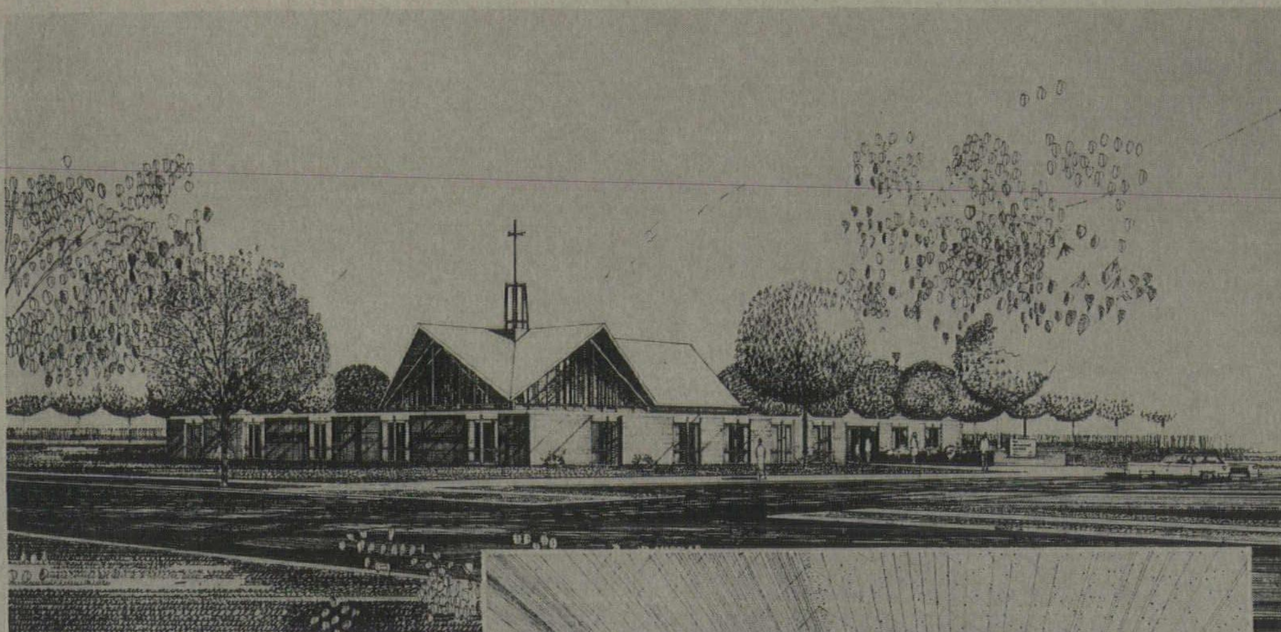
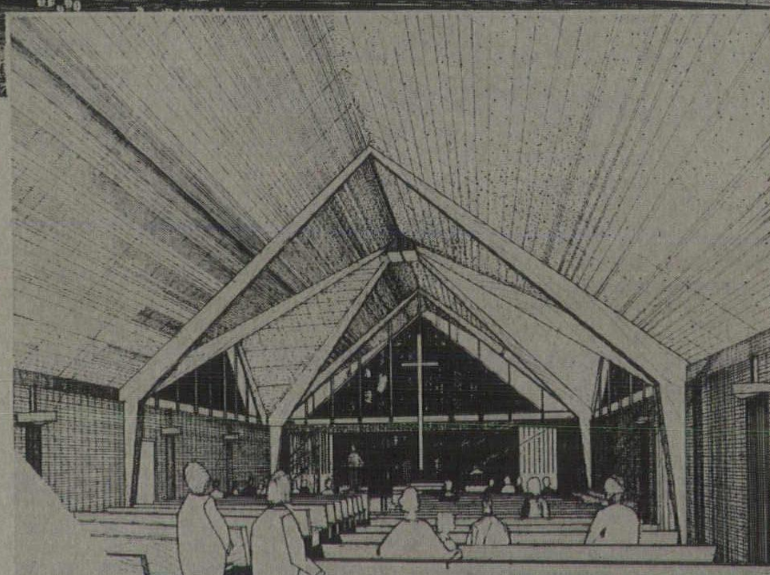
IOWA ARCHITECT

The face of architecture in Iowa





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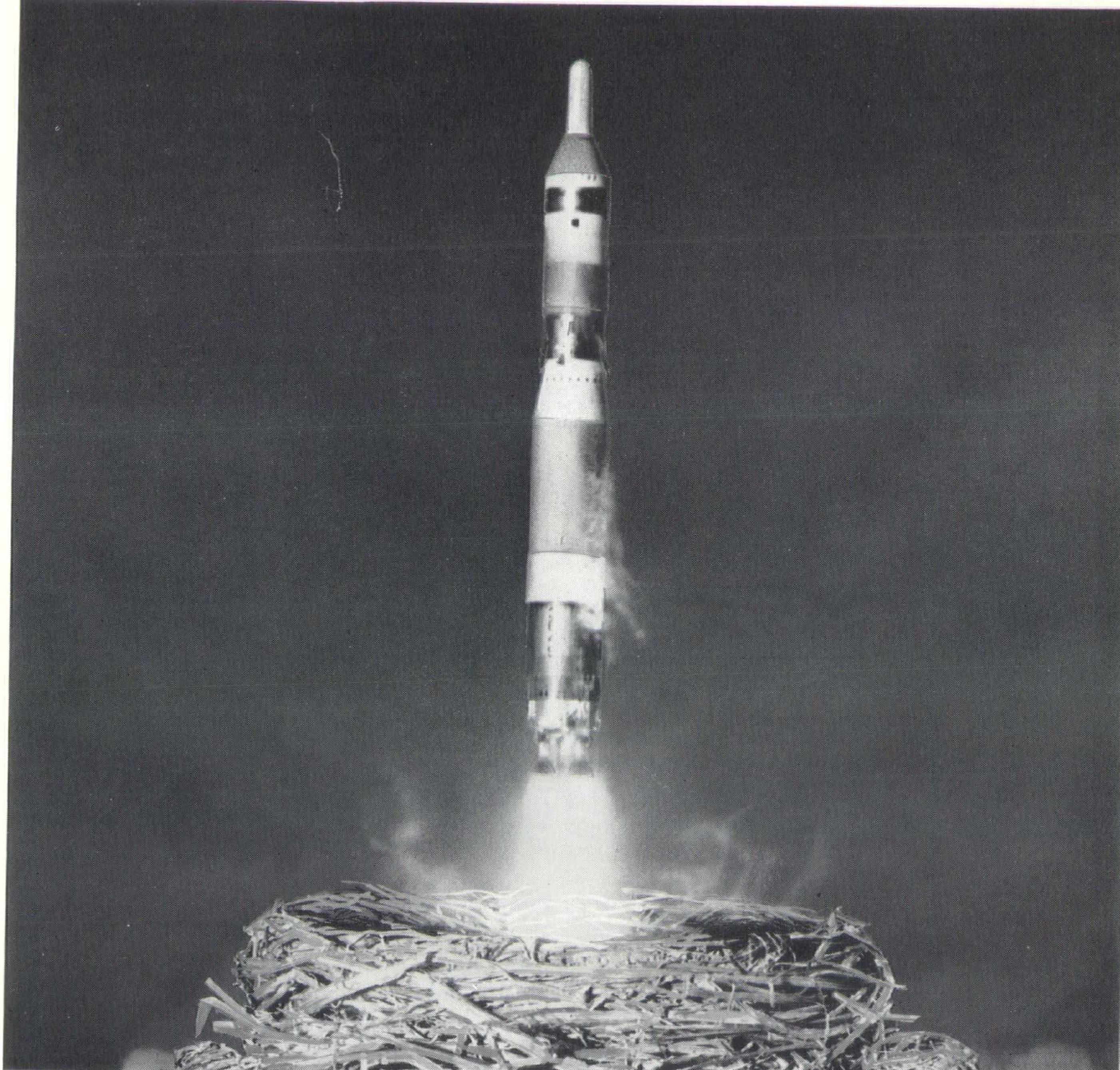

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In the preliminary design of multistory concrete buildings it is helpful if column size can be quickly approximated for a specific column spacing.

This can be accomplished by use of the formula and the chart shown below. Both are based on the Working Stress Design method (ACI 318-63). In structures such as 575 Technology Square, where wind load is resisted by shear walls, only the axial load of columns need be considered.

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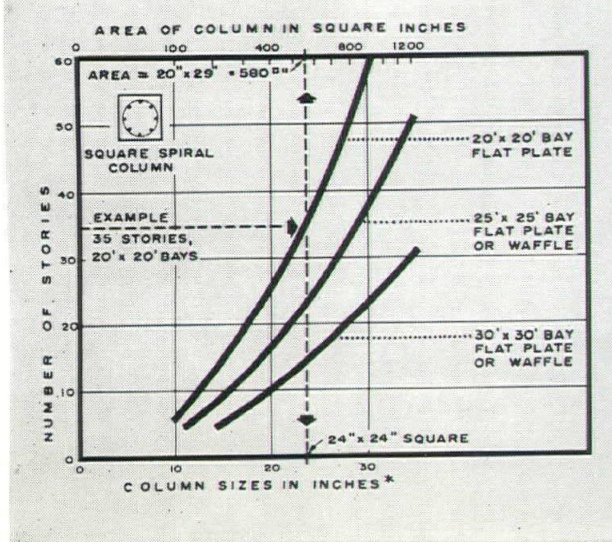


FORMULA:

The area of any column in square inches for any story is:

$$A = \frac{N\,(W_D + \tfrac{1}{2}\,W_L)\,B}{k}$$

A = column area in square inches

N = number of stories above

$W_D + W_L$ = dead and live loads (psf)

B = bay area (sq. ft.)

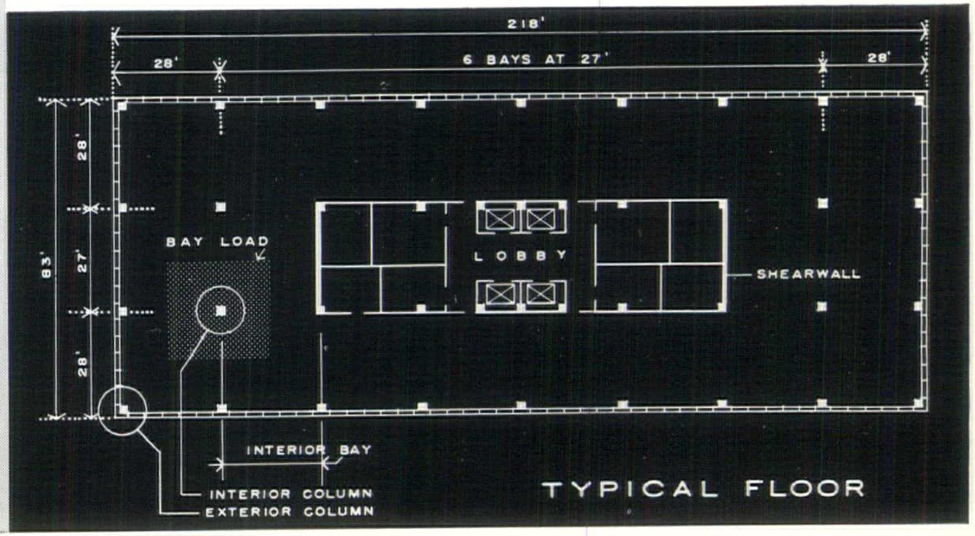
For 8% reinforcement + $f'_c = 5{,}000$ psi:

$k = 3{,}650$ for $f_y = 75{,}000$ psi.

$k = 3{,}170$ for $f_y = 60{,}000$ psi.

NOTE: The above equation and the graph are based on Working Stress Design (ACI 318-63)

*Columns are square with 8% reinforcement, $f'_c = 5{,}000$ psi, $f_y = 75{,}000$ psi and moment is negligible. In addition to the dead load of the structure, graph takes into account 35 psf for partitions, mechanical and ceiling. Assumed live load is 60 psf.



IOWA ARCHITECT

The face of architecture in Iowa

DIRECTORY ISSUE

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CONTENTS

Cover: Night photograph, Osmundson Manufacturing Co., Perry, Iowa, designed by architects Carl Hunter and Russell Parks. See page 34.

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Subsurface Investigation For Structures

BY MAX CALHOON, P.E.

Mr. Calhoon is Chief Soils Engineer for the Layne-Western Company and has been active in the field of soil mechanics since 1948. He is a graduate of Iowa State University with a B.S. degree in Civil Engineering and also a graduate of Northwestern University with an M.S. degree in Civil Engineering. His experience in the field of soil mechanics has covered a large geographic area of the United States and has included work on commercial and industrial buildings of almost every type and size, highways and bridges, earth dams, TV antennas, elevated water tanks, ground storage tanks, oil refinery equipment, etc. He is a registered professional engineer in six states.

Twenty five years ago it was seldom that an architect thought it necessary to have any type of subsurface investigation made at a building site to determine the nature of the underlying soils, but during the past few years there has been quite an increasing trend on the part of architects and structural engineers to obtain borings at the sites of buildings which they are designing. It is also becoming more common for architects and structural engineers to request that laboratory testing and an engineering analysis of foundation conditions be included in a subsurface report. They have increasingly come to the realization that the behavior of the underlying soil in reaction to the stresses imposed by a structure is a complex matter requiring the analysis and judgement of an experienced soils engineer.

The building materials that an architect ordinarily works with are usually produced under such rigid quality control that he knows within narrow limitations what he can expect from them in the way of performance. Unfortunately, Nature did not cooperate with us in this respect and the subsoil conditions at the site of each and every structure are almost invariably different from those at an adjacent site. Testing should be accomplished on the soils at each specific location and an engineering analysis should be made to determine the effect of the structural loads upon the underlying soils if the structural loads are to be supported by a foundation which will limit deformation of the structure to tolerable amounts.

A complete subsurface investigation would consist of performing borings at the structure site, performing laboratory testing on the soil samples which are recovered from the borings and performing an engineering analysis based on the results of the borings and laboratory testing, the latter being integrated with the structural design of the building. It is the purpose of this article to discuss each of the various steps in the subsurface investigation in order to acquaint the reader with the essential components of each phase.

It would be very easy for an owner or designer to contact a company which specializes in subsurface investigation work, tell them that he is designing a building at the corner of X Street at Y Avenue and request that they make a few borings and recommend a bearing value. Unfortunately, an intelligent subsurface investigation is not this simple—it must be tailored to fit the exact building that has been visualized. The location of the borings, the depth of the borings, the types of samples to be recovered, the sampling interval and the types of laboratory tests to be performed are all functions of the size of the building, the arrangement of the structural loads and the magnitude of the structural loads to be imposed on the subsoil; thus it is not prudent for the company to "make a few borings and recommend a bearing value" without having further knowledge of the proposed structure or knowing how the information is to be used. Perhaps this will become more apparent in the following discussion of the phases of a sound subsurface investigation.

BORING PROGRAM

Boring Layout

The first step in determining a boring program is to decide the locations of the borings. To be able to predict precisely the behavior of the soil beneath a spread footing would necessarily require that all the soil which will be stressed by the footing be observed and tested by a soils engineer. This, of course, is a physical impossibility. We could also consider putting borings down on 2-foot centers underneath each footing. By this procedure we would know less than if we had seen and tested all of the soil beneath the footing, but we would certainly know more than if borings had been put down on 10-foot centers. Obviously, this would be economically unjustified. It is therefore necessary to decide upon a spacing of borings that results in economy for the owner and a statistical average of the boring and testing information useful enough to assure reasonably correct predictions by the soils engineer. It has been found through experience that it is seldom necessary to space borings closer together than about 35 to 40 feet, or farther apart than 100 to 125 feet if spread footings are to be used. Spacings closer together than 35 to 40 feet cannot be justified economically in most cases, while spacings farther apart than 100 to 125 feet apart introduce the probability that some of the soil conditions at the site will not be observed at all.

The soils engineer should be in constant contact with the drilling crew so that boring spacings can be altered in accordance with the type of material being encountered and the type of loads that must be supported. For example, borings may have been planned on 60 foot centers for a building about 250 feet square. The first boring may indicate that spread footings cannot be used at the site and that the building will have to be supported by a deep foundation on a hardpan layer which may be 50 feet beneath the ground surface. If piles seem to be a better choice for the foundation, then the boring program might be limited to about 4 borings, one in each corner of the building, to define the location of the hardpan layer.

Boring Depth

If the soil conditions are such that the building can be supported upon spread footings, the boring should extend to at least a depth beneath the proposed footing elevation where the stress induced by the footing is reduced to approximately 10% of the stress immediately below the footing. For a square footing, this point is at a depth beneath the footing equal to approximately twice the width of the footing. For a long, narrow (strip) footing, this point is at a depth beneath the footing approximately 5 times the width of the footing. Since the dimensions of the footing are a function of the column load or wall load and of the subsoil conditions, it is not possible before the borings are made to determine just where the 10% stress level is located. For the purpose of estimating the depth of borings prior to starting them, it has been found practical to assume a unit footing pressure of 2,000 pounds per square foot and divide this value into the total column load or total wall load. This calculation will yield a footing dimension, and the depth of the borings can be tentatively determined, assuming that the configuration of the building is known to the extent that a minimum footing depth has been determined. If the soils engineer is in constant contact with the drilling crew, he can make a judgment after the first boring has been completed as to whether the soil conditions are better or worse than the assumed 2,000 pounds per square foot and adjust the depth of the borings accordingly. For example, if it appears that the soil conditions will support footings with 6,000 pounds per square foot unit pressure, then the depth of the remaining borings on the site can be reduced, providing that the soil conditions remain similar to those at the first boring. If it appears that the soil will not support footings with 2,000 pounds per square foot unit stress and the structural loads are relatively heavy, it is quite probable that a deep foundation of some type may be required. In this case, borings should be taken to a depth sufficient to determine pile lengths or to determine where caissons could be supported. It is strongly emphasized that for a specific area of soil, many drill holes taken to half the necessary depth are worthless, while a few holes at a wider spacing taken to the proper depth are invaluable.

One method of approximate boring depth determination has been explained. There is another, however, by which the soils engineer can refine the depth determination during the boring operation itself. This method uses concomitantly his knowledge of two tests: the consolidation test and the unconfined compressive strength test. Both tests are related to the settlement behavior of soil, and it is their combined characteristics that facilitate depth determination. Let us examine this point further.

The settlement behavior of a soil is dependent upon the stress to which it has been subjected in its geologic history. This stress consolidated the soil and thus is known as the "pre-consolidation stress." It is determined in the laboratory through the use of the first test mentioned, i.e., the consolidation test. Very little, if any settlement will occur in a soil until the preconsolidation stress is exceeded. If, then, we could use the consolidation test to determine this stress during boring operations, we would obviously have an invaluable indication of how deep to drill. Unfortunately, the consolidation test is a relatively expensive test which takes several days to complete, and therefore cannot be used by the soils engineer for this purpose. The second test, the unconfined compressive strength test can, on the contrary, be rapidly made during the boring operation. If we now examine the performance and interrelationship of these tests, we will see how a soils engineer's knowledge of both allows him to make depth decisions while drilling.

The consolidation test specimen is typically 2 inches to 4 inches in diameter and 0.5 inch to 1.0 inch in thickness. It is confined laterally within a circular metal ring while a vertical load is being applied. In contrast, the unconfined compressive strength specimen is typically 1.4 inches to 3.0 inches in diameter and 2.8 to 6 inches in height. In other words, the height of the consolidation test specimen is less than its diameter and it is confined within a metal ring, whereas the height of the unconfined compressive strength specimen is approximately twice its diameter and it has no lateral support during compression. The maximum allowable stress or "yield point" of a soil can be most exactly determined by a consolidation test but it can be logically deduced from the previous description of the manner of testing that the maximum allowable stress or "yield point" of the unconfined conpressive strength specimen will not be greater than the yield point of the consolidation test specimen. The "yield point" of the consolidation test specimen is the point on the stress-strain curve at which considerable deflection begins to take place with increasing load. This so-called yield point of the consolidation specimen is also the preconsolidation stress.

Since we can determine the unconfined compressive strength of the soil during the boring operation, we can also form some opinion of the preconsolidation or "no-settlement" stress because, as stated above, the unconfined compressive stress will not be greater than the pre-consolidation stress. Knowing the loads on and the unconfined compressive strength of the soil involved, the soils engineer can determine footing sizes as samples are recovered. If the unit soil pressure caused by a particular footing size is such that the total stress in the subsoil is approximately equal to or less than the unconfined compressive strength, there is little chance for settlement of the footing, and drilling can stop when the 10% stress level is reached. (The "total stress in the subsoil" means the total stress in that volume of soil beneath the footing that will be influenced by the footing.)

This procedure to determine boring depth is a valuable refinement of the original crude boring depth estimate which was made by assuming a footing size for 2000 pounds per square foot unit soil pressure and determining the boring depth using the rules of thumb mentioned at the beginning of this section. It should be emphasized, however, that the engineering analysis may later result in a recommended bearing value that is different than that used to determine the boring depth.

Sampling Interval

The interval of depth at which samples are recovered should be rather narrow in the upper portions of the borings and wider in the lower portion of the borings. In general, it is desirable to recover samples at maximum intervals of 2.5 feet from a point at the footing elevation to a point below the footing where the stress which is induced into the subsoil by the footing has been reduced to approximately 30% of the footing unit stress. Below this point, the sampling interval may be increased to approximately 5 feet. This procedure will assure that samples are recovered at close intervals in the zone beneath the footing where the major portion of the footing stresses are dissipated. It is within this zone that detailed information concerning the physical properties of the soil is required. In special cases, it may be desirable to recover samples continuously for the full depth of or for a partial depth of a boring. Again, there is the problem of determining before the borings are made the depth where the unit stress will be reduced to 30% of the footing unit stress. For the purpose of determining the sampling interval, it is usually sufficiently accurate to determine the 30% stress level by using a footing size which is determined on the assumption of a 2,000 pound per square foot unit stress. To give the reader some idea of the magnitude of the depth being discussed, if samples are recov-

figure 1

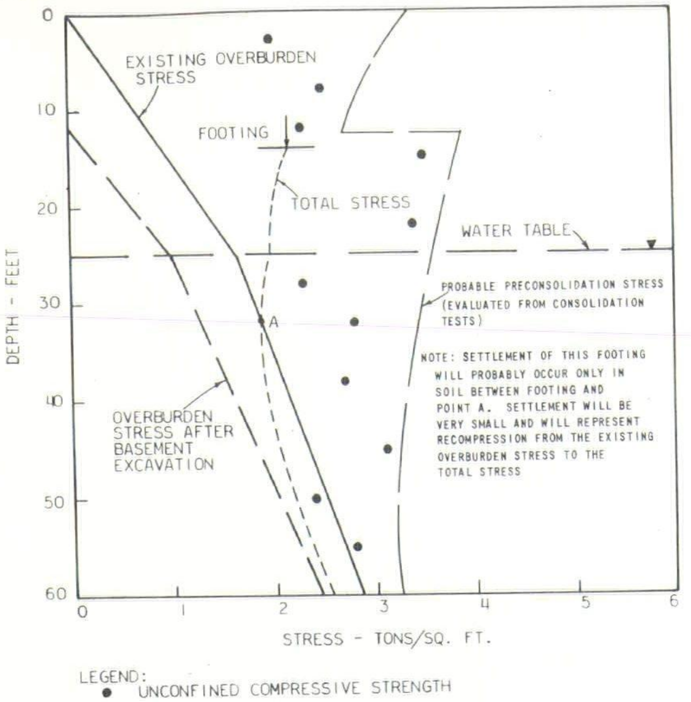


figure 2

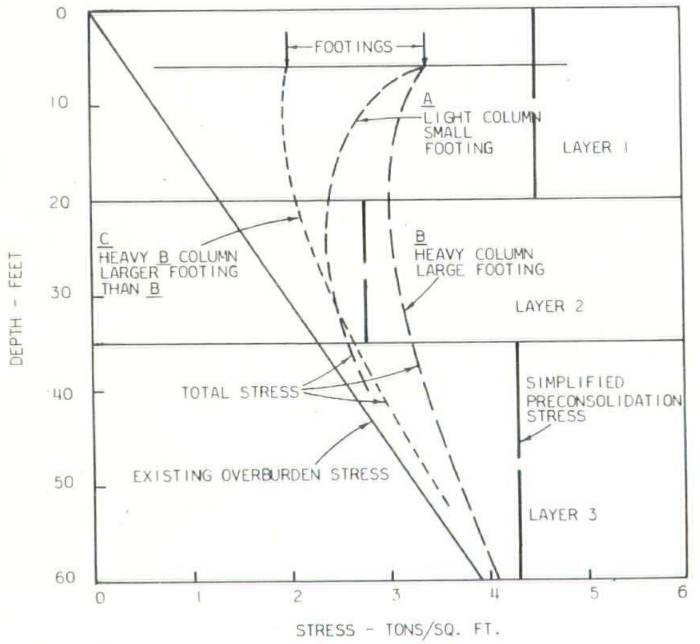
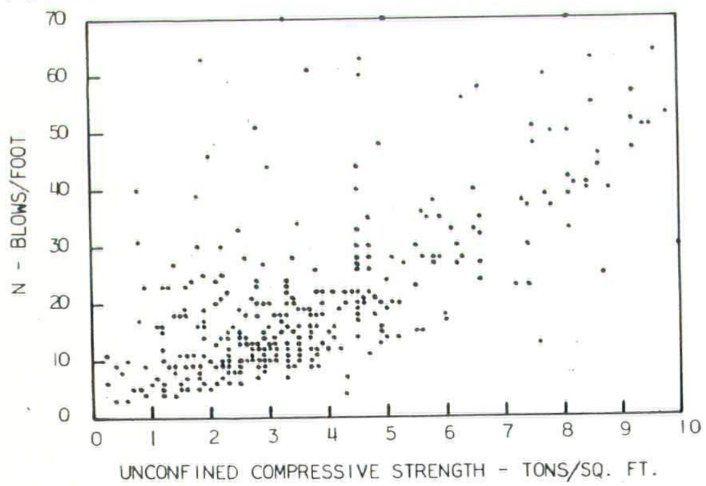


figure 3



ered at 2.5 foot intervals from a point at footing elevation to a point 15 feet beneath the footing elevation, such samples will have been recovered above the 30% stress level for a column load of up to 500 kips based upon a 2,000 pound per square foot unit soil pressure. For a 4,000 kip column, the 30% stress level would occur approximately 40 feet beneath a footing with a 2,000 pound per square foot unit soil pressure.

Soil Samplers

Although there are many types of soil samplers in use today, there are two types which are most commonly used. These are the samplers which are known as the "split-spoon" sampler and the thin-wall "Shelby tube" sampler. There are many more sophisticated types of samplers available; however, they are intended for use in special situations.

The split-spoon sampler was originally devised in about 1925 by Mr. Linton Hart and Mr. Gordon Fletcher of the Gow Company. The Sprague and Henwood Company also lays claim to having independently devised a split-spoon sampler at about the same time. This sampler was the outgrowth of an earlier pipe sampler which had been in use since about 1902. There have been few changes made in the split-spoon sampler through the years, and those being used today are very similar to the original sampler. The standard split-spoon sampler has an O.D. of 2 inches and an I.D. of 1-3/8 inches. The sampler is driven into the ground by the blows of a 140-pound weight falling a distance of 30 inches.

Thin-wall steel "Shelby tubes" with a wall thickness of about 1/16 inch came into use much later, after it was realized that the physical properties of the soils being recovered with the thick-walled split-spoon sampler were being altered by the procedure of driving it into the soil. The thin-wall tubes are usually pressed into the soil with a static weight. Disturbances to soil properties also occur using the thin-wall tubes, but are much less extensive than those caused by the split-spoon sampler. It is the general practice today to use the split-spoon sampler only in sand and gravel soils or cohesionless silt soils where disturbance to the soil sample is not a major factor. Samples of cohesive slits and clays should be recovered with thin-wall samplers to avoid disturbance of the sample as much as possible, although when dealing with very stiff to hard clay soils into which the thin-wall tubes cannot be pressed, it is permissible to either drive the thin-wall tube by light taps of the 140-pound hammer or to recover samples with the split-spoon sampler. Disturbance of very stiff or hard clays by the sampler is much less a factor than it is in softer materials. The following are recommendations concerning the choice of a sampler:

Sand and gravel:	Split-spoon
Cohesionless silt:	Thin-wall Shelby tube or Alternate split-spoon sampler with thin-wall Shelby tube.
Cohesive silt:	Thin-wall Shelby tube
Loess:	Thin-wall Shelby tube
Silty clay and clay:	Thin-wall Shelby tube

Groundwater Level

In addition to the other items of the boring program discussed above, another soil characteristic which must be observed during the drilling operation is the level of the groundwater table. The position of the groundwater table can have an influence on problems which may be encountered during the construction of the

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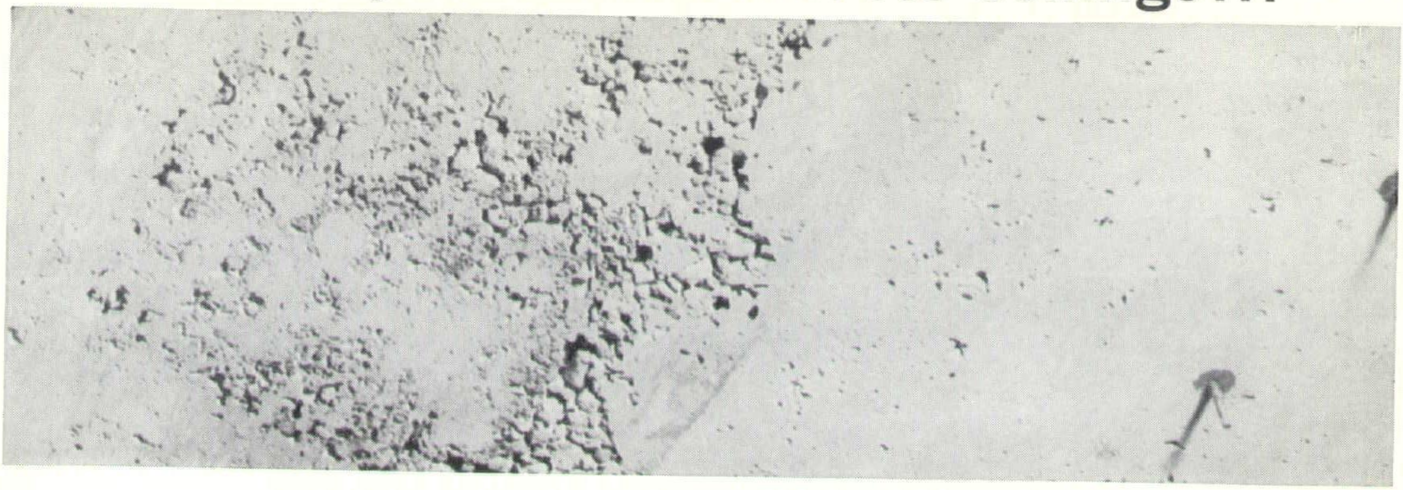

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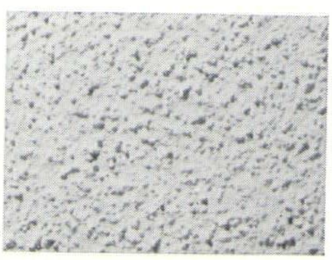

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foundation and may also have a bearing on the design of the basement walls and floor of the building to keep the building watertight. In addition, the position of the water table can also greatly influence the allowable soil bearing pressure for structural footings. The position of the groundwater table can affect the stress patterns which will develop beneath the structural footings and will therefore influence the behavior of the structural footings. The effect of the groundwater table on the stress pattern beneath a structural footing is illustrated in Figure 1. The elevation of the groundwater table also greatly influences the allowable bearing value for sand and gravel soils.

LABORATORY TESTING PROGRAM

In general, there is very little that can be done in the way of laboratory testing on sand and gravel soils since it is virtually impossible to obtain a sample of these materials in such a sufficiently undisturbed state that meaningful testing can be accomplished. Most soils engineers resort to the use of empirical design methods which involve the use of the split-spoon sampler blow count. Those soils engineers who do not believe in the use of the blow count as a design procedure have their own specialized methods of designing foundations on sand and gravel.

On cohesive silts, silty clay and clay soils, the laboratory tests which are most useful to the soils engineer are the moisture content test, the natural dry unit weight test, the unconfined compressive strength test and the consolidation test. Triaxial shear tests and Atterberg limit tests are also useful in many situations, although they are not usually a part of a standard laboratory testing procedure for foundation purposes. Quite often such tests as grain size analysis, field moisture equivalent, shrinkage limit, permeability, etc., are requested by architects and structural engineers, but these tests have little, if any application to most foundation problems.

It is the usual practice when dealing with cohesive soils to perform the natural moisture content test, the dry unit weight test and the unconfined compressive strength test. After studying the results of these tests, the boring data and the stress conditions to be imposed by the structure, the soils engineer can then decide whether further testing is required. If so, he can make an intelligent appraisal of just which samples should be subjected to consolidation testing, triaxial shear testing, or possibly Atterberg limit testing. The consolidation and triaxial shear tests are relatively expensive to perform, and should be used judiciously. Even so, their relative expense should not deter the owner or the designer from their use because the tests may be essential to the analysis of the problem and to the obtaining of a correct answer. When compared to the value of the entire project, the cost of these tests represents very inexpensive insurance toward proper performance of their particular structure.

As a rule of thumb, if column loads do not exceed about 100 kips or wall loads do not exceed about 8 kips per foot, it is usually sufficient to perform the moisture content test, the dry unit weight test and the unconfined compressive strength test on cohesive soils. Unless a structure covers a very large area with light column loads and wall loads, it will usually be found that the cost of consolidation testing and triaxial shear testing is not justified. For a lightly-loaded building, a footing design can be determined that will limit the total stress in the subsoil to an amount that is approximately equal to or less than the unconfined compres-

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sive strength, and settlement should thus be at a minimum. This concept is illustrated in Figure 1. If a lightly-loaded building covers a large area where there is a considerable amount of money involved in concrete for the footings, a few consolidation tests may be justified to obtain a more economical footing design.

When column loads exceed approximately 100 kips or wall loads exceed about 8 kips per foot, it is the opinion of the writer that the use of consolidation testing is mandatory in cohesive soils to assure that the settlement of the footings will be within tolerable limits and that the most economical footing design is being used.

ENGINEERING REPORT

The engineering report should include a complete discussion of all the elements of the entire investigation, beginning with an outline of the boring program and including a description of the laboratory testing program, a description of the subsurface conditions and a complete analysis of the foundation conditions. A complete discussion of the foundation analysis should be included in which the soils engineer summarizes the effects of the investigation elements on his thinking, and presents those methods of analysis which have influenced his final judgement. In short, the engineering report should present a detailed discussion of everything that has been accomplished for the owner and designer in a manner that will allow the designer, after studying the report, to form an intelligent opinion as to the correctness of the soils engineer's considerations and conclusions. The one-paragraph letter-report which does nothing more than to state conclusions and recommendations can hardly serve to offer the designer confidence that the soils engineer has thoroughly studied all facets of the problem and has arrived at the most logical conclusion.

Since spread footings are usually the most economical type of foundation to support a building, the possibility of using them should be thoroughly explored as a first order of business by the soils engineer. If it is determined that a spread footing foundation is not feasible for one reason or another, then the soils engineer should logically progress to the study of such other types as raft foundations and deep foundations. These latter types include friction piles, point-bearing piles and caissons. If spread footings cannot be used for a structure, the economics of using other more expensive foundations become the province of the designer. In many cases, there will be more than one of the more expensive foundations that can be used, and it would be necessary for the soils engineer to discuss and recommend several different types so that the designer can then determine which of them is most economical and most suitable for his purposes. On the other hand, there are many times when spread footings are not feasible, and only one of the more expensive types is feasible. In such cases, it is the responsibility of the soils engineer to make the proper recommendation.

In the case of spread footings, raft footings and caissons which bear on soil, there are two factors which must be investigated to insure a proper design. First, the settlement properties of the underlying soil must be studied to make certain that the foundations will not settle more than a tolerable amount, and second, the bearing capacity (or resistance against a shear failure) must be considered if such a failure in the soil beneath the foundation is to be avoided.

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evaluated by soils engineers through the use of the split-spoon sampler blow count. There are empirical relationships between the blow count and the approximate settlement that can be expected; however, as previously mentioned, there is a group of soils engineers who do not believe in the use of the blow count and have their own specialized methods for evaluating settlement in such soils. The position of the water table has a great influence on the allowable bearing value for sand and gravel and must be observed carefully in the borings. Future fluctuations of the water table must also be considered in the engineering analysis.

The settlement of silts, cohesive silts, silty clays and clays is evaluated through the use of the laboratory consolidation test. In the ideal situation, the soils engineer would perform a series of consolidation tests on many samples from a single boring. From these tests, a profile of the pre-consolidation stress can be established as illustrated in Figure 1. Many times it will be found that this profile of pre-consolidation stress agrees rather closely with the existing overburden stress profile. Clays with such profiles have never been subjected to stresses which are in excess of the existing overburden stress and are known as "normally loaded" clays. They will settle when any structural load is applied. The soils engineer may be able to recommend designs which will limit the settlement to a tolerable amount, and in such cases the ingenuity and experience of the soils engineer are invaluable to the designer. In rare cases, it may be found that the profile of preconsolidation stress is less than the existing overburden stress. This situation would mean that the soil mass has not yet consolidated under its own weight. Should a structural load be applied, the structure would settle not only due to the weight of the structure but also due to the natural settlement which is occurring because of the weight of the soil. The most usual situation is to find that the pre-consolidation stress profile is greater than the profile of the existing overburden stress, either for the full depth of the boring or possibly in the upper portions of the boring. Such soils are termed "pre-consolidated" and significant settlement of such soils will not usually occur until the total stress in the subsoil induced by the structural load exceeds the preconsolidation stress. Pre-consolidation may result from one of several occurrences in the geologic history of the soil. If the soil is a glacial till, it may have been subjected to the weight of overlying glacial ice for many thousands of years; nevertheless, the identification of a soil as a glacial till does not necessarily mean that it has been pre-consolidated under the weight of ice. Soils can also be pre-consolidated by the loss of their natural moisture, or by other weathering forces. Residual soils which have been formed by weathering of underlying consolidated materials are often preconsolidated by the forces of drying and weathering. Windblown loessial soils are pre-consolidated or at least indicate an apparent pre-consolidation because of the action of cementing agents such as calcium carbonate or montmorillonite between the silt or fine sand soil particles. Many other soils often indicate apparent preconsolidation due to chemical or mineral constituents. Pre-consolidation of softer soils can also be artificially induced through the use of surcharge fills.

From the foregoing discussion, it should be evident that the soils engineer must be familiar with the geology of the area in which he is working. Basic geological information is essential to the proper understanding and analysis of subsoil conditions. It should also be evident that the key to the proper analysis of the behavi-

continued on page 37

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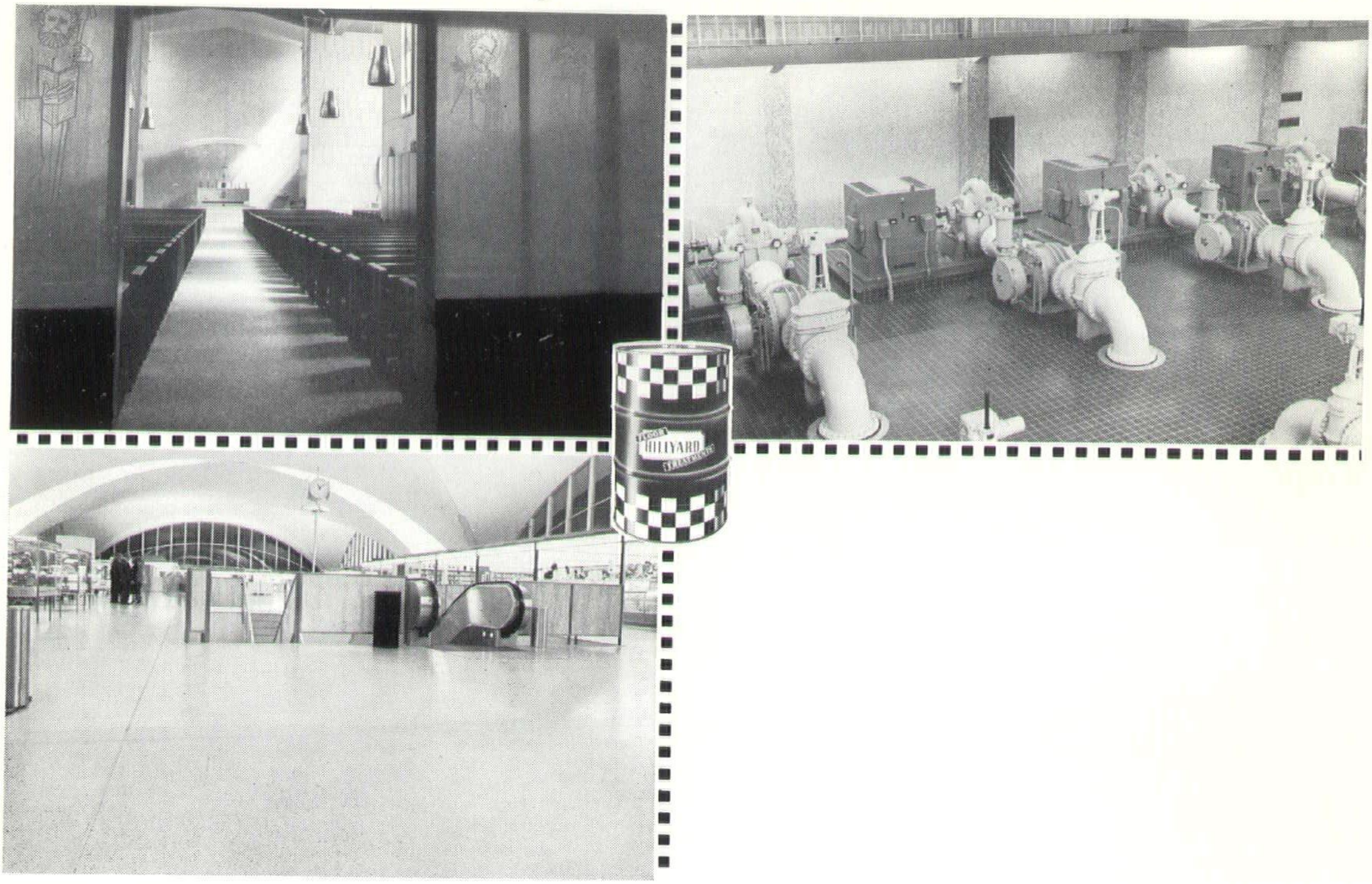

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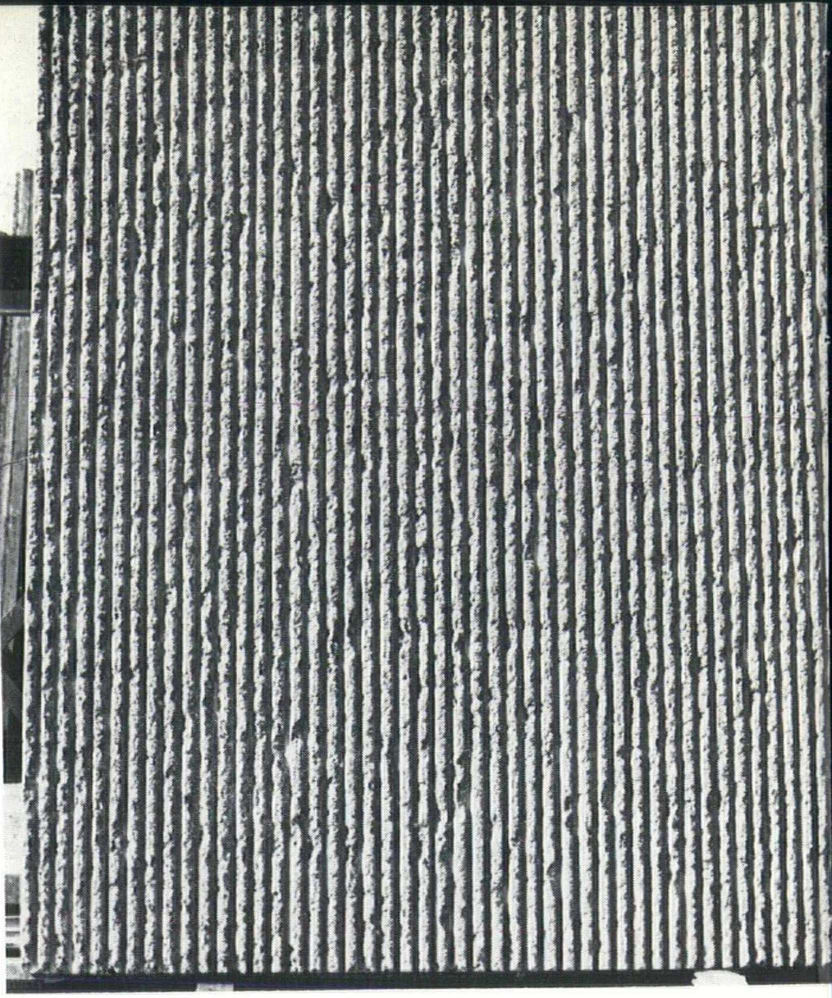

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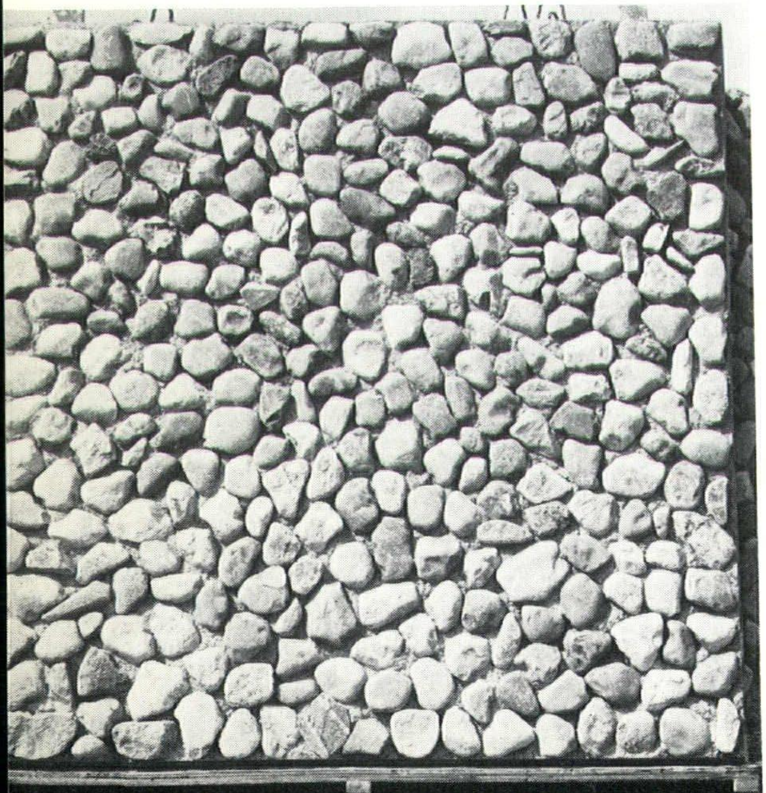

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Des Moines

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*680 59th St., Des Moines
Dougher-Frevert-Ramsey

McLennan, Donald M.
*1117 33rd St. SE, Cedar Rapids
Howard R. Green Co. 417 1st Ave. E

Robinson, Robert C.
415 Pensacola Ave.
Fairhope, Alabama

Spooner, George
7204 Reite Ave.
Des Moines, Iowa 50311

Tinsley, Vernon F.
13861 Barbados Drive
Largo, Florida 33542

Toenjes, David B.
132 Elmwood Ave.
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Sanford, Florida

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Brook-Borg

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Davenport

Bergland, Robert B.
*218 4th N.E., Mason City
Bergland and Bianco

Bianco, Harold F.
*43 River Heights Dr., Mason City
Bergland and Bianco

Blum, Carl R.
Dane D. Morgan and Associates
Burlington

Borg, Elmer H. (retired)
*1716 E. 31st Court
Des Moines

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*1700 Bell Ave., Des Moines
Brooks-Borg

Brierly, Robert S.
*Ridge Road, Carlisle
Porter/Brierly, Des Moines

Brom, Richard H.
*235 Four Seasons Drive, Waterloo
Thorson-Brom-Broshar

Broshar, Robert C.
*2229 Grand Blvd., Waterloo
Thorson-Brom-Broshar

Bunker, Franklin N.
*3812 Crestwood Dr. Des Moines
Board of Control, State Institutions
State Office Building, D.M. 50319

Camizzi, Francis J.
*2515 Indiana St. S.W., Cedar Rapids
Brown, Healey & Bock

Carpino, Ralph
*3118 SW 22nd St.,
Des Moines 50321
Amos Emery & Associates

Carrithers, Ira T. Jr.
*711 Perrin Ave., Council Bluffs
I. T. Carrithers, Architect

Carswell, Robin B.
*828 N. 4th St., Burlington
R. B. Carswell, Architect

Cleveland, Mortimer B.
*215 Prospect Ave., Waterloo
R. M. & M. B. Cleveland

Cleveland, Rhodes M.
*215 Prospect Ave., Waterloo
R. M. & M. B. Cleveland

Couch, Louis C.
*1215 Eastmere Dr., Bettendorf
Louis C. Couch, Architect

Cox, G. B.
*1445 Lakewood Dr., Bettendorf
G. B. Cox, Architect

Crites, Ray D.
*2600 Indian Hill Rd., SE.,
Cedar Rapids
Crites & McConnell

Dean, Waldo J.
*1704 45th St., Des Moines
Karl Keffer Associates

DeVoe, Robert C.
*1715 Orchard Dr., Cedar Falls
Robert C. DeVoe, Architect

Duffy, James M.
*3031 Jackson, Sioux City
James M. Duffy, Architect

Dunham, Edward M. Jr.
*1434 Highwood Dr. N.W.,
Cedar Rapids
Howard R. Green Co.,
417 1st Ave. SE

Earnheart, Robert E.
*608 Manor Drive, Iowa City
Powers & Associates

Eckman, Realand F.
*1231 2nd Ave. SE., Cedar Rapids
Kohlman-Eckman-Hukill

Emery, Amos B.
*2240 S.W. 11th St., Des Moines
Amos Emery & Associates

Feddersen, Phil H.
*818 North Second St., Clinton
Phil H. Feddersen, Architect

Fernbach, Richard B.
119 Devon Drive
Falls Church, Virginia 22042

Franzen, Archie W.
P.O. Box 151
Harpers Ferry, West Virginia

Frevert, W. David
*904 17th St., West Des Moines
Dougher-Frevert-Ramsey

Galvin, John C.
*121 W. 12th St., Spencer
Walsh, Keninger & Galvin

Gibson, Victor
*251 Four Seasons Drive, Waterloo
Deuth & Gibson

Gjelten, Gordon O.
*835 Forest Drive, Forest City
Gjelten and Schellberg

*indicates home address

Firm addresses on Page 23

Goewey, Richard W.
*4015 Cottage Grove, Des Moines
Winkler-Goewey, Architects

Gordon, Gene
*1160 Arrowhead Dr., Dubuque
Durrant-Deininger-Dommer-Kramer-Gordon

Gray, Donald Lowell
*305 5th Ave., Decorah
Altfillisch, Olson, Gray & Thompson

Griffith, Gerald I.
*2805 Terrace Drive, Des Moines
Griffith-Kendall

Griffith, Standford W.
*1515 Floral Ave., Ft. Dodge
The Griffith Company

Hansen, Richard F.
*433 Opland, Iowa City
Hansen-Lind-Meyer

Harrison, Roland T.
*1215 43rd St., Des Moines
Wetherell-Harrison-Wagner

Haynes, Kenneth L.
*724 24th St., Ames

Haynes, Stanley Foster
*1351 Ridgecrest Dr., Clinton
Prout, Mugasis and Johnson

Healey, Edward Hopkins
*2500 White Eagle Trail SE,
Cedar Rapids
Brown, Healey & Bock

Hecker, Robert D.
*407 Kings Highway, Sioux City
Robert D. Hecker, Architect

Heemstra, Howard Charles
Crites and McConnell, Cedar Rapids

Henry, Harvey W.
*Prairie Duchein Rd. R. R. 2,
Iowa City
Wehner and Henry

Herbert, Charles
*4906 SW 18th St., Des Moines
Charles Herbert and Associates

Horner, George L.
*1422 East College St., Iowa City
State University of Iowa
200 Gilmore Hall

Hotchkiss, Walter Alan
*2615 Druid Hill Dr., Des Moines
Iowa State University, Ames

Howard, Lyle P.
Lyle P. Howard, Architect, Ottumwa

Hueholt, Raymond L.
*3700 Park Ave., Des Moines
Smith-Voorhees-Jensen,
Architects Associated

Hukill, William V. Jr.
*2111 Greenwood Dr. S.E.,
Cedar Rapids
Kohlmann-Eckman-Hukill

Huneke, Ervin C.
*RR No. 2 Box 2, Fairfield
Ervin C. Huneke,
Architect & Engineer

Hunter, Carl John
*413 38th St. Place, Des Moines

Jensen, Myron E.
*4118 Maryland Pike, Des Moines
Smith-Voorhees-Jensen,
Architects Associated

Johnson, Robert Lewis M.
*1508 Lazy Lane, Clinton
Prout, Mugasis, and Johnson

Kendall, R. Kenneth
Griffith-Kendall
Des Moines

Keninger, Bernard J.
*503 West 9th St., Spencer
Walsh, Keninger & Galvin

Kocimski, Karol Jan
*416 Ash Ave., Ames
Dept. of Architecture
Iowa State University, Ames

Kohlmann, Ellsworth F.
*440 32nd St. SE, Cedar Rapids
Kohlmann-Eckmann-Hukill

Kramer, Donovan Dale
*1150 Victoria, Dubuque
Durrant, Deininger, Dommer,
Kramer, Gordon

Laffan, William J.
*2715 E. Hayes St., Davenport
Stewart-Robinson-Laffan

Langohr, E. Lawrence
Dane D. Morgan & Associates
Burlington

Lighter, Clyde W.
*333 45th St., Des Moines
Tinsley, Higgins, Lighter and Lyon

Lind, John Howard
*Rt. No. 1, 10 Highview Knoll,
Iowa City
Hansen-Lind-Meyer

Lindgren, Arthur A.
*4058 41st St., Des Moines
Lindgren & Taylor

Lobe, Horst W.
*3125 SW 23rd, Des Moines
Univ. of Wisconsin, Madison, Wisc.

Locke, John P.
*1404 Watrous, Des Moines
Charles Herbert & Associates

Keith E. Lorenzen, Architect
*443 4th Ave. SE
107 5th Ave. SE,
Blooming Prairie, Minn.

Lundblad, Glenn E.
Smith-Voorhees-Jensen,
Architects Associated, Sioux City

Lynch, James A.
James Lynch & Associates,
Des Moines

Lyon, R. Wayne
*5830 Windsor Dr., Des Moines
Tinsley, Higgins, Lighter & Lyon

McConnell, Richard D.
*4266 Fox Meadow Dr. SE,
Cedar Rapids
Crites & McConnell

McGinn, Donald P.
*865 Kirkwood, Dubuque
Donald P. McGinn, Architect

McIntosh, Robert D.
*1308 Pierce St., Sioux City
Foss-Engelstad-Foss

McKeown, Donald I.
*326 Hickory Dr., Ames
Dept. of Architecture
Iowa State University, Ames

McKlveen, John H.
*400 SW 42nd St., Des Moines
Wetherell-Harrison-Wagner

Maiwurm, Donald J.
*1649 No. 23rd St. Ft. Dodge
Maiwurm-Wiegman

Marquart, Gail E.
*Apt. No. 4D, 1800 Watrous,
Des Moines
Wetherell-Harrison-Wagner

Martin, William L.
*228 Linn, Boone
William L. Martin, Architect

Meehan, William R.
*330 52nd St., Des Moines
Wm. R. Meehan, Architect

*indicates home address

Firm addresses on Page 23

Meyer, Carl D. Jr.
*18 Leamer Ct., Iowa City
Hansen-Lind-Meyer

Miller, Gerald H.
*1718 Middle Rd., Bettendorf
Bracke-Hayes-Miller, Architects
Moline, Illinois

Morgan, Dane D.
*2500 Charette Lane, Burlington
Dane D. Morgan and Associates

Mugasis, Alexander P.
*904 Pleasant Valley, Clinton
Prout, Mugasis and Johnson

Neilsen, William N.
*1706 36th St., Des Moines
William Nielsen, Architect-Engineer

Normile, John
3213 Grand Ave. Apt. 32,
Des Moines
John Normile, Architect

Olson, Roger M.
*701 Center Ave., Decorah
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O'Neil, Eugene C.
*6510 Sunset Terrace, Des Moines
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Patten, Lawton M.
*2022 Friley Road, Ames
Dept. of Architecture
Iowa State University, Ames

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*5215 Ovid Ave., Des Moines
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*4330 Eaglemere Ct., Cedar Rapids
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Porter, Thomas C.
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*3921 Muskogee, Des Moines
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Marshalltown
Russell J. Prescott, Architect

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*930 2nd Ave. So., Clinton
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Prusinger, Lawrence A.
6523 Ridge Circle
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*6425 Center, Des Moines
Dougher-Frevert-Ramsey

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*2140 Glass Rd. NE, Cedar Rapids
Crites & McConnell

Reed, Raymond
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Dept. of Architecture
Iowa State University, Ames

Rice, John S.
*1001 6th, West Des Moines
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419 Pearson, Ames
Dept. of Architecture
Iowa State University, Ames

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Shane, Herbert T.
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Soenke & Wayland, Architects

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Steffen & Stoltz

Stenson, Marvin L.
*2178 Grand Blvd., Waterloo
Toenjes, Stenson and Warm

Stevens, Wayne Townsend
*1104 So. Marshall St., Rock Rapids
DeWild, Grant, Reckert and Stevens

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*4210 Rodeo Rd., Davenport
Stewart-Robison-Laffan

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Waggoner, Karl M.
*1054 2nd NW, Mason City
Waggoner and Waggoner

Wagonner, Thomas Marshall
*32 Lakeview Court, Mason City
Waggoner & Waggoner

*indicates home address

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Wagner, William J.
*RFD No. 1, Dallas Center
Wetherell-Harrison-Wagner

Wallerstedt, W. Kenneth
*6916 Holcomb, Des Moines
Smith-Voorhees-Jensen,
Architects Associated

Walsh, James H.
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*2011 Pleasant Dr., Cedar Falls
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Wehner, Roland C
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Werner, Marvin E.
*714 Woodlawn, Muscatine
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Wetherell, Edwin H.
*328 38th St. Pl., Des Moines
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Davenport

Wiegman, John H.
*1851 9th Ave. No., Ft. Dodge
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*1431 Cook Rd., West Des Moines
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PROFESSIONAL ASSOCIATES

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De Kovic, Charles William Jr.
*709 Douglas, Ames
Dept. of Architecture
Iowa State University, Ames

Drey, John E.
*1905 75th St., Des Moines
Dougher-Frevert-Ramsey

Dutt, Charles A. (retired)
*4007 Ovid Ave., Des Moines

Enzmann, Herbert K.
*7071 NW 88th Place, Grimes
Smith-Voorhees-Jensen,
Architects Associated

Hestness, Adrian C.
*2027 White Way, Cedar Falls
Johnson-Jamerson Associates

Higgins, Tom G.
*1005 40th St., Des Moines
Tinsley, Higgins, Lighter and Lyon

Jamerson, Robert H.
*1020 Franklin, Cedar Falls
Johnson-Jamerson Associates

Jordison, Richard R.
*2410 Friendship St., Iowa City
University of Iowa, Gilmore Hall

King, Pierce E.
*932 State St., Bettendorf
Charles Richardson & Associates

Kugler, Oskar
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Gjelten & Schellberg

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*1716 Second Ave. SE, Cedar Rapids
G. Richard McGinn, Architect

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Durrant-Deininger-Dommer-
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Skinner, Sammy Lee
*1734 18th St., Bettendorf
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Slater, Bernard J.
*601 Hayward, Ames
Dept. of Architecture
Iowa State University, Ames

Wagner, Kenneth A.
Kenneth A. Wagner, Architect

Walden, Brock A.
*1702 Walnut, Cedar Falls
Thorson-Brom-Broshar

Wirkler, Norman E.
*711 Laurel Park Rd., Dubuque
Durrant-Deininger-Dommer-
Kramer-Gordon

Zarnikow, Werner E.
*1204 18th St., West Des Moines
Smith-Voorhees-Jensen,
Architects Associated

ASSOCIATES

Allsup, Edgar Dean
*315 N. 3rd, Oskalooosa
Smith-Voorhees-Jensen,
Architects Associated

Anderson, Robert J.
*2015 W. 5th St., Sioux City
Foss-Englestad-Foss

Appell, Donald W.
*7215 Bellaire, Des Moines
Peterson and Appell,
Structural Engineers

Belknap, Dale D.
*2731 60th St., Des Moines
Smith-Voorhees-Jensen,
Architects Associated

Benz, John D.
*1407 Tomahawk Dr., Dubuque
Durrant-Deininger-Dommer-
Kramer-Gordon

Bernabe, Richard O.
*1916 Vine, West Des Moines
Smith-Voorhees-Jensen,
Architects Associated

*indicates home address

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Blackledge, Leland D.
*1353 60th St., Des Moines
Dougher-Frevert-Ramsey

Bloodgood, John D.
*4114 Greenwood Dr., Des Moines
Meredith Publishing Co.

Bock, Carl V.
*508 30th St., SE, Cedar Rapids
Brown, Healey & Bock

Boetel, Patricia M.
*2219 56th St., Des Moines
Karl Keffer Associates

Bossenberger, William H.
*4411 Hickman Rd., Des Moines
Brooks-Borg

Bullington, Harold J.
*3100 S.W. 14th St., Des Moines
Karl Keffer Associates

Campbell, Royce
*1781 Flower St., Waterloo
Gross-Mulford, Inc.

Carlson, Keith M.
*2822 Neola, Cedar Falls
Toenjes, Stenson and Warm

Carney, Leo A.
*4128 33rd St., Des Moines
Wetherell-Harrison-Wagner

Carson, Jack
*2323 Thornton Dr., Des Moines
Jack Carson, Consulting Engr.

Champion, James
*6510 NE 22nd St., Ankeny
James Lynch & Associates

Coon, Kenneth V.
*911 40th St., Des Moines
Brooks-Borg

Dale, William C., Jr.
*1602 Cherry Hills Rd., Bettendorf
Stewart-Robison-Laffan

Den Hartog, Eugene E.
*3817 Lanewood Dr., Des Moines
Tinsley, Higgins, Lighter & Lyon

Dicken, David M.
*1844 "A" Ave. N.E., Cedar Rapids
Brown, Healey & Bock

Esch, Delmer D.
*217 S.W. 39th St.
Woodburn & O'Neil

Freitag, Maurice E.
*6842 University Ave., Des Moines
Brooks-Borg

Fudge, Wm. R.
*529 25th St., West Des Moines
Lindgren & Taylor, Architects

Gehrls, G. Hugo
*4318 Lincoln Ave., Des Moines
Smith-Voorhees-Jensen,
Architects Associated

Griffith, M. Bruce
*2513 Meadow Lane,
West Des Moines
Savage and Ver Ploeg

Hall, Harold C.
Shive-Hall-Hattery
511 Iowa Ave., Iowa City

Hammond, Arthur E.
*3525 62nd St., Des Moines
Tinsley, Higgins, Lighter & Lyon

Hansen, Charles
*2815 Gilmore, Des Moines
Hansen Planning Consulting

Harmeyer, R. J.
*2205 36th St., Des Moines
Woodburn & O'Neil

Harrison, Arthur L.
*515 24th St., Ames
Harrison & Rippel

Hart, J. Jr.
*620 40th, Des Moines
Brooks-Borg

Huntley, Jack Clyde
*Route No. 2, Waterloo
Toenjes, Stenson & Warm

Ingebretson, Thomas
*5016 Ingersoll, Des Moines
Prall Architects Engineers

Johnson, Donald Alvin
*3707 37th St., Des Moines
Smith-Voorhees-Jensen,
Architects Associated

Johnson, Robert L.
*305 East "A" St., Forest City
Gjelten & Schellberg

Jones, Daniel C.
*117 Joder, Waterloo
Thorson-Brom-Broshar

Kludas, James Ralph
*2308 Fairlawn Dr.,
West Des Moines
Smith-Voorhees-Jensen,
Architects Associated

Lamond, Charles O.
*820 Circle Dr., Carlisle
Federal Housing Admin. 7th & Park
Des Moines

Larson, Jerome W.
*1067 46th St., Des Moines
Northwestern Bell Telephone Co.

Luethie, Donald H.
*5358 Northwest Blvd., Davenport
Charles Richardson & Associates

McMullin, Richard N.
*807 31st St., Des Moines
Prall Architects Engineers

Mahan, William T.
*682 36th St., Des Moines
Charles Herbert & Associates

Mathieu, Robert J.
*110 Arden, Waterloo
Thorson-Brom-Broshar

Moore, Larry R.
*431 Cottage Grove, Cedar Rapids
Moore & Bouse, Consulting Engrs.

Munzenmaier, Edward W.
*1201 Oak Park, Des Moines
Savage and Ver Ploeg

Nasr, A. (Raymond)
*306 29th St., Des Moines
Griffith-Kendall

Nigro, Anthony J.
*700 Boulder Ave., Des Moines
Griffith-Kendall

Olson, Clarence L.
*918 35th St., Des Moines
Prall Architects Engineers

Olson, Eugene A.
*3611 Cheyenne Blvd., Sioux City
William L. Beuttler, Architect

Osborn, William L.
*1931 N. Nevada, Davenport
Soenke & Wayland

Overton, Charles T.
*3710 3rd St., Des Moines
Brooks-Borg

Paxton, James Alfred
*3931 Lincoln Pl. Dr., Des Moines
Porter/Brierly Associates

Peterson, Carlyle W.
*4201 Beavercrest Dr., Des Moines
Peterson and Appell,
Structural Engineers

*indicates home address

Firm addresses on Page 23

Petre, George M.
*3909 Madison Ave., Des Moines
William R. Meehan

Polujan, Rouald K.
*1400 2nd Ave. SE, Cedar Rapids
Crites & McConnell

Pulley, Frank L.
*6815 Reite Ave., Des Moines
Frank Pulley, Consulting Engineer

Ralston, Donald E. Jr.
*1612 Market St., Burlington
Robin B. Carswell

Ratcliffe, John R.
*2100 30th St., Des Moines
Brooks-Borg

Rieniets, James H.
*1341 Harold Dr. SE, Cedar Rapids
Crites & McConnell

Rietz, Paul W.
*2917 Grand Ave., Des Moines
Brooks-Borg

Sauer, Edward G.
*1500 Oakland, Apt. 510,
Cedar Rapids
Crites & McConnell

Schilling, Ralph R.
*4311 Woodland Ave,. Des Moines
Stevenson-Flanagan-Schilling

Schmitt, Walter J.
*2336 23rd St. SW, Mason City
Bergland & Bianco

Seaboch, Donovan Lee
*625 Irving, Apt. No. 5
Des Moines

Shirk, Keith E.
*841 63rd St., Des Moines
Tinsley, Higgins, Lighter & Lyon

Shuck, Terry
*321 Tonawanda Dr., Des Moines
Prall Architects Engineers

Snedden, Donald E.
*504 W. Lawn Dr., Ankeny
Savage and Ver Ploeg

Snyder, Wayne J.
*806 Clay St., Cedar Falls
Thorson-Brom-Broshar

Stevenson, Daniel B.
*631 42nd St., Des Moines
Stevenson-Flanagan-Schilling

*indicates home address

Stone, Robert B.
*1524 Robeson, Ave., Bettendorf
Charles Richardson & Associates

Stouffer, Scott
*1320 41st St., Des Moines
Charles Herbert & Associates

Sundquist, Herbert E.
*730 S. 12th, Clinton
Prout, Mugasis and Johnson

Swanson, Byrl E.
*1319 Anthony Pl., Box 306
Camanche
Louis C. Kingscott & Associates

Ugulini, Aldo Edward
*1221 W. 3rd St., Ankeny
Smith-Voorhees-Jensen,
Architects Associated

Vander Linden, Charles, Jr.
*2904 34th St., Des Moines
Vander Linden & Dennis

VandeVenter, Robert L.
*928 13th, West Des Moines
Savage and Ver Ploeg

Walters, Paul A.
*526 Glenview Dr., Des Moines
Paul A. Walters, Cons. Engr.

Weber, Delano B.
*1126 Locke Ave., Waterloo
Thorson-Brom-Borshar

Whitmarsh, Wayne
*2122 Virgil St., Cedar Falls
Robert C. DeVoe, Architect

Whitmer, Wayne M.
*1826 8th Ave. SW, Cedar Rapids

Wilkins, James W.
*4000 Fagan Dr., Des Moines
Charles Herbert & Associates

HONORARY ASSOCIATES

Kirsch, Dwight
*1701 Casady Drive, Des Moines

FIRMS

Altfillisch, Olson, Gray & Thompson
126-½ W. Water St.
Decorah, Iowa

Herbert A. Arthur
310-½ Main St.
Ames, Iowa

Firm addresses on Page 23

Bergland & Bianco
11½ S. Federal Ave.
Mason City, Iowa

Brooks-Borg
815 Hubbell Bldg.
Des Moines, Iowa 50309

Brown, Healey & Bock
131 36th St. Dr., SE
Cedar Rapids, Iowa 52402

I. T. Carrithers, Architect
6 N. 1st Street
Council Bluffs, Iowa

Robin B. Carswell
Suite 308 F. & M. Bank Bldg.
Burlington, Iowa 52601

R. M. & M. B. Cleveland
424 E. 4th Street
Waterloo, Iowa 50703

Louis C. Couch, Architect
207 Plaza Bldg.
Bettendorf, Iowa

G. B. Cox, Architect
1630 State Street
Bettendorf, Iowa

Architects Crites & McConnell
1953 1st Ave., SE
Cedar Rapids, Iowa

Deuth & Gibson
417 Logan St.
Waterloo, Iowa

Robert C. DeVoe, Architect
311-B Main Street
Cedar Falls, Iowa

Dewild, Grant, Reckert & Stevens
301½ Main Street
Rock Rapids, Iowa

Dougher-Frevert-Ramsey
Professional Center
3839 Merle Hay Rd.,
Des Moines, Iowa 50310

James M. Duffy, Architect
208 Security Bldg.
Sioux City, Iowa

Durrant-Deininger-Dommer-
Kramer-Gordon
1122 Rockdale Road
Dubuque, Iowa 52003

Amos Emery & Associates
820 Locust Street
Des Moines, Iowa 50309

Phil H. Feddersen, Architect
818 No. Second St.
Clinton, Iowa

Foss-Engelstad-Foss, Architects
1308 Pierce
Sioux City 5, Iowa

Gjelten & Schellberg
205 S. Clark St.
Forest City, Iowa

The Griffith Co.
P.O. Box 917
S. Kenyon Road
Fort Dodge, Iowa

Griffith-Kendall Architects
3810 Ingersoll
Des Moines, Iowa 50312

Hansen-Lind-Meyer
14 South Linn St.
Iowa City, Iowa 52240

Robert D. Hecker
407 Toy Nat'l Bank Bldg.
Sioux City, Iowa

Charles Herbert & Associates
709 Bankers Trust Building
Des Moines, Iowa 50309

Hollis & Miller
120 Council Bluffs Sav. Bank Bldg.
Council Bluffs, Iowa

Lyle P. Howard
208 Kresge Building
Ottumwa, Iowa

Ervin C. Huneke, Architect
First National Bank Bldg.
Fairfield, Iowa

Johnson-Jamerson Associates
2417 Main Street
Cedar Falls, Iowa 50613

Louis C. Kingscott & Associates
321 W. Kimberly
Davenport, Iowa 52806

Karl Keffer Associates
208 Masonic Temple Buldg.
Des Moines, Iowa 50309

Kohlman-Eckman-Hukill
610 Tenth St., SE
Cedar Rapids, Iowa

Lindgren & Taylor
6311 Hickman Road
Des Moines, Iowa 50322

James Lynch & Associates
314 Savings & Loan Bldg.
Des Moines, Iowa 50309

Maiwurm-Wiegman
Warden Building
Fort Dodge, Iowa

William L. Martin, Architect
821 15th Street
Boone, Iowa

Donald P. McGinn, Architect
740 Fischer Bldg.
Dubuque, Iowa

G. Richard McGinn, Architect
1716 Second Ave. SE
Cedar Rapids, Iowa 52403

William R. Meehan, Architect
2215 Grand Ave.
Des Moines, Iowa 50312

Dane D. Morgan & Associates
314 N. 4th
Burlington, Iowa

William Nielsen, Architect
307 Masonic Temple Bldg.
Des Moines, Iowa 50309

John Normile, Architect
3213 Grand Ave. Apt. 32
Des Moines, Iowa

Leo C. Peiffer, Architect
4330 Eaglemere Ct. SE
Cedar Rapids, Iowa 52403

Porter/Brierly Associates
707 Insurance Exc. Bldg.
Des Moines, Iowa 50309

Powers & Associates
PO Box 368
Iowa City, Iowa

Prall Architects Engineers
4717 Grand Ave.
Des Moines, Iowa 50312

Russell J. Prescott, Architect
126½ W. Main St.
Marshalltown, Iowa

Prout, Mugasis and Johnson
709 5th Ave. S.
Clinton, Iowa
Charles Richardson & Associates

1001 Kahl Building
3rd & Ripley Streets
Davenport, Iowa 52801

George Russell, Architect
1221 Savings & Loan Bldg.
Des Moines, Iowa 50309

Savage & Ver Ploeg
1200 Grand
West Des Moines, Iowa

Smith-Voorhees-Jensen
Architects Associated
1040 5th
Des Moines, Iowa 50314
and
424 Badgerow Building
Sioux City, Iowa

Soenke & Wayland, Architects
601 Brady St.
Davenport, Iowa

Steffen & Stoltz
Box 601
Ottumwa, Iowa

Stewart, Robison & Laffan
Priester Building
601 Brady Street
Davenport, Iowa

Thorson-Brom-Broshar
219 Waterloo Bldg.
Waterloo, Iowa

Tinsley, Higgins, Lighter & Lyon
526 Liberty Building
Des Moines, Iowa 50309

Toenjes, Stenson & Warm
3404 Midway Drive
Waterloo, Iowa 50701

Kenneth A. Wagner, Architect
605 Union Arcade
Davenport, Iowa

Waggoner & Waggoner
15 S. Federal Avenue
Mason City, Iowa 50401

Walsh, Keninger & Galvin
Glass Block Building
Box 467
Spencer, Iowa 51301

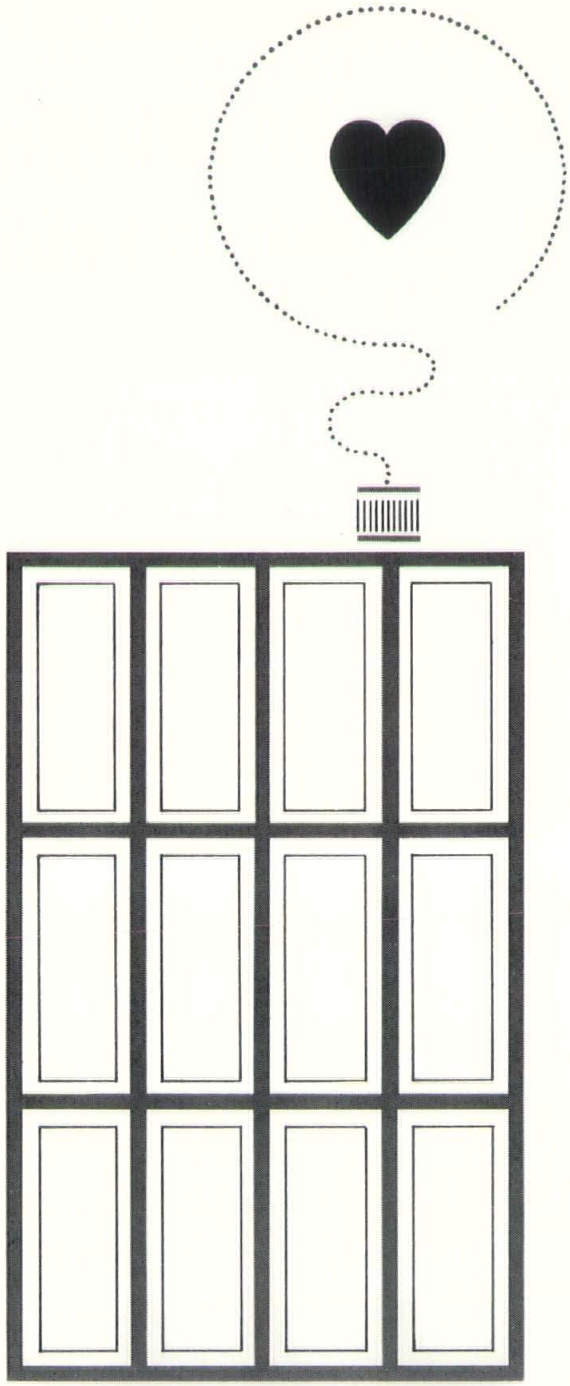
Wehner and Henry
114 E. Prentiss St.
Iowa City, Iowa 52240

Wetherell-Harrison-Wagner
500 Hubbell Bldg.
Des Moines, Iowa 50309

Raymond Whitaker, Architect
1202 Adams Street
Davenport, Iowa

Winkler-Goewey, Architects
502 Shops Building
Des Moines, Iowa 50309

Woodburn & O'Neil
201 Jewell Building
Des Moines, Iowa 50309



Award of Merit

BROWN, HEALEY, AND BOCK, ARCHITECTS
Cedar Rapids

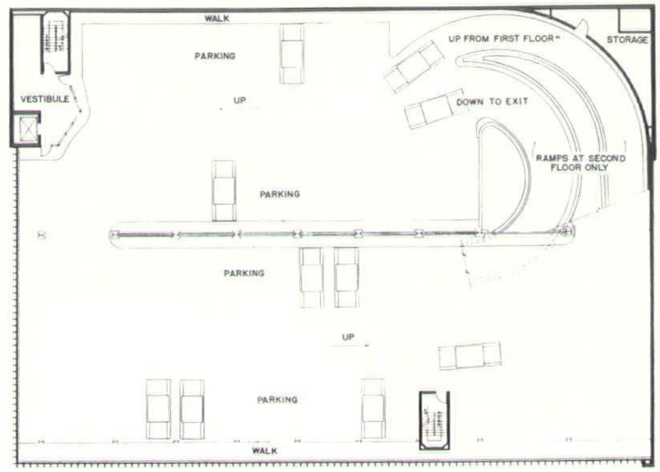
MERCHANTS NATIONAL DRIVE-THROUGH BANK AND MULTI-LEVEL PARKING FACILITY
Cedar Rapids

Jury Comment:

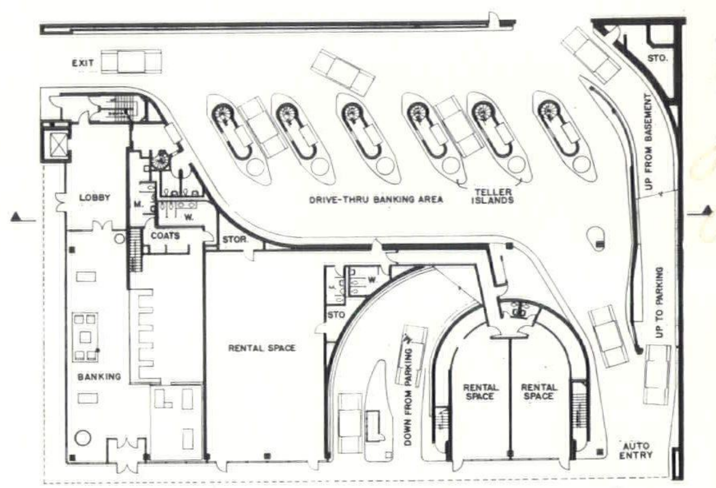
Solution of Problem
A straightforward solution to two needs of today's auto traffic: compact center-city parking and drive-in banking. Most of the ground banking level is commendably left to the pedestrians, with the parking levels entirely off the street.

Architectural Excellence
A good example of the background architecture needed to house what should be background functions, especially parking. The architectural excellence of this project is as much in the logical answers to the problems at hand as it is in the restrained handling of massing and materials.

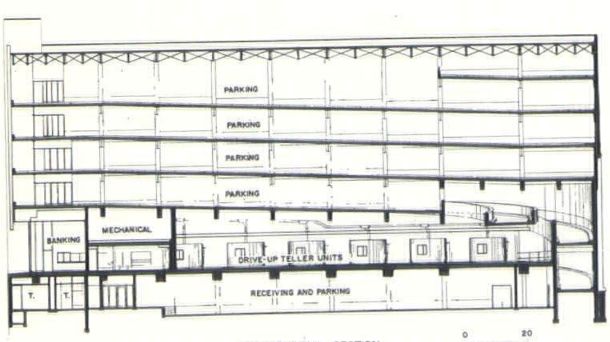
Relation of Building to Site
The grille-enclosed parking floors are connected to the ground by large glass areas enclosing the bank. The parked cars are screened from view above while the pedestrian level is left open and inviting.



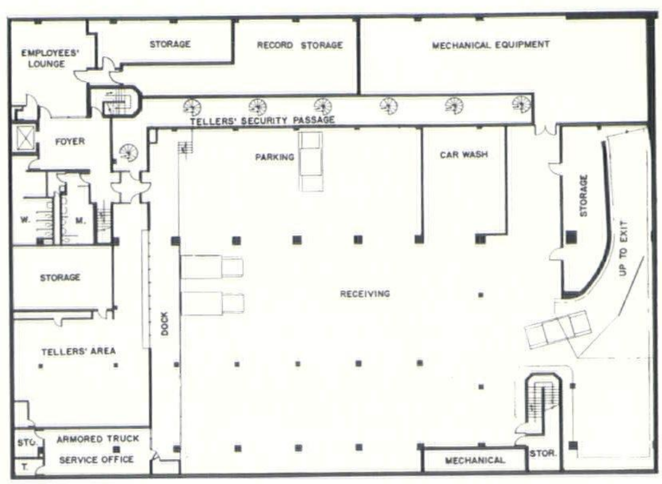
TYPICAL FLOOR PLAN 0 20



FIRST FLOOR PLAN 0 20



LONGITUDINAL SECTION 0 20



BASEMENT FLOOR PLAN 0 20

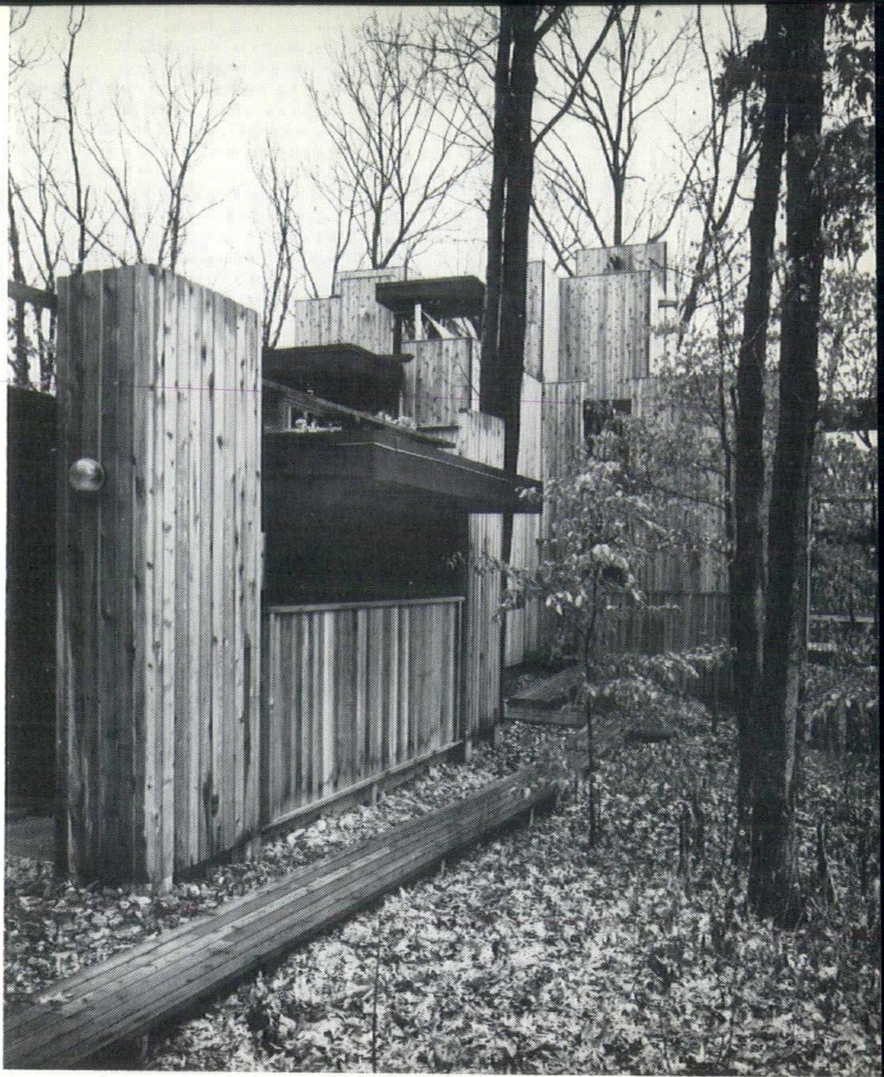
Architect's Comment:

The family consists of mother, father and three children; a daughter 11, a son 5 and a daughter 4. It was desired to construct an environment for family living and entertaining with adequate space at a modest cost.

The site is located in a heavily wooded area adjacent to the east edge of Cedar Rapids, Iowa. This area of 22 acres has been sub-divided into 17 sites and all homes will be designed and coordinated by one firm. The house was conceived in a fashion to provide privacy for both children and adults in the sleeping area of the house and also to provide space for independent and simultaneous activities for both adults and children. The character of the timber is quite vertical and the house has been conceived to provide both view and an opportunity for exterior living at various levels within the trees. Framing is basically exposed fir joists with 2" T & G fir floor and roof deck which forms the finished floor in the bedroom areas.

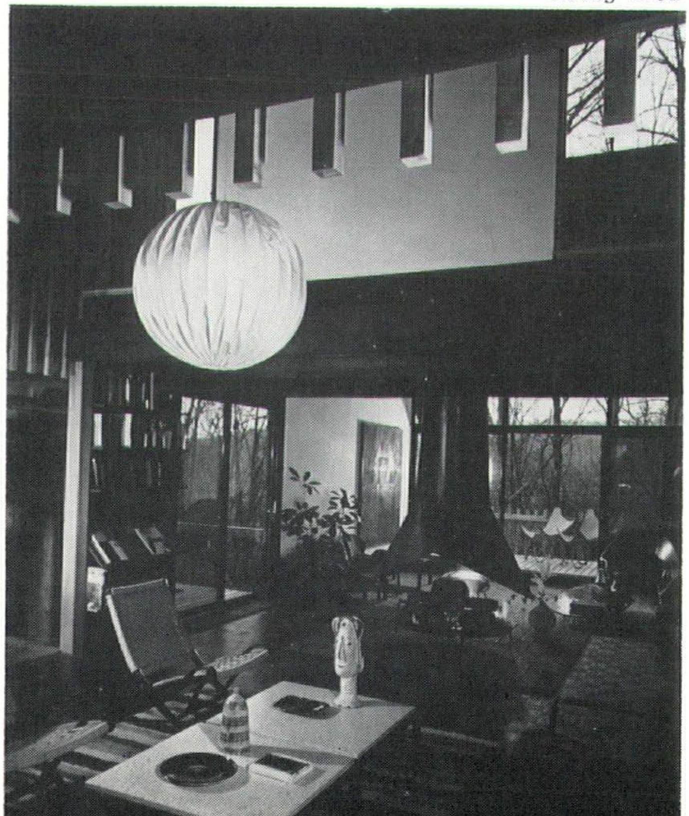
The exterior is rough sawn cedar vertical boards and the interior is smooth T & G cedar applied vertically. Colors are predominantly neutral so that the changing light and color of the forest will dominate the interior.

JULIUS SHULMAN



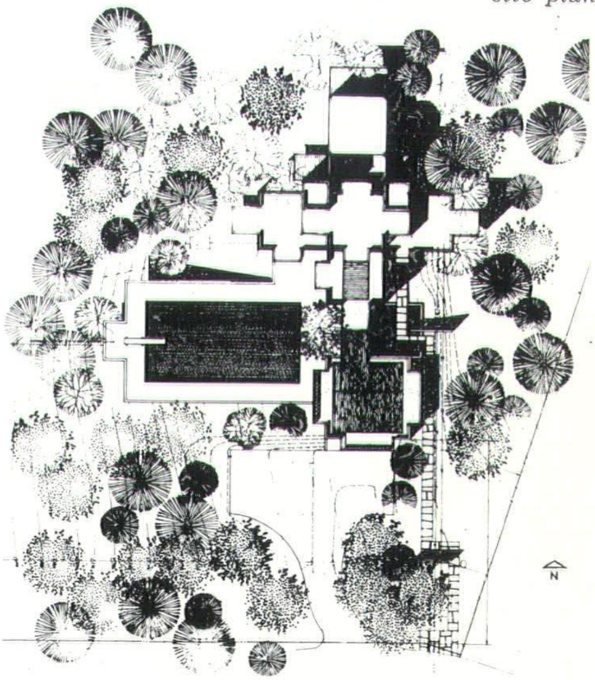
approach to entry

living area



JULIUS SCHULMAN

site plan



First Honor Award

CRITES AND McCONNELL, ARCHITECTS
Cedar Rapids

Jury comments are by Mr. Victor Christ-Janer, A.I.A., Chairman of Design of the School of Architecture, Columbia University, and a jury of his selection.

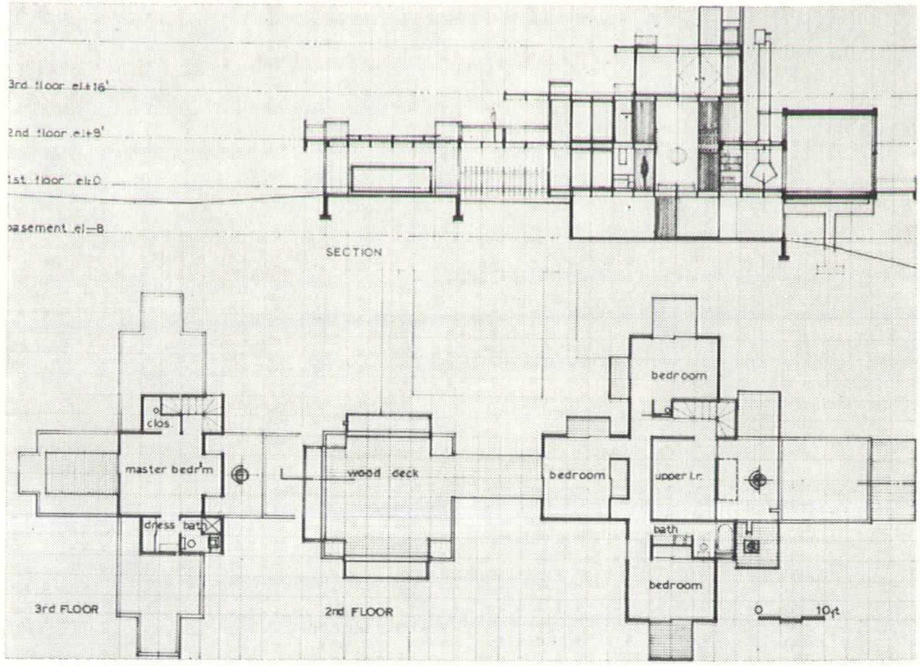
CRITES RESIDENCE
Cedar Rapids

Jury Comment:

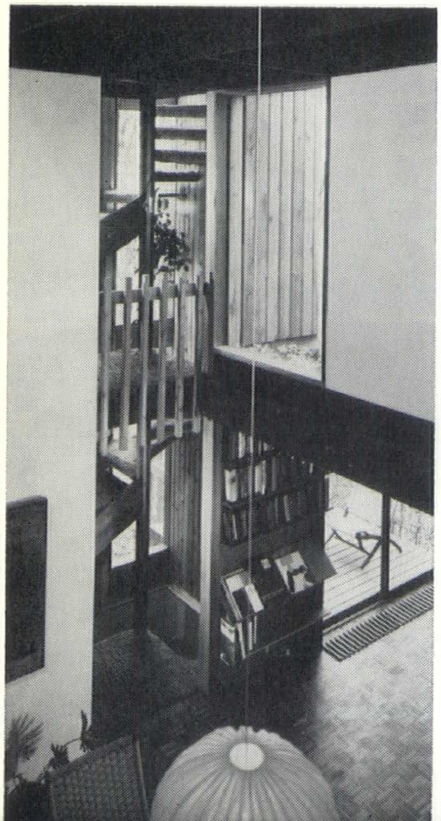
Solution of Problem
Problem of housing a large family in individual privacy very well solved. Family unity nonetheless preserved by the large living zone penetrating vertically and horizontally, around which the private spaces are grouped.

Relation of Building to Site
The random aesthetic reflects a natural setting, giving feeling that house is part of nature.

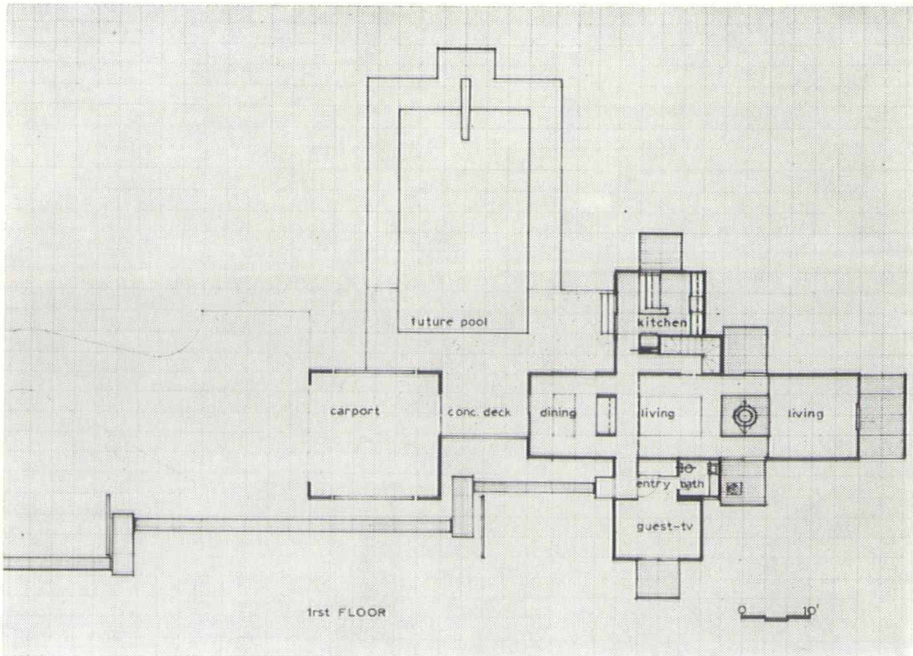
Architectural Excellence
Excitement achieved out of the order of spaces themselves, expressed with simple materials used in a fresh aesthetic. Massing and sequence carefully controlled but uncontrived.

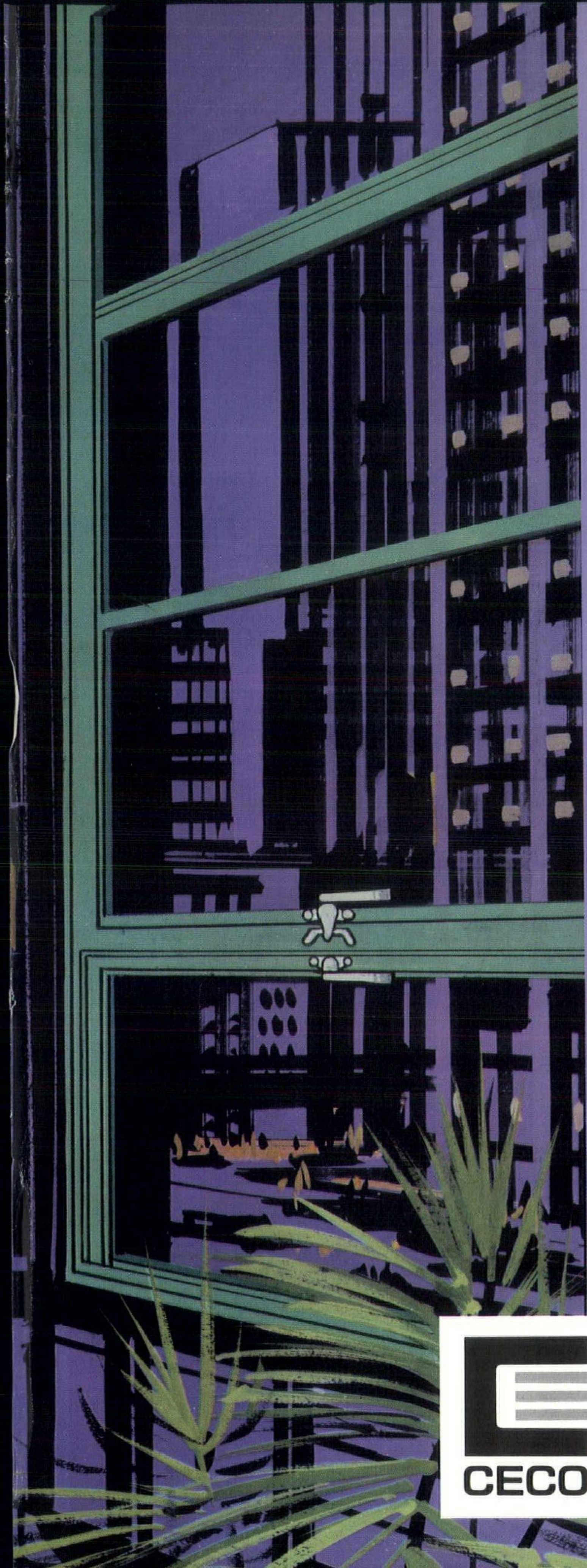


stair



JULIUS SCHULMAN





exterior

ROBERT FRENCH

Architect's Comment:

The problem was to provide seven drive-up banking windows, some walk-up banking facilities, an armored vehicle loading area, parking space for bank cars, and parking space for the office tennants and customers of the parent bank building across the street.

The solution provides drive-through, walk-up banking and rental areas at street level. Armored vehicles load and unload in the basement onto a conveyor belt. The belt is in a tube built under the street and connects to the vault area of the bank across the street.

Upper floors are enclosed with soft gold anodized aluminum vertical fins and an open screening having a dark "Duranodic" finish.

Construction is now underway on a glass-walled, air conditioned connecting bridge over the street to provide direct access from the parking decks to the bank building.

Interior spaces were all designed by the architect.

banking room

ROBERT FRENCH

Award of Merit

BROOKS-BORG, ARCHITECTS
Des Moines

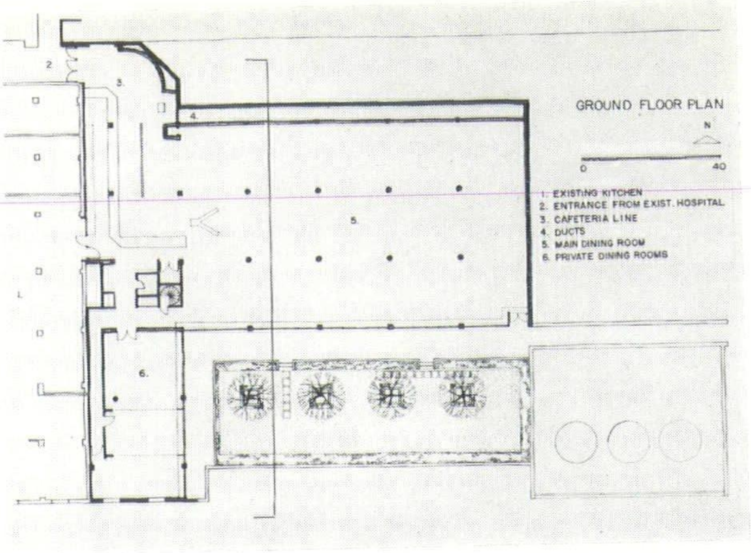
SERVICE ADDITION TO IOWA METHODIST HOSPITAL
Des Moines

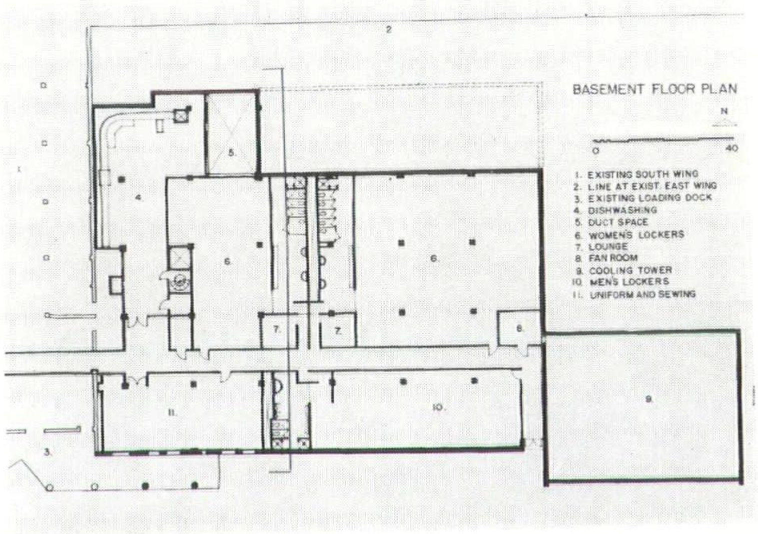
Jury Comment:

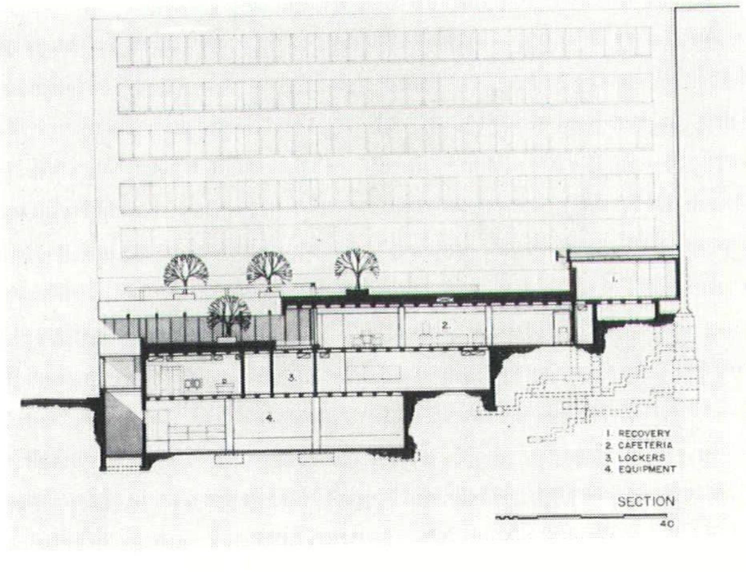
Relation of Building to Site
Dug into the ground, the building becomes part of the site, following existing contours and using the roofs as new land — in good contrast to the exisiting high building.

Solution of Problem
A good solution to the difficult environmental problem of providing open yet sequestered green area for convalescing patients in the central city.

Architectural excellence
Through bold forms, expressed only in glass, concrete and grass, the addition remains an entity separate from the main hospital, without seeming unrelated to it.



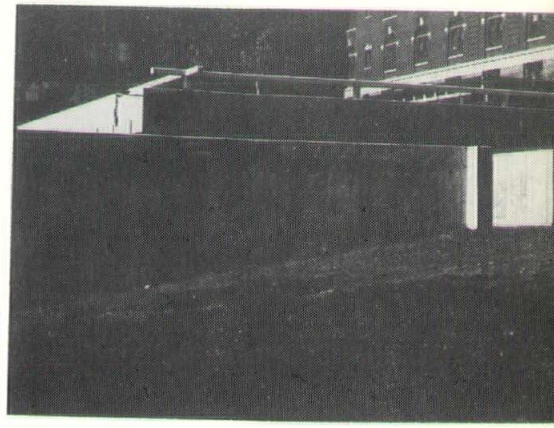




CARL HUNTER

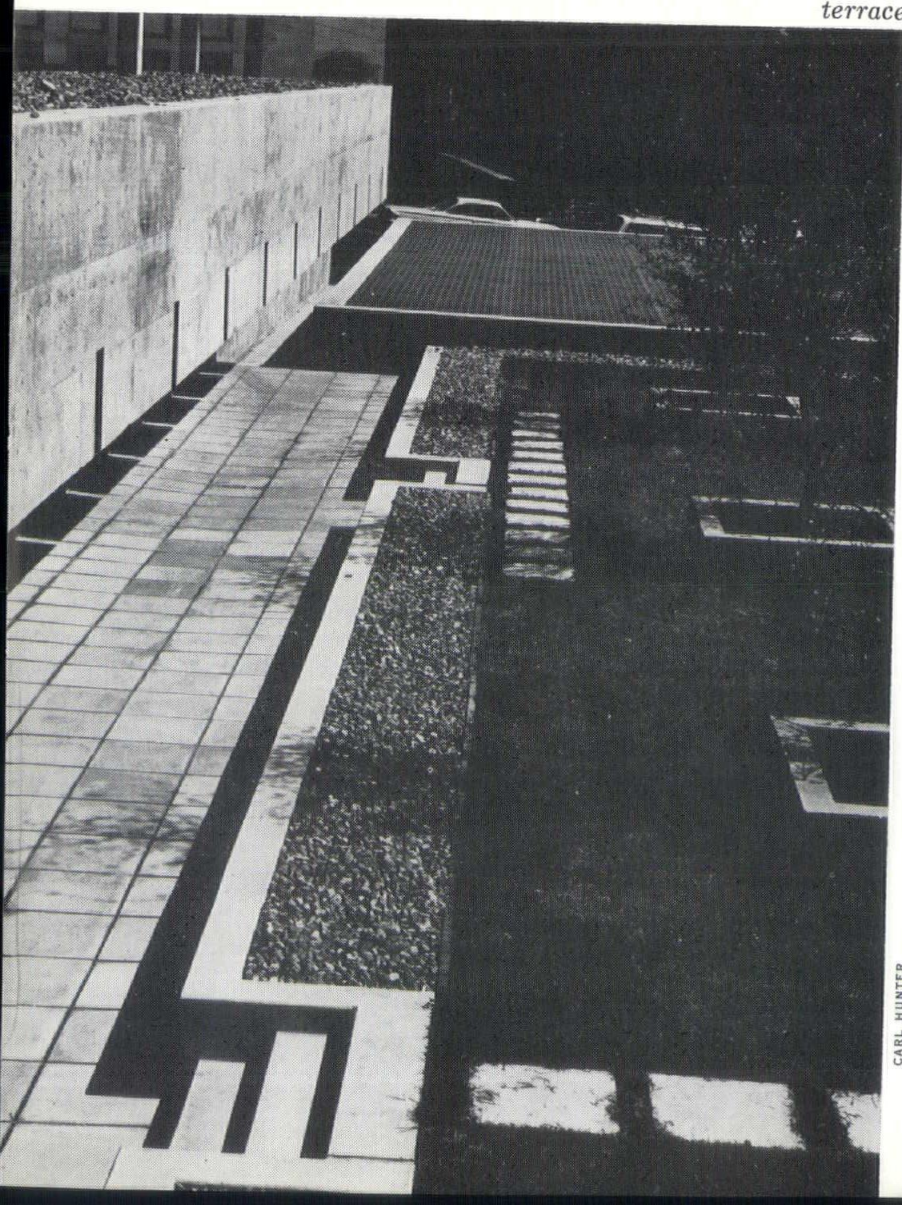
terrace

view from southeast



CARL HUNTER

terrace



CARL HUNTER

Architect's Comment:

This project, which is part of the Iowa Methodist Hospital's long range plan that leads ultimately to a 1,000 bed hospital, contains service facilities such as cafeteria, employee locker rooms, and central air conditioning equipment. The older buildings, to be systematically replaced or remodeled, are situated on a ridge of land overlooking downtown Des Moines with the majority of patient rooms opening on grassed and treed lawns. The prospect of losing these to large areas of roof over the service areas required at the base of the patient towers led naturally to a sod-covered terrace concept which was enthusiastically endorsed by the Hospital.

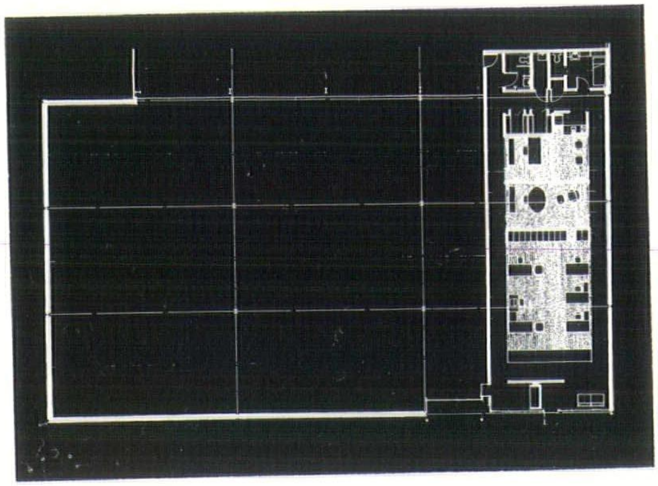
The plain no-nonsense concrete surfaces are a part of the long-term concept of a concrete plinth or platform socketed in the side hill out of which present and future towers will rise.

The boldly formed spandrels, exposed concrete waffle slabs, grass and gravel, and carefully selected trees are used to create a sequence of semi-urban spaces that are enjoyed equally from the patient rooms and from the staff and visitors in the cafeteria.

Award of Merit

CARL HUNTER AND RUSSELL PARKS, ARCHITECTS
Des Moines

OSMUNDSON MANUFACTURING COMPANY
Perry



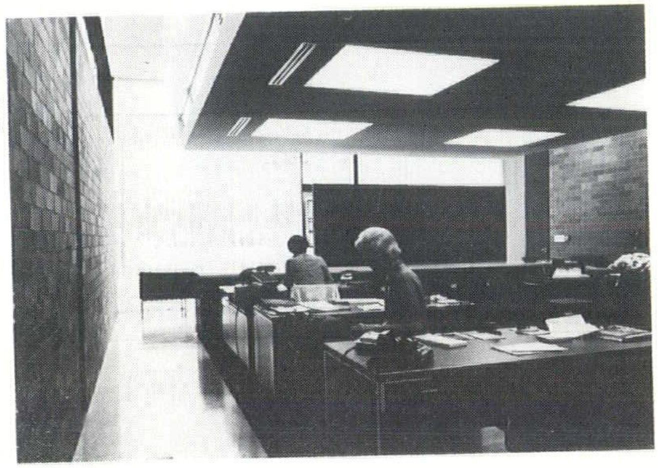
Jury Comment:

Relation of Building to Site
This building offers a quiet and refreshing change from side-of-the-highway non-architecture. A contribution to rather than a detraction from the landscape.

Solution of Problem
A good, logical solution to office and warehouse space for a small business which will someday require expanded facilities. Differentiation is made between office and warehouse inside and out, without destroying the unity of the total plant.

Architectural Excellence
It expresses only what it is: some office but mostly warehouse. Simplicity of massing, sophistication of detail, restrained handling of materials add up to a unified, dignified design.

CARL HUNTER



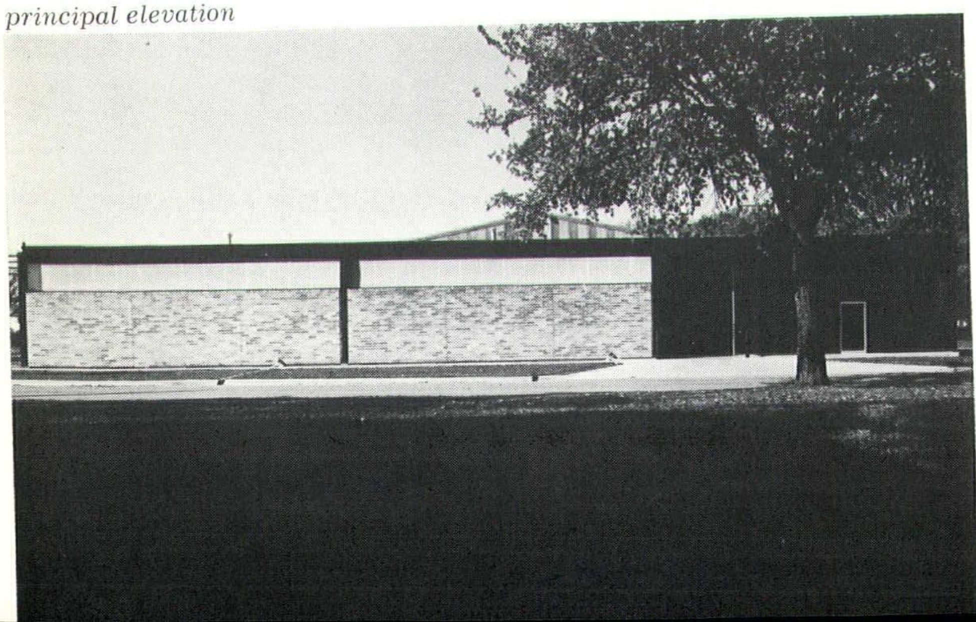
office area

Architect's Comment:

The problem was to provide new offices and additional warehouse space for an old established company manufacturing farm tillage tools and to consider a master plan for future redevelopment of the entire property. The site is bounded on three sides by streets in an industrial strip and on the fourth side by the rail line of the Milwaukee Road. Initial expansion was possible only to the east, adjacent to a steel pre-fabricated warehouse building. The next step in redevelopment will take place at the west end where the old offices will be demolished. Final redevelopment will take place in stages between these two ends of the property.

The offices are separated from the manufacturing plant by the warehouse space to diminish the noise problem and to provide direct control at the truck entrance for pick-up of finished goods.

principal elevation



CARL HUNIER

Does Iowa Need a State Bureau of Architecture?

The following article was prepared by the Iowa Chapter, American Institute of Architecsts.

A study of present and past experiences by state and Federal governments brings out two questions that directly concern the taxpaying public, as well as an indication of the answers:

1. *Q: Does a government-operated department of architecture reduce the cost of planning and construction?*

   A: All evidence examined to date indicates that it does not. The record actually shows that planning costs are increased substantially and in some cases are double in the same localities.
2. *Q: Are utility and beauty of government buildings improved through design by a government bureau of architecture rather than by private professionals?*

   A: No evidence to date shows this to be the case. But the evidence does reveal concern with standardized, stereotyped design—both by the taxpayers and by many in government, including the late President Kennedy. Through a 1962 policy statement concerning a Federal building program, Mr. Kennedy called for "designs that embody the finest contemporary American architectural thought."

PROCEDURES IN OTHER STATES

The last available survey of procedures used by the various states to procure public buildings was made in 1959. Information was gathered from 47 states and Puerto Rico and showed the following:

29 states engaged private architects for all work.

9 states use private architects for all but minor projects.

10 states maintain architectural bureaus but let out a considerable amount of work to private architects.

The same survey showed the following policies in the states surrounding Iowa: Illinois, Nebraska and South Dakota — all work by private architects. Missouri — all projects over $30,000 by private architects. Minnesota — all projects over $50,000 by private architects. Wisconsin — approximately half the work by private architects, half by the state.

WHAT WOULD IT COST?

Designing a building, preparing working drawings and specifications and supervising construction take manpower, materials and working space, whether the job is done by private or government-paid professionals. The private professional pays for these costs out of his fee, which is included in the total cost of the building project.The government bureau has the same expenses which also become part of the total project cost.

The experience of the Federal government and of states from which information is available indicates that expenses for government architectural services actually add more to building costs than do fees of private architects and engineers. One reason, as suggested by the evidence: The private professional's staff and facilities are available when needed but cost the state nothing when not in use. Overhead for a state bureau is continuous whether it has work to do or not. (In 1960, the California state architect testified that his bureau "came out of the war with a staff of a little under 100 and it has a staff now of a little over 900.")

HOW MUCH PROFIT IN A FEE?

The Texas Research League, in a current study commissioned by the Texas Legislature to find possible economies in state construction, reports that "there appears to be widespread misunderstanding about what is obtained for the (professional architect's) fee and apparently some belief that a very large part of it represents a profit to the architect." The League studied the fees and expenses of several successful Texas architects and reported that a $6,000 fee paid to an architect for designing a $100,000 state building would break down as follows:

Salaries, materials, supplies, printing and other outlays directly traceable to the specific job	*$2,400*
Fees paid to consultants—primarily structural electrical and/or mechanical engineers for work on specific job	*$1,224*
Overhead expenses (rent, telephone, insurance, clerical, etc.), plus $12,000 annual salary to principals in firm	*$1,746*
Profit before Federal taxes	*630*
TOTAL FEE	*$6,000*

FEDERAL EXPERIENCE WITH COSTS

The Hoover Commission, in surveys for its second report, discovered that the costs of engineering, design and inspection by government agencies ran up to 18% of estimated construction costs, as against an average of 4% for design and 4% for inspection ordinarily paid to private consulting firms which had to meet their entire overhead bills and pay full taxes.

After studying $8 billion in Federal Construction, the Hoover Commission reported:

"By contracting to private architect-engineer and construction organizations all phases of design and construction work . . . relatively small supervisory engineering organizations in the (government) executive agencies could furnish the control essential for all government projects, without maintaining complete engineering and construction staffs."

TEXAS

Following its study mentioned earlier, the Texas Research League prepared a bill for the Texas Legislature which would encourage an even closer working relationship between the private architect and engineer and the existing Texas State Building Commission. In a statement of the philosophy guiding the proposed legislation, the League said Texas "should place its primary reliance upon the private architect and engineer as the best means of meeting the building needs of the State."

IOWA

A survey of Iowa architects in February, 1965, asked for figures on work performed for the State in the last five years — in project costs, fees paid and actual profit before taxes.

Based on the responses of 11 firms which had done work for the state in that period, the average fee in Iowa was 3.96%. Pre-planning, supervision and inspection costs of the various government departments involved ranged from 1.1% to 2%. Even when these costs are added to the architect's fee the total cost in Iowa is below the government costs found by the Hoover Commission well over a decade ago, and below that of California's state bureau of architecture, which was over 7% in 1960.

The actual profit realized by the Iowa architect from his fee averaged only .66% of the total cost of the project.

WHAT EFFECT ON DESIGN?

During a controversy over a $28 million dormitory building program in California in 1960, a State Assemblyman told the state architect: "I don't like your buildings. I think they're senile, I think they're dead. . . . These things look like a combination between a park structure and a prison . . . I'm trying to find out here how we can utilize the great architects that we have in the State of California to design buildings."

This concern with the appearance of architecture by bureau is reflected in a Federal policy statement established by President Kennedy on May 31, 1960, with respect to a proposed $425 million program of Federal office building construction and a major redevelopment of Pennsylvania Avenue in the nation's capital. Here are excerpts from the statement:

"The policy shall be to provide requisite and adequate facilities in an architectural style and form which is distinguished and which will reflect the dignity, enterprise, vigor and stability of the American National Government.

"Major emphasis should be placed on the choice of designs that embody the finest contemporary American architect thought . . .

"The development of an official style must be avoided. Design must flow from the architectural profession to the Government, and not vice versa . . ."

CALHOON . . . continued from page 14

or of a soil deposit due to a structural load is a properly constructed stress profile. This profile will indicate the existing overburden stress; the alterations to the existing overburden stress which have been caused by the excavation of basements or the placement of fills over an area; the stress induced by structural footings; and the evaluation of any existing pre-consolidation stress in the soil mass. This stress analysis or profile is a prerequisite to the intelligent application of laboratory testing to the problem under consideration.

There is a popular misconception that a given soil at a given depth has a given bearing capacity. Quite often the soils engineer is requested to provide an allowable bearing capacity for a soil at a certain depth without any knowledge of the magnitude of the loads to be supported. To see why this is an ill-advised practice, let us examine Figure 2. Total Stress Curve "A" represents the total stress beneath a small footing which supports a light column with a unit soil pressure of about 6,000 pounds per square foot. It will be noted that the total stresses indicated by this curve are less than those indicated by the pre-consolidation stress curve in all three soil layers, and settlement should be negligible. Total Stress Curve "B" is for a heavy column with a large footing also with a 6,000 pound per square foot unit pressure. In this case, the total stresses indicated by this curve exceed those indicated by the pre-consolidation stress curve in layer 2 and this phenomenon would indicate the possibility of footing settlement of a magnitude that could not be tolerated. Total Stress Curve "C" is constructed for the same column load as Stress Curve "B", but with a larger footing having a unit soil pressure of about 3,000 pounds per square foot. Here the total stress is less than the pre-consolidation stress in all three layers and settlement should be nominal. From this illustration, it is evident that for the soil conditions which prevailed at this site, the allowable bearing capacity at the proposed footing elevation was 6,000 pounds per square foot for a light column, but only 3,000 pounds per square foot for a heavy column. Thus it is seen that it is not always possible to recommend an allowable bearing capacity at a given depth which can be used for column loads of any magnitude.

A second popular misconception is that the allowable bearing capacity of clay is equal to its unconfined compressive strength. It is true that the unconfined compressive strength can be used as the allowable bearing capacity, but in most cases this will result in an ultra-conservative design. The allowable bearing capacity is usually greater than the unconfined compressive strength, but it is necessary to perform laboratory consolidation tests to verify the amount by which he unconfined compressive strength can be exceeded without causing objectionable amounts of settlement. The relationships between the structural dead load, live load, permanent live load and the character of the live load will also influence the allowable bearing capacity.

A third popular misconception is that an allowable bearing capacity for cohesive soils can be reliably determined from the split-spoon sampler blow count. Figure 3 indicates the relationship between this blow count or "N" value and the unconfined compressive strength of the clay soils present at a number of actual projects. It will be seen that there is no meaningful relationship between the "N" value and the unconfined compressive strength. For example, if we use an "N" value of 10, the unconfined compressive strength can reasonably be assumed to vary somewhere between 0.25 and 4.0

or more tons per square foot. For an "N" value of 30 blows per foot, the unconfined compressive strength could be expected to vary from 0.5 to 10 or more tons per square foot. The use of the "N" value in cohesive soils to determine an allowable bearing capacity (or "presumptive" bearing capacity) is an unsafe method which completely overlooks the progress that has been made in the field of soil mechanics during the past forty years.

The bearing capacity of a soil, or its resistance to a shear failure, is a function of the shear strength of the soil. In sand and gravel and in cohesionless silt soils, shear resistance is provided by the frictional properties of the soil. In sands and gravels, the soils engineer usually evaluates the shear resistance by employing his knowledge of certain empirical relationships of this shear resistance to the split-spoon sampler blow count. In cohensionless silts, the triaxial shear test must be used to evaluate the frictional properties. In cohesive silt, silty clay and clay soil, the shear strength may be provided by cohesion, friction or a combination of both, depending on the properties of the particular soil and upon the manner in which failure is expected to occur. Under certain conditions, a given clay will provide shear resistance through cohesion alone, and in other circumstances, the same clay soil may provide shear resistance entirely through friction. These differences are dependent upon the manner of failure which might be expected, and it is the responsibility of the soils engineer to determine the probable manner of failure and the correct shear resistance of the soil.

In most cases, the shear strength of cohesive soils can be determined by the unconfined compressive strength test; however, there are certain special situations in which the triaxial shear test must be used. Again, it is the responsibility of the soils engineer to know which test is applicable. The performance of the unconfined compressive strength test has been relatively standardized, while the triaxial shear test can be performed in an infinite variety of ways, depending upon the manner in which the prototype is expected to fail. Since the proper performance of a triaxial shear test can become very complicated, it must be performed under the direct supervision of an experienced soils engineer.

If a pile foundation appears to be the best solution to a foundation problem, then the soils engineer must recommend the use of either friction piles, point bearing piles, or piles which obtain their support through a combination of friction and point bearing. The soils engineer should evaluate the manner in which a pile will obtain its supporting characteristics and should recommend the best types of piles to be used in a particular situation. He should also include his best estimate of the length of pile that will be necessary. There are static methods available for making a reasonable estimate of pile lengths during the subsurface investigation phase of the project.

Most architects are interested in the approximate cost for a complete subsurface investigation. As a very rough rule of thumb, it can be said that such investigations will vary in cost from approximately 0.1 per cent of the total project cost on large projects to 0.5 per cent or, in some cases, to as much as 1.0 per cent of the total project cost on smaller projects. For cost information beyond this, it is suggested that the architect consult a company which specializes in subsurface investigations to determine detailed costs for a specific project.

Sometimes the comment is made by a designer that a subsurface investigation is unnecessary for a structure which is being designed because the building next door appears to be in sound condition. Unfortunately, the original formation of soil deposits and the drying and weathering of soil deposits is by no means uniform, even over relatively small areas. The "building next door" may be supported upon hard silty clay glacial till, while the site which the designer is neglecting to investigate may have been an ancient erosion gully in the glacial till which has since been filled with windblown loess, water-deposited alluvium or man-made fill. The latter might consist of anything from loosely dumped clay to cinders, organic garbage or other rubbish. In addition there could easily be a layer of buried peat on the proposed site, and probably none of these aforementioned objectionable features would be visible to a casual observer walking over the area. There must also be doubt concerning the similarity of conditions at the "building next door" unless its structural design has been examined closely to determine if its loads are at all comparable to those of the building being designed. This should not be construed, however, to mean that useful information cannot be obtained by observing adjacent buildings. There are many cases where the performance of an adjacent, similar structure can provide as much useful information as the subsurface investigation. The point is that such information should be used in conjunction with a subsurface investigation and not in lieu of it. In the writer's opinion, the saving of a very small fee for a proper subsurface investigation is mistaken economy when the extent to which the designer is staking his reputation and financial security is considered. An intelligent subsurface investigation is a coordinated effort of a designer and an experienced soils engineer. They should jointly determine the boring locations, the boring depths, the sample interval, the sampler type, the laboratory testing to be accomplished, and agree upon an analysis of the sub-surface conditions by the soils engineer which is properly tailored to the specific structure which the designer has visualized.

In closing, it should be mentioned that the practice of soil mechanics or soil engineering is a professional service; consequently, the services of a soils engineer should be obtained on a negotiated basis in accordance with recognized professional principles for the engagement of a consulting engineer. More specifically, the services of a soils engineer should not be obtained by competitive bidding or by any method which borders on competitive bidding. If sub-professional drilling and /or laboratory testing services have been obtained by competitive bidding, then a soils engineer not financially associated with the drilling or testing contractor should be engaged to direct the drilling and testing and to analyze the results. In recent years, there has been a strong movement in the soil engineering field toward the recognition of the *entire* subsurface investigation as a professional service in a manner that will allow the drilling, laboratory testing and engineering to be the subject of negotiation as a professional service rather than allowing it to be obtained on the basis of a competitive bid. It is believed that this procedure will result in a better subsurface investigation at the least overall project cost to the owner.

DURRANT AND BERGQUIST FIRM CHANGES NAME

The firm of Durrant and Bergquist has a new name. Because of the untimely death of Raymond Bergquist 18 months ago, the American Institute of Architects required that the firm name be changed. Now to be known as Durrant, Deininger, Dommer, Kramer, Gordon, Architects and Engineers, the firm will continue to office at Dubuque, where a new building has recently been built, and at Watertown, Wisconsin.

The original firm was started by Joseph G. Durrant in the year 1933, and he subsequently formed a partnership with Raymond G. Bergquist, past president of the Iowa Chapter, American Institute of Architects. The firm, Durrant and Bergquist, gained George E. Deininger, Jerold W. Dommer, Donovan D. Kramer and Gene P. Gordon as partners after extended association, but continued to use the original name until the present time. All partners are registered architects and members of the Iowa Chapter, American Institute of Architects.

Employed by the firm is a staff of 41 people, consisting of registered architects, professional engineers and specially trained personnel. Two new associates have been named: architect Norman Wirkler and professional engineer Ronald Baker.

DIRECTORY OF ADVERTISERS

KOCH BROTHERS HAS NEW INTERIOR DESIGNER

Kathleen Ulku has joined Koch Brothers Office Equipment Company, Fourth and Grand Avenues, Des Moines, as coordinator of interior design for the entire commercial interiors department. A graduate of the University of Minnesota in Interior Design, Miss Ulku will be available to coordinate and assist in the selection of fabrics, furniture, and other items offered by the commercial interiors department. She was formerly employed by Farnhams, Minneapolis.

IOWA CHAPTER CHANGES PUBLIC RELATIONS COUNSEL

Public relations counsel for the Iowa Chapter, A.I.A., has been Keiffer Associates, 3706 Ingersoll Avenue, Des Moines, since February 1, 1965. The chapter office will be maintained at this address in the future, thus completing the change-over in public relations services from the firm of Bonomi Associates, Des Moines, to the Keiffer Agency.

Robert F. Bonomi, having served the chapter since July, 1959, in the capacity of Executive Secretary and public relations counsel, assumed responsibility as managing editor of the "IOWA ARCHITECT" in September, 1956, it's third year of existence, and has aided its development from a mimeographed sheet to its present format. A professional journalist and free-lance creative writer, Mr. Bonomi's functions have ranged from that of writing many interesting articles and news items for the magazine to that of advising the chapter on such matters as legislation and executive policy. Mr. Bonomi leaves his position with the Iowa Chapter taking with him its best wishes.

APRIL-MAY-JUNE ISSUE KEYED TO AESTHETICS

Members of the chapter are reminded that material for the aesthetics issue is due April 15. The works of art that have been incorporated in designs by Iowa architects and drawings, paintings, sketches, etc. of chapter members will be featured in special sections.

Several chapter members have contacted the editor recently concerning work which they have done, and it is hoped that others are also preparing material. Please give this issue your consideration.

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