JOURNAL *of the*
BOSTON SOCIETY
OF
CIVIL ENGINEERS

1848
BOSTON
SOCIETY
OF CIVIL
ENGINEERS

119 YEARS
1848-1967

JANUARY - 1967

VOLUME 54 NUMBER 1

JOURNAL OF THE
BOSTON SOCIETY
OF
CIVIL ENGINEERS

Volume 54 JANUARY, 1967 Number 1

CONTENTS

Journal of the Boston Society of Civil Engineers is indexed regularly by Engineering Index, Inc.

Second-Class postage paid at Boston, Massachusetts

Published four times a year, January, April, July and October, by the Society
47 Winter St., Boston, Massachusetts

Subscription Price $7.00 a Year (4 Copies)
$1.75 a Copy
All orders must be accompanied by check

BOSTON SOCIETY OF CIVIL ENGINEERS

OFFICERS, 1966-1967

PRESIDENT

JOHN M. BIGGS

VICE PRESIDENTS

HARRY L. KINSEL
(Term expires March, 1967)

HARL P. ALDRICH, Jr.
(Term expires March, 1968)

SECRETARY

CHARLES O. BAIRD, Jr.

TREASURER

PAUL A. DUNKERLEY

DIRECTORS

ROBERT H. CULVER
MYLE J. HOLLEY, Jr.
(Term expires March, 1967)

ERNEST A. HERZOG
FRANK J. HEGER
(Term expires March, 1968)

PAST PRESIDENTS

JOHN F. FLAHERTY — WILLIAM A. HENDERSON — LESLIE J. HOOPER

SANITARY SECTION

ROBERT L. MESERVE, Chairman — CHARLES A. PARTHUM, Clerk

STRUCTURAL SECTION

ROBERT L. FULLER, Chairman — FITZ F. HAMPE, Clerk

TRANSPORTATION SECTION

CASIMIR J. KRAY, Chairman — ROBERT J. KILEY, Clerk

HYDRAULICS SECTION

NICHOLAS LALLY, Chairman — ATHENASIOS A. VULGAROPULOS, Clerk

CONSTRUCTION SECTION

ROBERT J. VAN EPPS, Chairman — ARTHUR H. MOSHER, Clerk

Editor—RONALD E. BUCKNAM
47 Winter Street,
Boston, Massachusetts 02108

THE STRUCTURAL DESCRIPTION OF THE JOHN HANCOCK CENTER – CHICAGO

by
E. Alfred Picardi *

The John Hancock Center should be one of the notable engineering achievements of our time. If it were just another 100-story building it would merely be another engineering accomplishment in the wake of the Empire State Building and would come just prior to the slightly higher twin towers of the World Trade Center planned for New York. However, this description will show why the John Hancock Center will be unique in many ways and perhaps a breakthrough in structural engineering technique.

The site is located on Chicago's upper Michigan Avenue between East Delaware Place and Chestnut Street. It occupies an entire block except for the northeast corner which is occupied by the Casino Club. A plaza and one level below grade will cover the entire site. The tower will occupy less than 50% of the site area. Vertically the structure extends from bedrock, about 143 feet below grade, to the roof at 1,106 feet above grade. Two 344 foot TV antennas on the roof extend the structure to the maximum FAA permissible height of 1,450 feet above street level.

Fig. 1. shows a rendering of the building as it will appear looking north along Michigan Avenue. The famous Chicago Water Tower is on the left and the Palmolive Building is in the background. This picture shows the diagonal system in the exterior frame which causes the four exterior walls of the structure to act structurally as a tube. The taper of the tower is quite apparent. Because of the multi-purpose function of the building as both an office building in its lower section and an

*Associate Partner – Chief Struct. Engr.
Skidmore, Owings & Merrill – Chicago

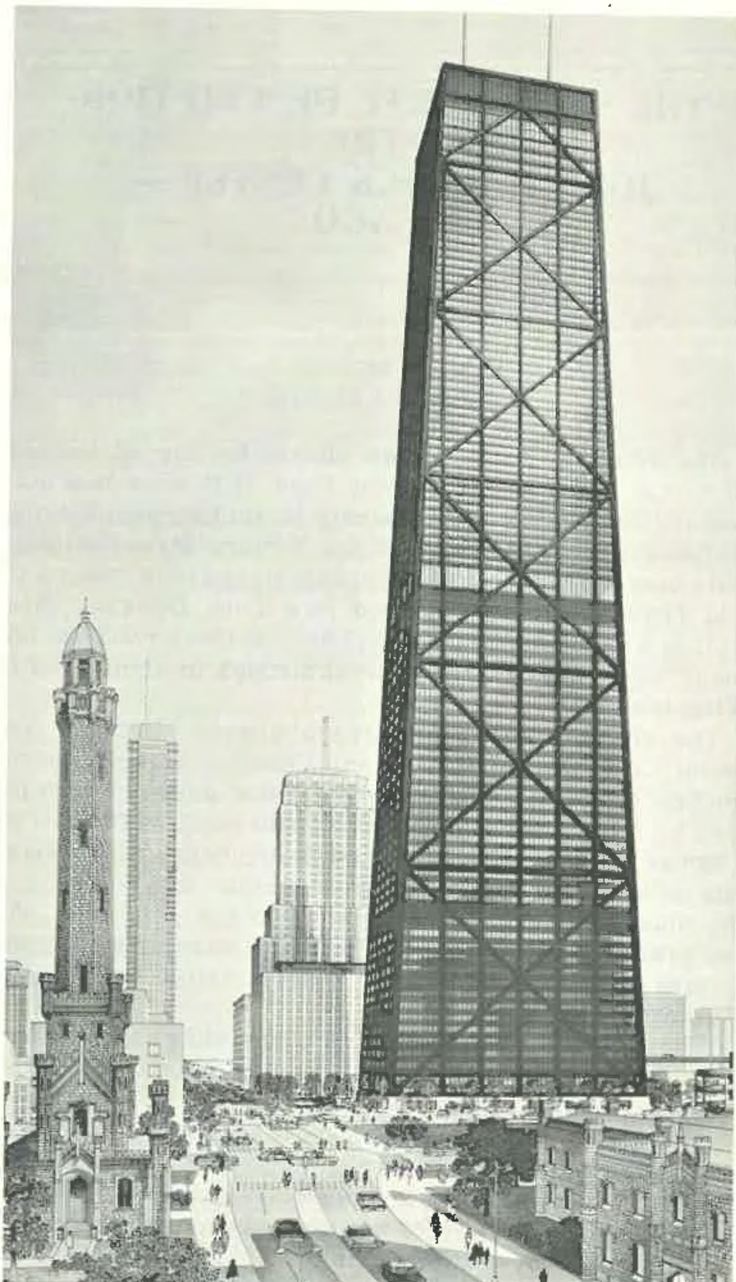


Fig. 1. The John Hancock, Chicago

apartment building in its upper half, the solution for ideal floor areas in each section required either a setback design from 40,000 square foot office floors to about 20,000 square foot apartment floors or a tapering structure. Insofar as the tapering structure permitted the more efficient structural design, this form was finally selected. The rendering shows the scale of the framing members which are clearly expressed in the facade. The architectural treatment includes aluminum covering of the structural frame. The glass infill completes the expression of the structure columns, spandrels and diagonals.

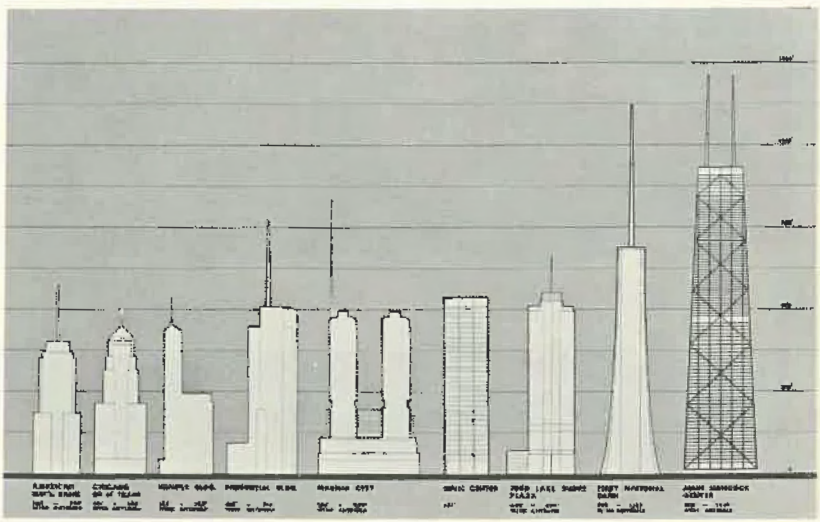


Fig. 2 A Comparison of Chicago's Tallest Buildings

A comparison of mass and height of the John Hancock Center with the eight tallest buildings in Chicago is shown in Fig. 2. Reading from right to left there are the John Hancock and the First National Bank Building, presently under construction. Both of these are steel frame buildings. The next is 1000 Lake Shore Drive, the highest concrete frame structure in Chicago, while the fourth building from the right is the steel frame Civic Center. The twin concrete towers of Marina City are next followed by the Prudential Tower, Kemper Insurance Company Building, the Chicago Board of Trade and the American National Bank Building. All of the last four are steel frames.

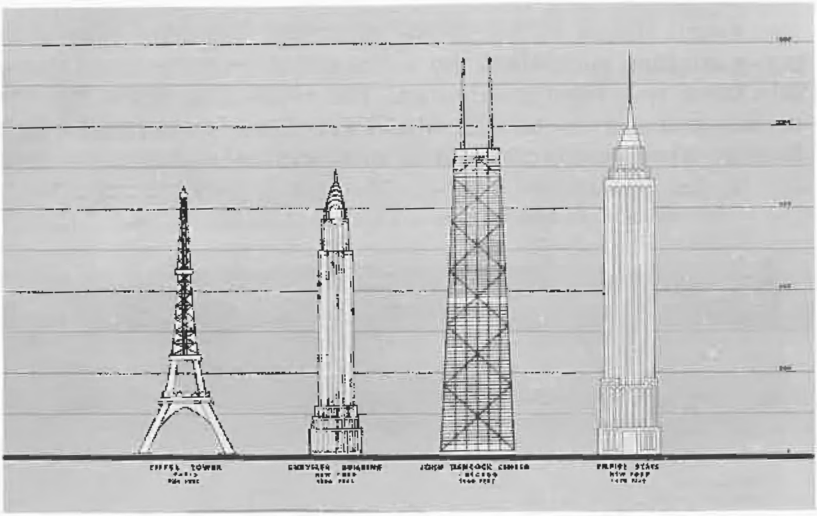


Fig. 3 A Comparison of the World's Tallest Structures

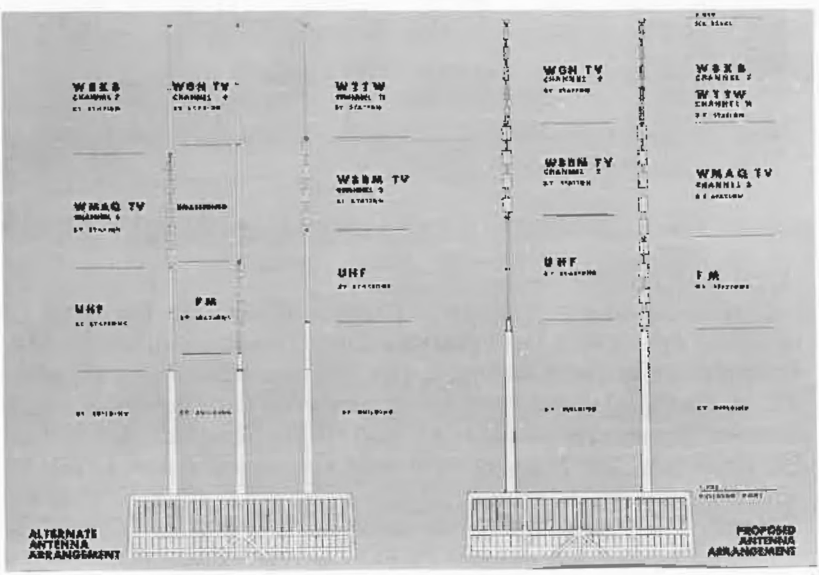


Fig. 4. Antenna Arrangement

Fig. 3. shows the mass and height of the John Hancock Center compared to the Empire State Building on the far right and the Chrysler and Eiffel Tower on the left.

The antenna arrangements at the top of the building are shown in Fig. 4. These are stacked antennas to accommodate all of the TV and UHF stations in the area. The antennas are free standing and supported by an anchoring system attached to interior core columns.

The primary structural materials quantities are as noted in Fig. 5. A total of 41,800 tons of structural steel will be used. The columns and diagonal system will require 68% of the total. The total weight of structural steel is only 31 pounds per square foot. This is attributed to the geometry of the structure, i.e., the taper, the nearly square plan and the tube type action of the frame which resulted in an extremely stiff and efficient structure for both gravity and wind loading. Bending under lateral load is essentially that of a cantilever beam.

STRUCTURAL STEEL	TONS	P.S.F.	% OF TOTAL
COLUMNS	21,200	15.8	51
DIAGONALS & TIES	7,200	5.3	17
FLOOR BEAMS	13,400	9.9	32
TOTAL	41,800	31.0	100

REINFORCING STEEL	34,000 TONS
CONCRETE	
FOUNDATION	6,400 C.Y.
BASEMENT	13,600
SUPERSTRUCTURE	40,000 (INCLUDING PLAZA)
TOTAL	60,000 C.Y.
EXCAVATION	100,000 C.Y.
BACKFILL SAND	19,000 C.Y.
SHEET PILING	715 TONS (M-27)
FLOOR AREA	
TOWER	2,700,000 S.F.
PLAZA	100,000 S.F.
TOTAL	2,800,000 S.F.

Fig. 5. Structural Material Quantities

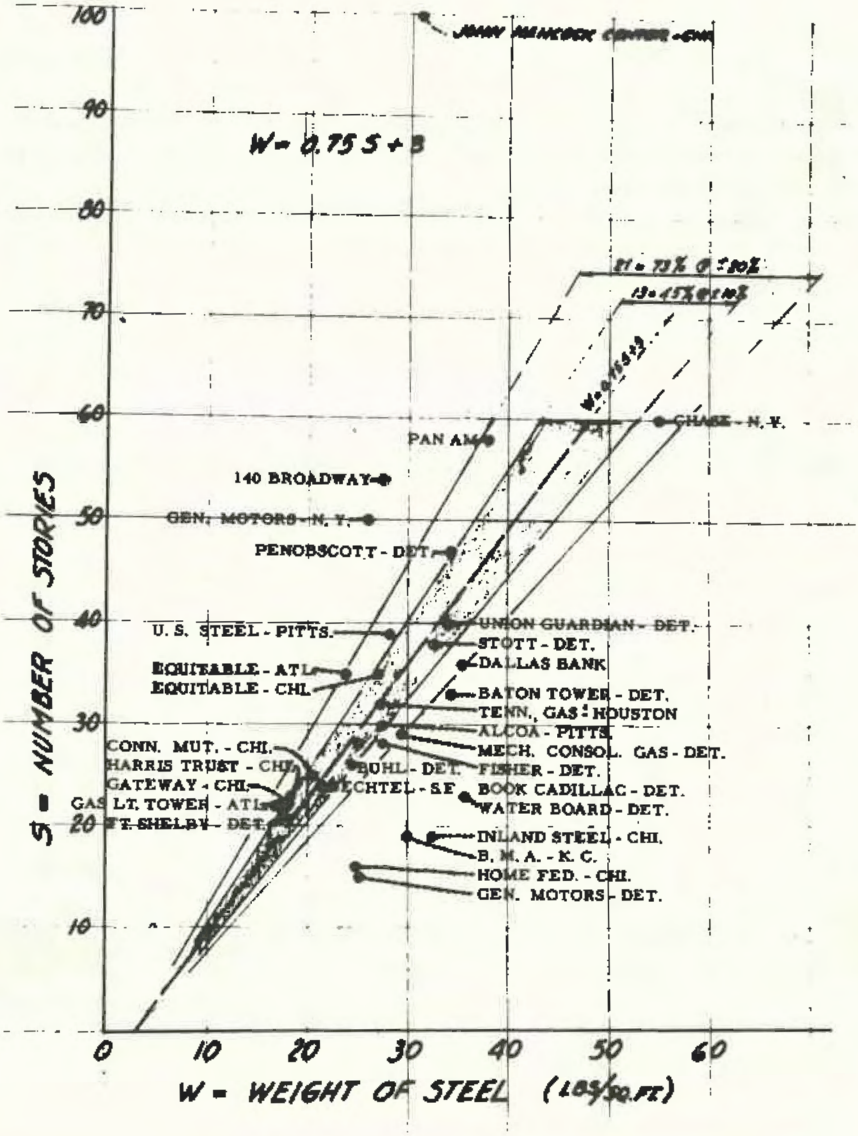


Fig. 6. Comparison of Frame Weight
with Other High-Rise Buildings

Comparison of the structural frame weight of the John Hancock Center with other contemporary and older high-rise buildings is shown in Fig. 6. Weight of steel in pounds per square foot is plotted against height in stories. The sample shows 30 buildings. Eleven are pre-World War II and 18 are post World War II buildings. The figure shows that 73% of the sample fits the curve within an envelope of ± 20%, and 45% of the sample fits the curve within an envelope of ± 10%. The John Hancock Center does not relate to these buildings at all and its steel weight is that of a conventional building of 30 to 45 stories.

Fig. 7. shows the weight of steel in pounds per square foot in each of the six sections of the building. The almost uniform decrease with height is as expected and the increase at the sixth level is primarily due to the heavier floor system of the mechanical and observatory levels and tonnage in the antenna support and anchorage structure.

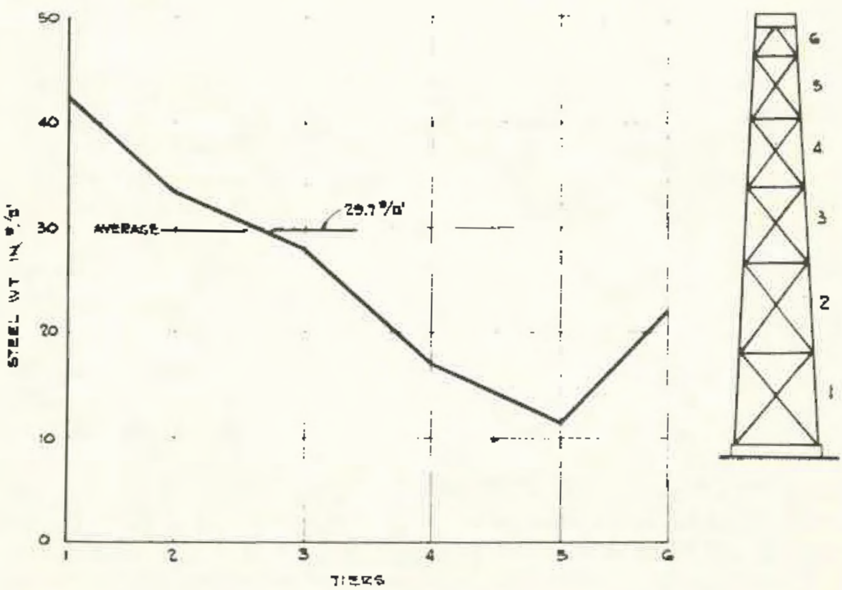


Fig. 7. Steel Weight per Tier

The plan at plaza level is shown in Fig. 8. The perimeter columns of the tower and the clear span from these perimeter walls to the central core should be noted. This long clear span from core to perimeter made it possible to get sufficient gravity load on the perimeter to counteract any uplift effect of wind from the action of the perimeter tube note under wind loading.

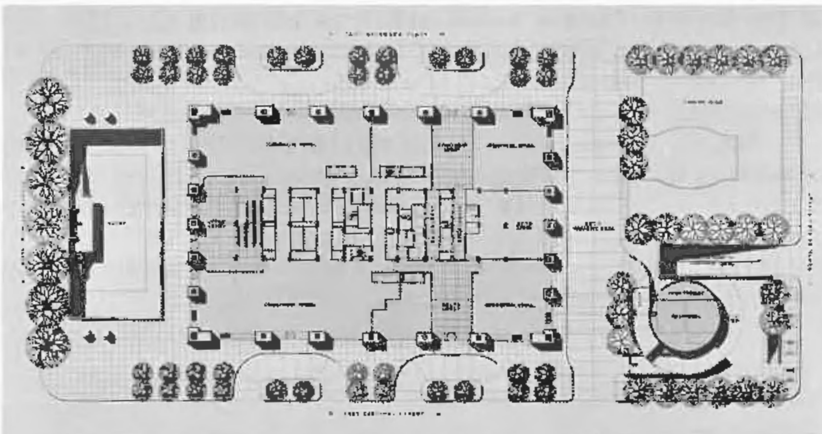


Fig. 8. Plan at Plaza Level

The framing of typical tower floors, Fig. 9., shows the central core and clear spans to the perimeter wall. These clear spans vary from 57 feet at the first floor to 25 feet at the roof. The floor system throughout is a composite design of steel stringers with shear connectors on the top flanges connecting the steel to light-weight concrete slabs varying from 5 to 7 inches in thickness. Sprayed-on fireproofing is used on the remainder of the steel stringer. All columns, girders, diagonals and spandrels have sprayed-on fireproofing.

One of the major concerns in the design was development of an adequate wind analysis. Selection of a proper wind loading was one of the first concerns. Fig. 10. shows the data development. Using data from observations at Midway Airport from which isotach maps have been prepared, a 50 year wind of 85 m.p.h. at 30 feet was used. Assuming a gradient height of 900 feet for flat open country and the 1/7 power law, the gradient velocity at Midway was found to be 138 m.p.h. This velocity was used and a gradient height of 1,100 feet was assumed at the John Hancock Center near the center of a large

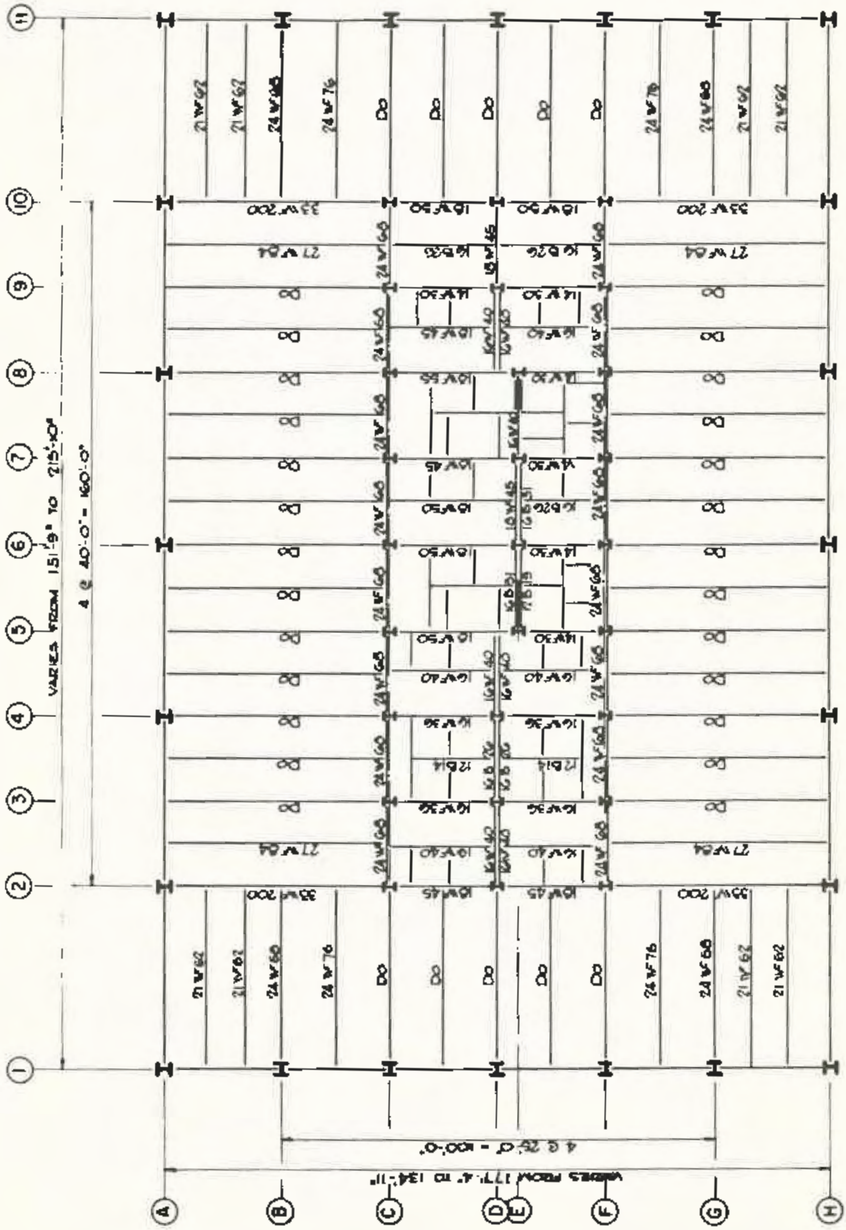


Fig. 9. Typical Office Floor Framing Plan

WIND VELOCITY AT MIDWAY AIRPORT:

RECURRENCE INTERVAL	FASTEST MILE AT 30 FOOT ELEVATION
100 YRS.	74 M. P. H.
50	68
10	57
2	46

WIND VELOCITY AT MIDWAY AIRPORT (ISOTACH MAP):

50 YRS.	85 M. P. H.
10	65
2	47

USING 85 M. P. H. AT 30 FEET

$$V_G = V_Z\left(\frac{Z_G}{Z}\right)^{1/n} = \text{GRADIENT WIND} = 85\left(\frac{900}{30}\right)^{1/7} = 138 \text{ M. P. H.}$$

ASSUMES GRADIENT HEIGHT FOR FLAT OPEN COUNTRY = 900 FEET AND 1/7 POWER LAW.

$$V_Z = V_G\left(\frac{Z}{Z_G}\right)^{1/n} = \text{VELOCITY AT HEIGHT } Z$$

$$V_{1100} = 138\left(\frac{1100}{1500}\right)^{1/3} = 124.6 \text{ M. P. H.} = \text{VELOCITY AT 1100 FEET AT JOHN HANCOCK CENTER}$$

ASSUMES GRADIENT HEIGHT FOR PROXIMITY TO CENTER OF LARGE CITY = 1500 FEET AND 1/3 POWER LAW.

Z = 1100	V_Z = 124.6
800	= 112.1
600	= 101.8
400	= 88.9
200	= 70.6

$$P_Z = 0.00256\left(V_Z\right)^2 \times \text{DRAG COEF.} \times \text{GUST COEF.}$$

DRAG COEF. = 1.3 FROM MODEL TEST

GUST COEF. = 1.3 AT GROUND TO 1.0 AT Z_G = 1500 FEET

$$\text{THEREFORE, } P_Z = 0.00256\left(V_Z\right)^2 \times 1.3 \times \left(1.3 - \frac{Z}{5000}\right)$$

Z = 1100 FT.	P_Z = 55.8 P. S. F.
800	= 47.6
600	= 40.7
400	= 32.1
200	= 20.1

Fig. 10. Data Development for Wind Loading

city with the 1/3 power law. This facilitated the establishment of velocities at 1,100 feet and below. The pressures at various heights were then computed according to the formula shown.

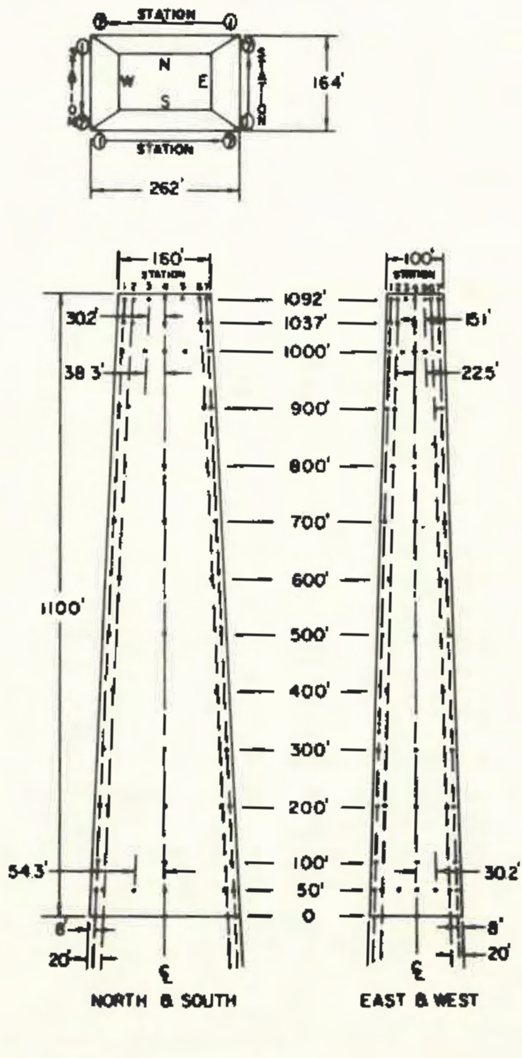


Fig. 11. Model Used in Wind Tunnel Test

The drag coefficient, 1.3, was based on data from a model test in a wind tunnel. The gust coefficient was assumed as 1.3 at ground level and varied uniformly to 1.0 at 1,500 feet. In this manner design wind loads were computed as shown, varying from 20 p.s.f. at grade to 55.8 p.s.f. at 1,100 feet.

A sketch of the model used in the wind tunnel test is shown in Fig. 11. Pressure orifice locations were established on each face of the model to detect both positive and negative pressures. The model could be rotated to measure pressures from wind blowing in any direction. The model included not only the tower, but also all buildings to scale within a 1,000 foot radius of the tower.

Wind Tunnel Tests Results:

A. Static wind pressure for all angles of wind incidence is significantly affected by the turbulence caused by surrounding buildings. As a result, the pressures are considerably lower than would be theoretically predicted for unhindered flow. The tapering shape of the structure may also contribute to the lower pressures.

B. Higher suction pressures around the corner areas seem to occur in the lower half of the building. In fact, the maximum suction pressure coefficient, 1.29, occurs at 50'-0" above ground level with wind from the southwest.

C. The maximum drag coefficient is caused by wind perpendicular to any face of the building. However, under such a condition the sides are completely under suction pressure indicating a total separation of wind flow line at the sides. It is, therefore, a fair assumption that the wind flow does not produce appreciable friction forces on the sides.

D. Wind pressure contours were plotted for wind incidence angles of 45° and resultant drag coefficients have been computed. Maximum drag coefficients for wind were 1.08 on the long face and 1.28 on the short face. These figures are considerably smaller than the originally assumed drag coefficient of 1.40.

A plot of wind pressure versus height, Fig. 12., compares the Chicago Building Code minimum wind pressure with the wind pressure used in the design computed as previously mentioned. The computed curve fits very closely the Chicago pressure multiplied by 140%.

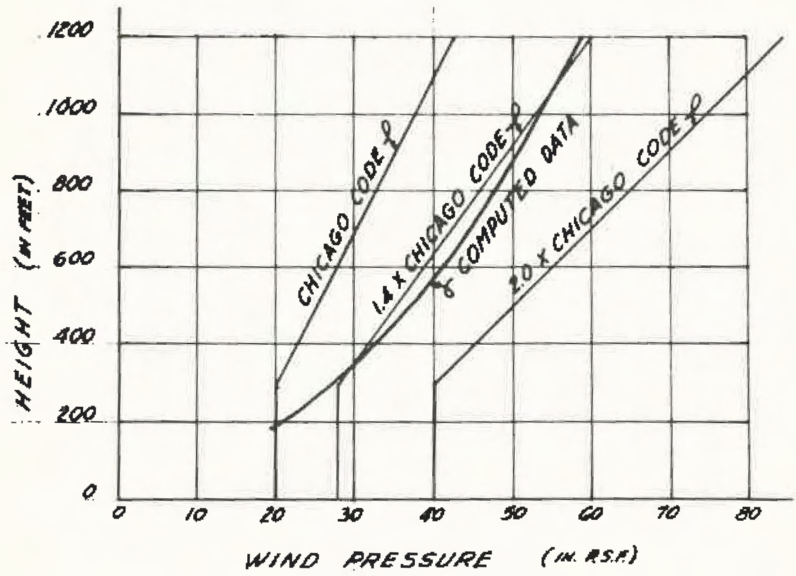


Fig. 12. Wind Pressure vs. Height

The criteria for stresses in any member used in the design was as follows:

A. DL + LL Stress = Working Stress
B. DL + LL + Wind = 1.33 x Working Stress (where wind pressure is 1.4 x Chicago Code).
C. DL + LL + Wind = 30 KSI (where wind pressure is 2.0 x Chicago Code).

Fig. 13. shows the results of a study on resonance possibility for the structure by plotting critical velocity versus height for both wind on the long face and short face of the building. Superimposed is the 50 year wind velocity versus height curve. It is apparent that the intersections are sharp and occur at about mid-height in the tower. The taper, which produces the nega-

tive slope of the critical velocity curves, helps considerably and it is apparent that resonance would be impossible under these conditions.

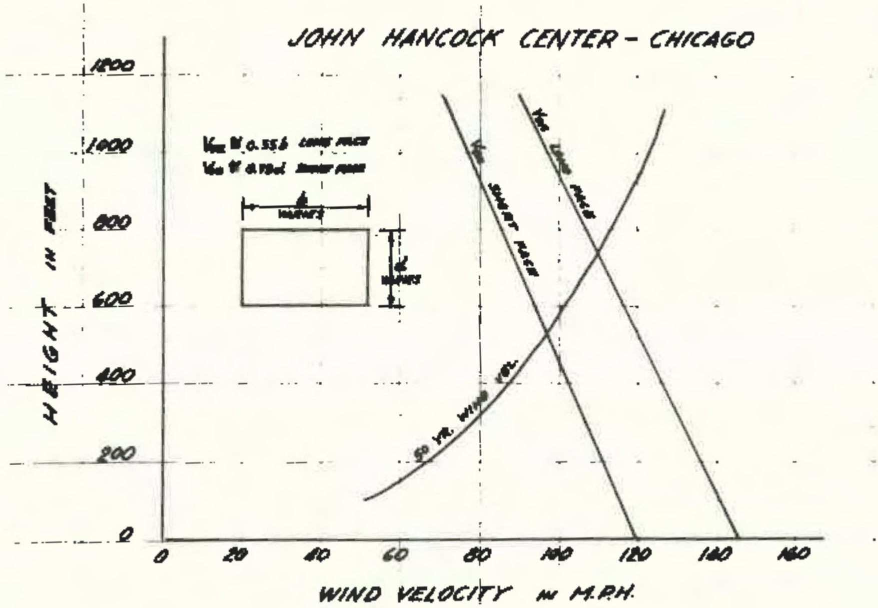


Fig. 13. Critical Velocity vs. Height for Resonance Study

The structure consists primarily of a diagonally braced exterior column system designed to act as a tube carrying all lateral loads and a portion of the vertical loads. A central core column system carries the remainder of the vertical loads. It was found that a minimum number of diagonals added in the plane of the exterior walls could accomplish this tube or rigid box effect.

Preliminary analysis considered the structure as a space truss and was used in proportioning primary members, columns, diagonals and ties. Columns were initially proportioned for gravity and wind loads using a reasonable allowable compressive stress assuming the structure would act like a cantilever tube under wind load. Diagonal members were proportioned to limit total shear wracking to 20% of the total lateral deflection. In order to eliminate excessive secondary moments in the columns from excessive strain in the ties, the ties were proportioned for a maximum strain of one quarter of an inch.

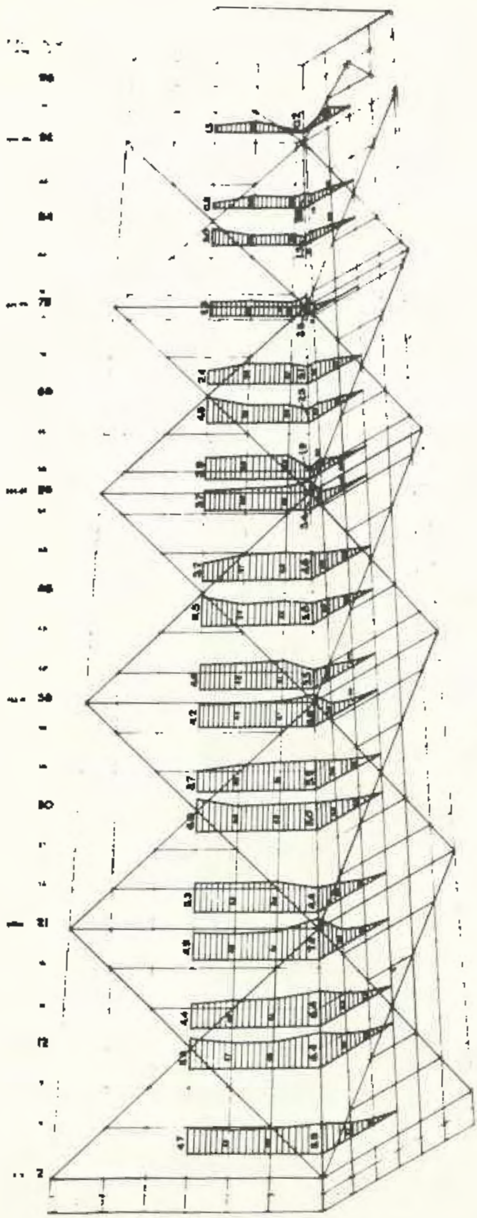


Fig. 14. Column Stresses Due to Wind on Long Face

After preliminary analysis and further adjustment of member sizes the structure was analyzed as a space frame using the STRESS program. Several runs were made for each loading condition and minor adjustments were made prior to finalizing the design.

Fig. 14. is one of the plots of member stresses which show dramatically the tube action of the frame under lateral loading. This plot shows column stresses due to wind on the long face of the structure. It should be noted that the windward columns are almost uniformly stressed and collectively act as elements of the flange of a tube. It should also be noted that the columns in the faces parallel to the wind direction have the stress distribution which would normally be expected in the web of a tube.

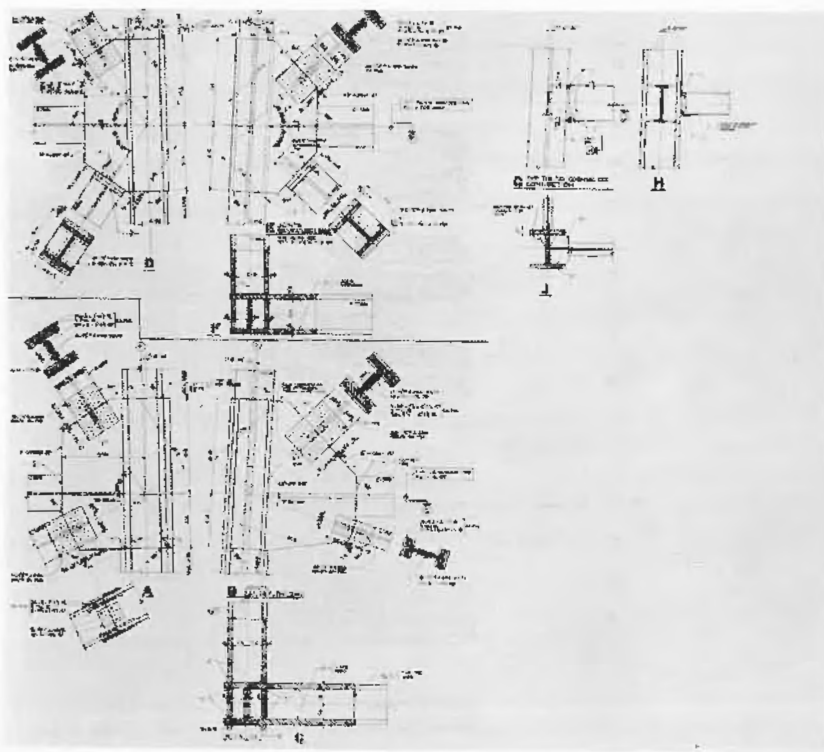


Fig. 15. Detail of Connection Between Vertical Column, Spandrel and Diagonal

This stress distribution is primarily the result of the effectiveness of the diagonal system as a distributor of the wind forces. The diagonal system works equally as well in distributing the dead load and live load stresses equally to the columns.

The detail of a connection between the vertical column, spandrel and diagonal is shown in Fig. 15. The joint made up of gusset plates, web section and spandrel stubs is shop fabricated. The column splice and spandrel connections are field welded. One end of the diagonal is field welded and the other

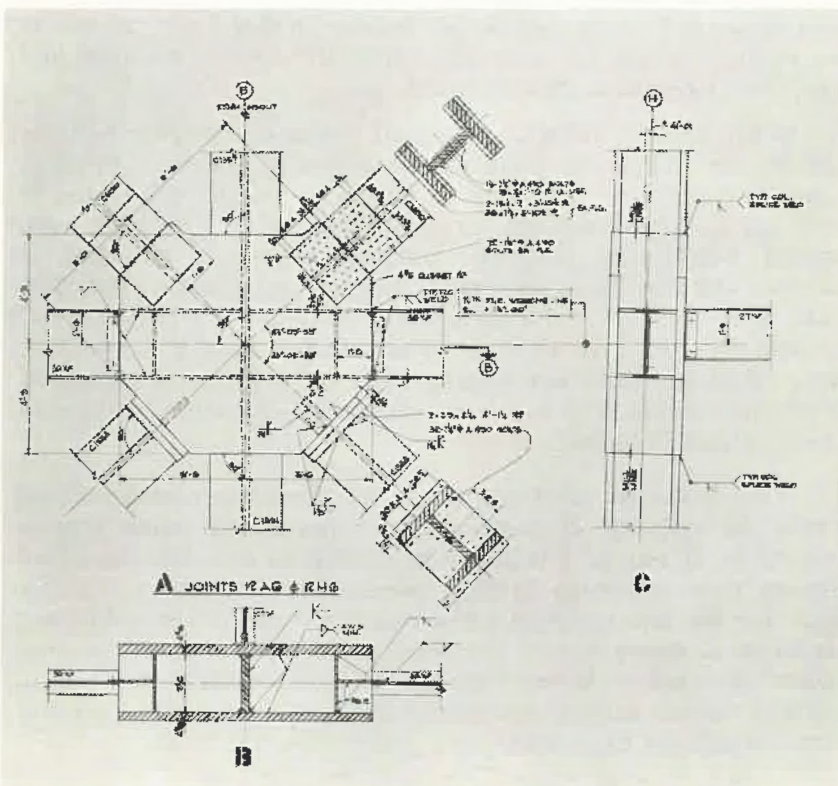


Fig. 16. Detail of Connection Between Column, Spandrel and Two Diagonal Members

end is bolted to facilitate erection and alignment. Joints in the lower half of the building are A-441 steel. All other steel throughout the frame is A-36.

Fig. 16. shows a typical joint between column, spandrel and intersection of two diagonal members. The shop inspection and field inspection includes use of magnetic particle inspection according to ASTM E-109, radiographic inspection according to ASME Boiler Code paragraph UW-52 Section 8 and ultrasonic inspection according to ASTM E-164-60T. All "H" sections built-up are magnetic particle inspected on the root pass and last pass for 9 inches at each end and 5% of the remainder of the welds. All joints are fully magnetic particle inspected on the root and last pass. All joint welds are also fully inspected using ultrasonic technique and 5% of the welds are radiographed. All plate used for built-up "H" sections and joints is ultrasonically inspected.

In the case of field welding, all welds of exterior column splices are magnetic particle inspected on the root and last pass. For interior columns in the core 25% of the welds of each splice are magnetic particle inspected. Diagonal connection welds are magnetic particle inspected on 100% of the root and final pass and also ultrasonically inspected. The main ties of each of the tiers are also magnetic particle inspected on root and final pass and ultrasonically inspected. Then 5% of all welds are radiographed. On ordinary spandrel to column welds only magnetic particle inspection of the entire root and final pass is used.

Two bolts in all floor beam to girder connections and girder to column connections are inspected using torque wrenches. If any bolt is found to have less than the specified tension then all bolts in the connection are checked. Shear studs for the composite floor beams are visually inspected and ten studs of three beams per floor are bend tested 30 degrees toward the center. If one stud fails then all studs on the member are tested and an additional beam is tested for each one having a failure of a stud.

A typical detail of the aluminum cladding, insulation and fire-proofing of the exterior frame is shown in Fig. 17. The first six stories of the frame will be erected by crawler cranes on the basement level. Thereafter, four creepers lifted into

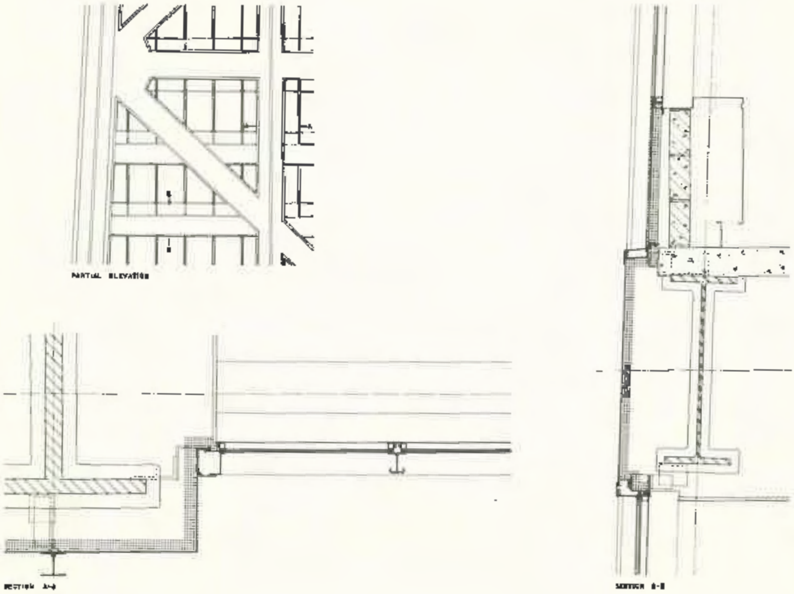


Fig. 17. Detail of Cladding, Insulation and Fireproofing

place by the crawlers will erect the remainder of the frame as shown in Fig. 18. One creeper will be attached to exterior columns on each face of the frame up to the 80th story. Thereafter, two creepers will complete the job. Each creeper will consist of a support platform connected to two of the building columns, a 35 foot high tower on the platform and a stiff leg derrick on the tower. The derricks will have 105 foot booms up to the 80th story and 120 foot booms thereafter. The four hoists will be mounted on the building frame and will be spotted three times at the 2nd, 38th and 75th floor. The lead-in will have a 20,000 pound pull. The maximum lift of the creepers will be 28 tons. The maximum lift of the crawlers will be 99 tons. The creepers will be jumped 108 times: 30 times on the end walls and 23 times on the side walls. At each creeper position additional bracing of about 15 tons will be required on three floors. The bracing will be dismantled and reused each time the creeper is jumped.

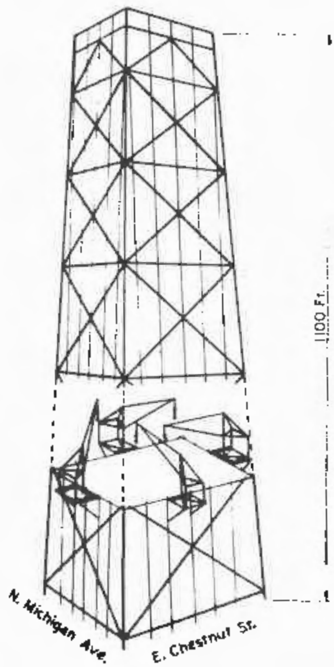


Fig. 18. Creeper Placement

Figs. 19. and 20. deal with the foundations. The site is about 12 to 14 feet above the level of Lake Michigan and ground water level is at the lake level. From grade to a depth of 25 feet the strata is lake sand. Clays, varying from soft to tough strata underlay the sand down to about 90 feet. Between 90 and 106 feet the strata consists of very hard silty clay with traces of sand gravel and some boulders and is generally classified as "hardpan" in Chicago. From 106 to about 136 feet the strata consists of medium dense to dense silt, sand, gravel and boulders. This strata is water-bearing with a hydrostatic head which could rise up to about 50 feet below grade. Broken weathered limestone underlays this strata in thickness varying from about a foot to as much as 5 or 6 feet. The solid rock under this strata is dolomitic limestone with some joints and fractures and small seams.

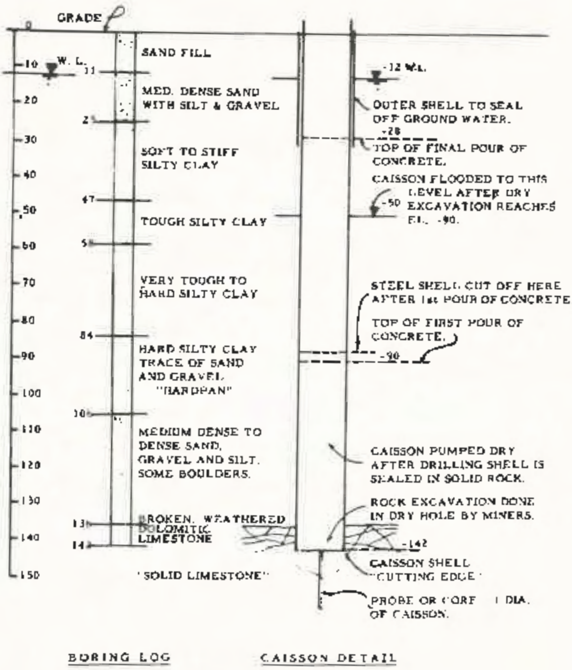


Fig. 19. Caisson Construction

The foundations for the main tower consist of cylindrical concrete caissons founded on the solid limestone. The foundations for the plaza areas consist of concrete caissons with bell bottoms founded in the hardpan at about 90 feet below grade. Typical boring log and detail of the caisson construction scheme are shown in Fig. 19. There are 57 caissons to rock supporting the tower. There are also 22 caissons belled out on hardpan supporting a portion of the first floor of the tower and 164 hardpan caissons supporting the plaza area.

Rock caissons under the tower are as follows:

Location	Diameter	Maximum Load
Corner	10'-0"	14,300 KIPS
Long Side	8'-0"	8,700
Short Side	6'-0"	4,800
Core	6'-0" to 8'-4"	4,800 - 9,700

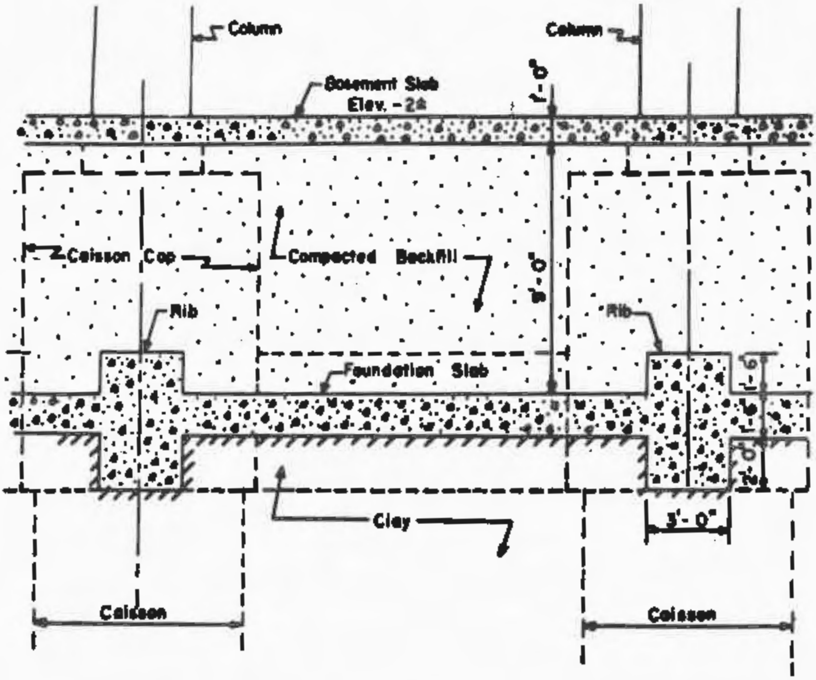


Fig. 20. Section Through Substructure

All caissons are designed with 5,000 p.s.i. stone concrete using flyash. Working stress is 25% of f'_c or 1,250 p.s.i. Maximum contact pressure on the rock is 90 tons per square foot.

The procedure for caisson construction was as shown on the detail in the figure. Caissons varied from 3'-6" to 10'-0" in diameter and all were machine excavated using rotary auger drills down to the weathered rock. The rock was cut by rotating the steel shells which had hardened steel tooth cutting edges. The procedure consisted of augering through the sand fill and forcing an outer steel shell into the clay to seal off the ground water. Augering was then continued through the clay down to about 90 feet below grade under dry conditions. At this time the hole was flooded with water and a mixture of bentonite. Augering continued through the hardpan seal and into the water-bearing sand and gravel strata. The slurry head in the caisson was always kept above the hydrostatic head in the sand-gravel strata. When the augers hit the weathered rock steel shells were inserted in the shafts and rotated to cut into the rock to a depth where it was probable that a seal was possible. The hole was then pumped and if a seal had been made, it was pumped dry. Miners then entered the shaft to remove the weathered rock. The solid rock was then drilled and probed. Any water leakage was caulked and the bottom dried and inspected prior to pouring concrete. Concrete was poured up to about elevation 90 feet below grade. A few days later the shells were burned off about 10 feet above the top of the pour and the remainder of the shaft was filled with concrete. The upper portion of the steel shell was then withdrawn and salvaged for use on other shafts.

A section through the substructure, Fig 20., shows the arrangement of construction consisting of a lower slab and rib grid system connecting the caisson caps. The purpose of the grid system is primarily to transfer wind shear from the underlying clay strata. Total wind shear using 1.4 x Chicago Code wind is 8600 kips. The shear strength of the clay is about 500 p.s.f. and the factor of safety against a shear failure is about 2.5. The lower rib grid is surcharged with 9 feet of compacted fill plus the basement slab. This provides the necessary load to prevent uplift from the hydrostatic head from lake level water and also keeps the grid slab in contact with the clay strata.

Fig. 21. shows the detail for soil solidification of the sand strata under the footings of the Casino Club at the northeast corner of the site. This building on continuous shallow footings has its south and west walls within ten feet of the Hancock basement structure and it was obvious that the driving of sheet piling and excavation might disturb the sand on which the footings are founded. It was decided to solidify the sand between the bottom of the footings and the clay strata. A chemical grout consisting of silicate of soda, formamide and calcium chloride was used. About 40,000 gallons of grout were used. Pipes spaced 2'-6" on center were driven as shown in the detail. 306 pipes were used. The truncated section was solidified to a compressive strength of over 200 p.s.i. and spread the footing load

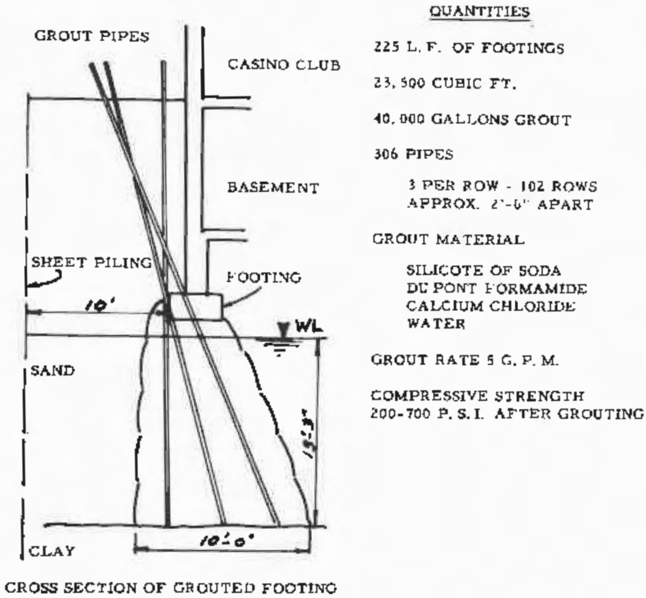


Fig. 21. Grouting Operation Under
South and West Wall of Casino Club

out on the clay to a width of about 10 feet. This procedure has been successful and to date only very minor damage to the old building has been observed.

STABLE TEST PADS FOR INERTIAL NAVIGATION SYSTEMS**

by
K. Tsutsumi *

INTRODUCTION

There are many challenging civil engineering problems in aerospace projects. Most of these problems are associated with ground support facilities such as roads, buildings, drainage systems, dams and causeways, power stations, water supplies, launch pads, docks and wharves, special equipment handling devices, fuel manufacture and storage, and assembly and testing of missile components. One of these problems, which is the subject of this paper, is the design and construction of foundations which support testing equipment used in the calibration of inertial navigation systems and their individual components.

An inertial navigation systems is a self-contained combination of motion-sensitive devices with appurtenant mechanical and electronic subassemblies. When mounted in a vehicle, the system not only determines vehicle position continuously but does it automatically, and also directs the craft to its destination. One unique feature of inertial navigation is that it requires no contact with the outside world after takeoff. Inertial navigation systems were initially developed by Dr. C. Stark Draper, Professor and head of the Department of Aeronautics and Astronautics, Director of the Instrumentation Laboratory, Massachusetts Institute of Technology.

The primary sensing instruments in inertial navigation are gyroscopes and accelerometers. The performance of an

---

* Professor of Civil Engineering, Tufts University, Medford, Massachusetts.

** Presented before the Society, February 9, 1966.

inertial navigation system is dependent upon the product of the precision of each component. Therefore, in order to build a high-performance system, it is necessary to make each component as precise as possible.

In the past, components such as gyroscopes and accelerometers were calibrated satisfactorily on test equipment that was mounted on the basement floor of a building or on a concrete pad resting directly on the ground. In recent years the instruments being tested have become so sensitive that this method of mounting test equipment has become inadequate. The problem of providing a good support for test equipment has become so important that various groups in the aerospace industry have held meetings and symposia on the subject of test pad stability. The American Institute of Aeronautics and Astronautics has a special subcommittee on this subject.

The purpose of this paper is to review some of the environmental conditions that affect the problem of constructing a stable test foundation, and to describe some present solutions. Environmental conditions include geology of the site, variations in temperature and humidity, seismic vibrations, and wind and barometric pressure.

SOME BASIC APPROACHES

There are at present three known basic approaches to the construction of stable foundations on which are mounted test equipment for calibration of motion-sensitive instruments with varying performance specifications.

1. Find an environment which meets the test pad performance specifications and construct a test pad at this site. Specifications for a test facility may require an environment which has the least possible natural seismic noise. The quickest way to find such an environment is to look at a map similar to that shown as Fig. 1, choose a location, and confirm the map data with tests in that locality. This map shows that the shoreline areas have the greatest seismic disturbances and the Rocky Mountain regions have the smallest seismic disturbances.

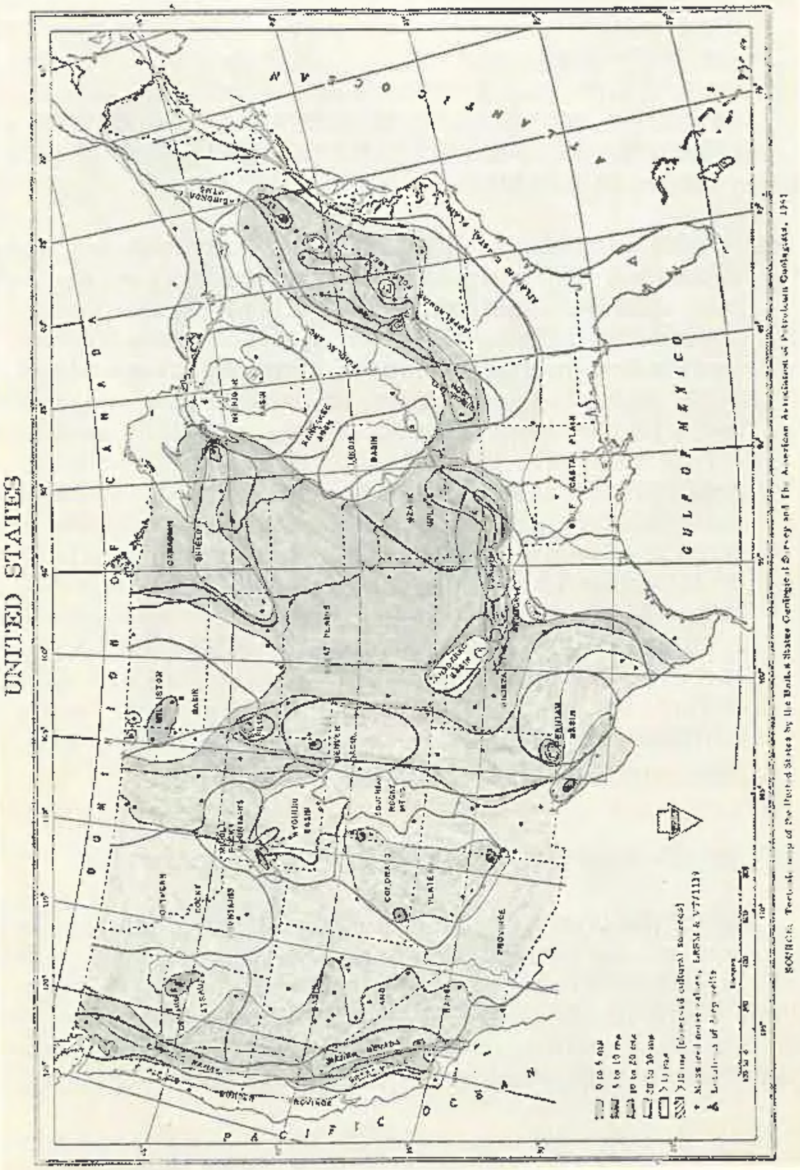


Fig. 1. Map Showing Background Seismic Noise

2. Accept a poorer environment but monitor all foundation motions in order to apply corrections to the test results. This method means that the corrections to be applied to the test results are only as accurate as the equipment used to monitor the motions of the foundation. Monitoring equipment errors lead to uncertainties in the test results. Improper application of corrections will also cause erroneous calibrations.

3. Design and build a servo-stabilized test foundation that is optimized and tuned to meet the particular needs of the test. An ideal stabilized platform is isolated from the six degrees of motion of the ground. These six degrees of motion are the linear motions along the three orthogonal axes and angular motions about the same axes. This ideal has not been attained. Most servo-stabilized platforms today are limited to the isolation of two or three degrees of freedom and have limitations of the range of isolation for each degree of freedom. Some of these systems are isolated for low frequencies which have periods of less than 10 seconds per cycle. Complete servo-isolation has not been attained for frequencies higher than 1/10 cps. A few test foundations have been isolated with passive elastic devices for frequencies of 10 cps and higher. No system, servo-driven or passive, has been developed to completely isolate the test foundation for frequencies between 1/10 cps and 10 cps.

TEST FACILITY IN A QUIET ENVIRONMENT

One of the best test foundations located in a quiet seismic environment was built by the Martin Company in Denver, Colorado. This test facility was described by (Ref. 1) L. O. Mathis, Chief, Facilities Engineering, Martin Denver, at the American Institute of Aeronautics and Astronautics Guidance and Control Conference held in Minneapolis, Minnesota, on August 16, 1965.

Fig. 2 shows the location of the selected site after a geological study of the area was conducted, and after measurements were made with seismographs to determine the charater of the vibrations in the ground.

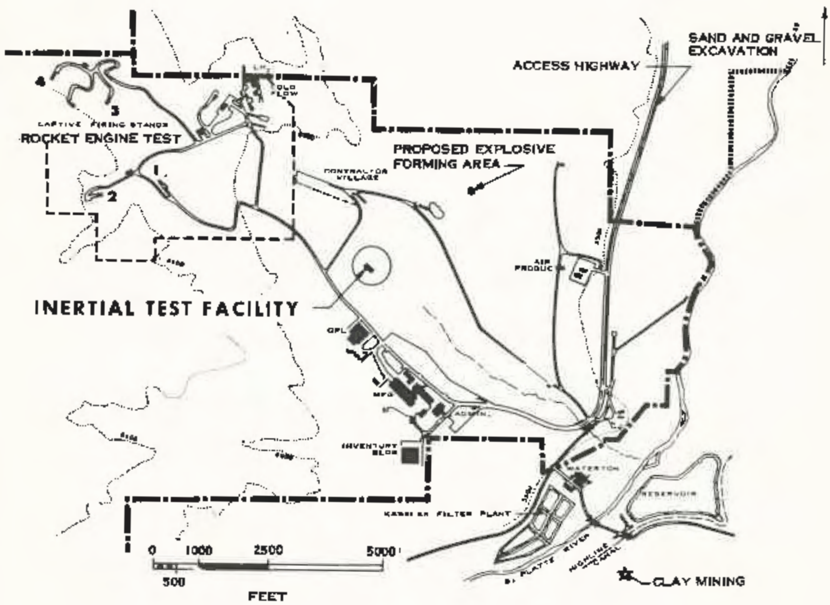


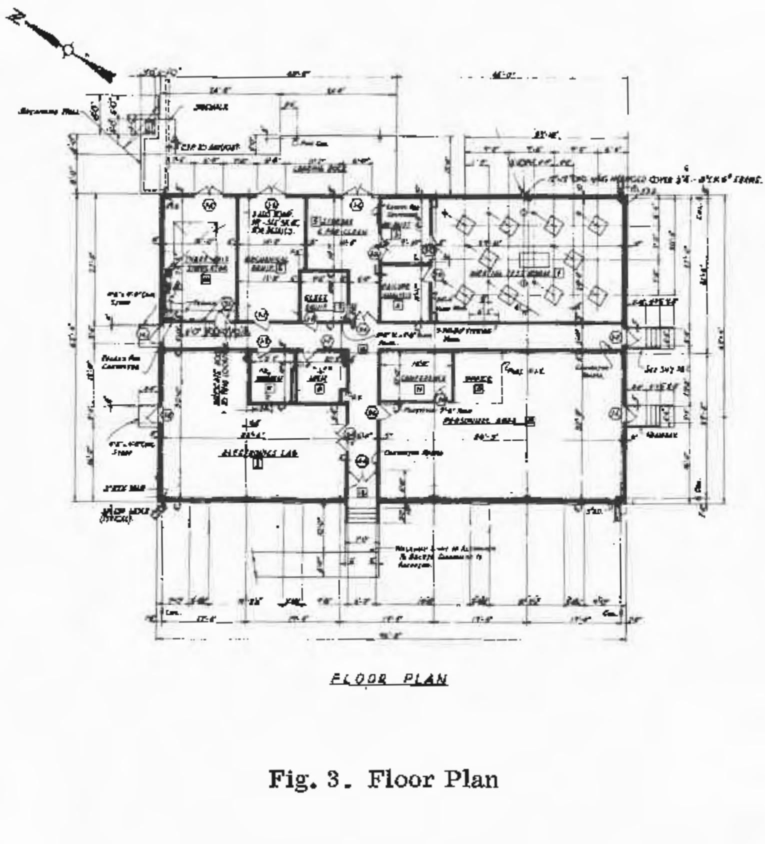
Fig. 2, Martin-Denver Area Plan

The Martin test pad is located in the eastern quadrant of the inertial test facility building, which is approximately 62 ft. by 96 ft. in plan. The pad is about 23 ft. by 39 ft. in plan, and approximately 8 ft. thick. It rests on fresh rock and is surrounded by compacted soil fill 7 to 8 ft. thick. The fill is held away from the pad by a retaining wall which rests on about 8 inches of styrofoam plastic at the rock surface, which is level with the bottom of the pad. The retaining wall is about 11 ft. high, and it is supported at mid-height by a footing extending back into the compacted fill. A false floor on soft springs spans over the pad. Concrete piers from the pad pass through holes in the false floor and extend about six inches above it. The holes in the floor are larger than the piers so that there is no physical floor-pier contact. Test instruments are mounted on the piers. All mechanical equipment such as the boiler, air conditioning blowers and compressors is mounted on mechanical spring isolators.

The Martin test facility is shown in the following figures:

Fig. 3 is a floor plan of the inertial test facility building.

Fig. 4 is a plan view of the structural framing and test pad.



FLOOR PLAN

Fig. 3. Floor Plan

Fig. 5 is the east-west section of the test pad.

Fig. 6 is a photograph of the completed inertial test room.

Fig. 7 is a close-up view of the soft spring supports for the false floor.

Fig. 8 shows the tilt and vibration sensing instrumentation for testing the piers. There are three seismographs, one for each orthogonal axis, and one tilt meter.

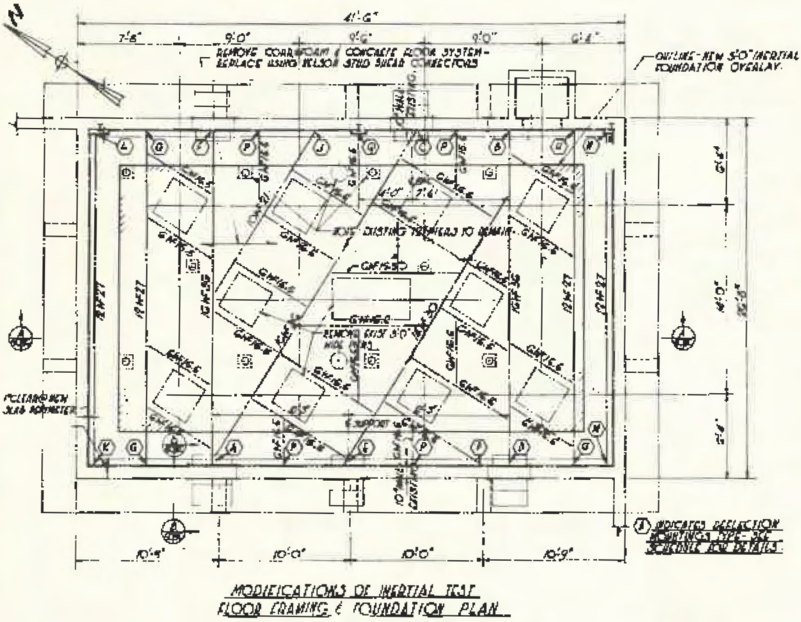


Fig. 4. Plan View of Structural Framing and Test Pad

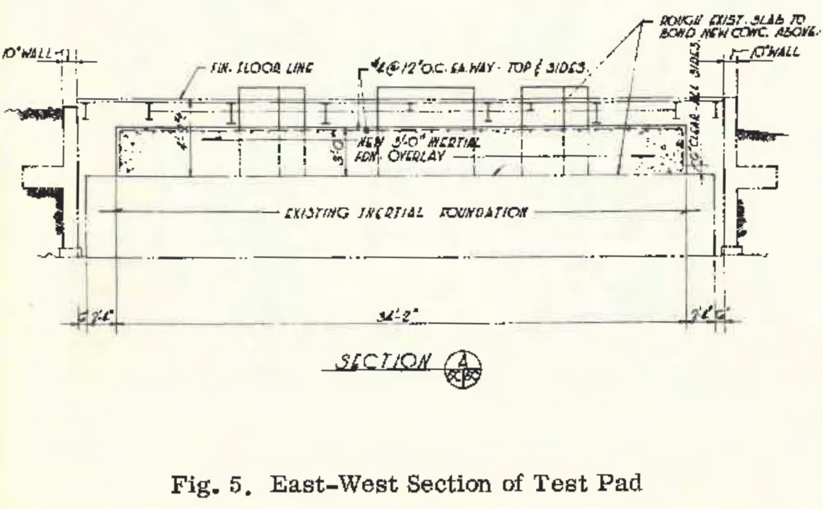


Fig. 5. East-West Section of Test Pad

Fig. 6. Inertial Test Room

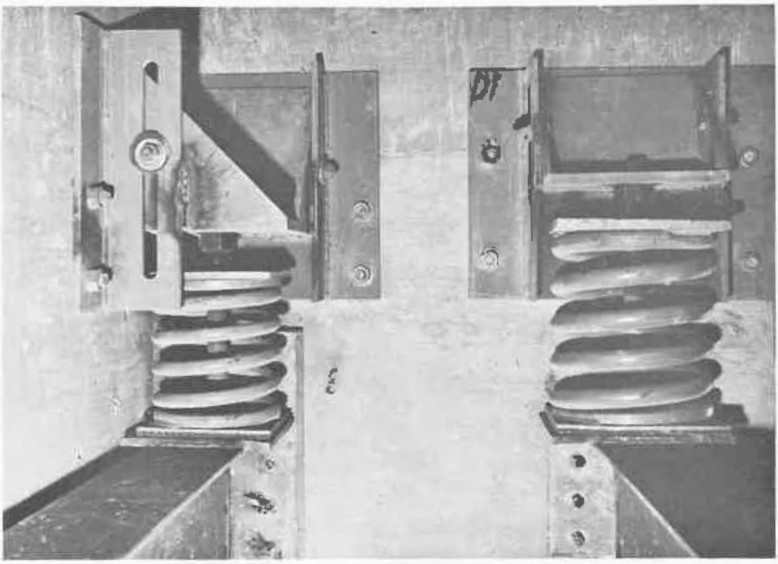


Fig. 7. Soft Spring Supports for False Floor

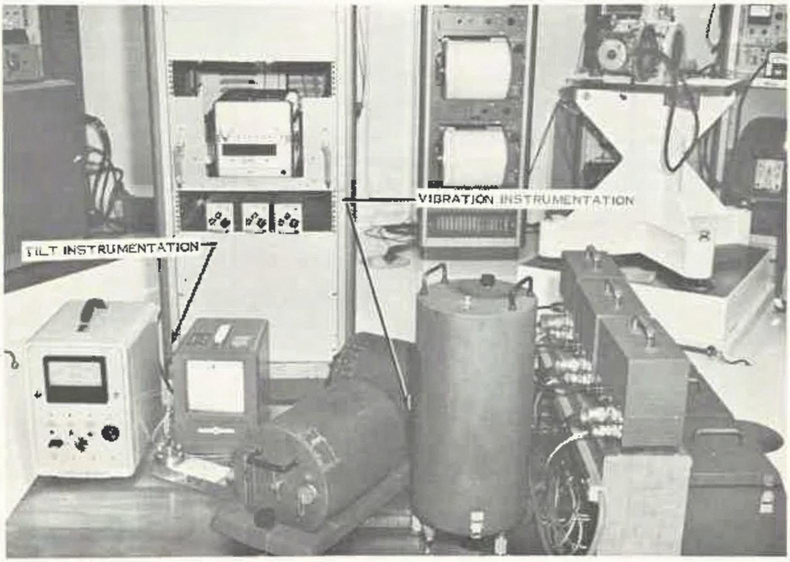


Fig. 8. Tilt and Vibration Sensing Instrumentation

The performance of this inertial test pad is summarized for vibration in Fig. 9 and for tilt in Fig. 10. Tilts are defined as angular vibrations of long periods.

Performance specifications for the Martin test pad were a maximum acceleration of 50 micro-g's and a tilt stability of 2 arc seconds within 24 hours. The summarized vibration and tilt measurements show that these specifications were fulfilled except in unusual cases for vibration such as (1) when personnel walked outside the building adjacent to the test pad, (2) when a 2-1/2 ton truck with solid tires crossed an outcrop of the test pad bedrock formation, 700 ft. away, and (3) when rocket engines were fired 8000 ft. away from the test pad.

TEST CONDITION	ACCELERATIONS					
	VERTICAL		HORIZONTAL			
			N-S		E-W	
	a µg	f cps	a µg	f cps	a µg	f cps
1. AMBIENT CONDITIONS						
a. All Equipment OFF	0.4	4	0.7	4	1.0	5
b. All Equipment OFF. (Gusty Winds)	3(T) ④	9	19(T)	22	29(T)	31
c. All Equipment ON - Normal Operation	21	25	21	25	34	25
2. AIR CONDITIONERS						
a. AC #1B Start and Run	6(T)	14	NIL ①		6(T)	14
b. AC #4 Start and Run	17	25	40	25	42	25
3. WALKING						
a. On Suspended Floor in Inertial Test Room	0.6	4	2	4	3	4
b. Corridor Adjacent to Inertial Test Room	14	28	34	32	21	28
c. Personnel Room	NIL		24	28	20	28
d. Outside Building Adjacent to Test Pad	40	28	75	28	85	28
4. DOORS CLOSING						
a. Air Lock Entrance to Inertial Test Room	NIL		10	27	NIL	
b. Inertial Test Room Door	24	27	42	30	29	29
5. BOILER START (HEATING SYSTEM)	8	36	10	26	10	27
6. REMOTE DISTURBANCES						
a. Light Van at Loading Dock	28	25	28	25	48	25
b. Shakers in Env. Lab (1300 Feet)	NIL		NIL		7	7
c. Semi-Trailer Tank Truck (700 Feet)	6	10	5	10	9	9
d. Truck - 2½ Ton, Solid Tires, (700 Feet) ②	62	13	113	13	54	13
e. Simulated Explosive Forming - (3800 Feet) ③	23	5	9	4	13	5
f. Rocket Engine Firing (430,000 lbs. Thrust) - 8000 Feet ③	2900	182	3100	125	3700	60

Notes:

① "NIL" Signal Not Distinguishable from Ambient

② Truck Crossed Outcrop of Bedrock Formation on Which Test Pad is Located.

③ Measured on Bedrock Prior to Installation of Test Pad.

④ (T) Indicates Transient Occurrence

Fig. 9. Summary of Vibration Measurements on Test Pad

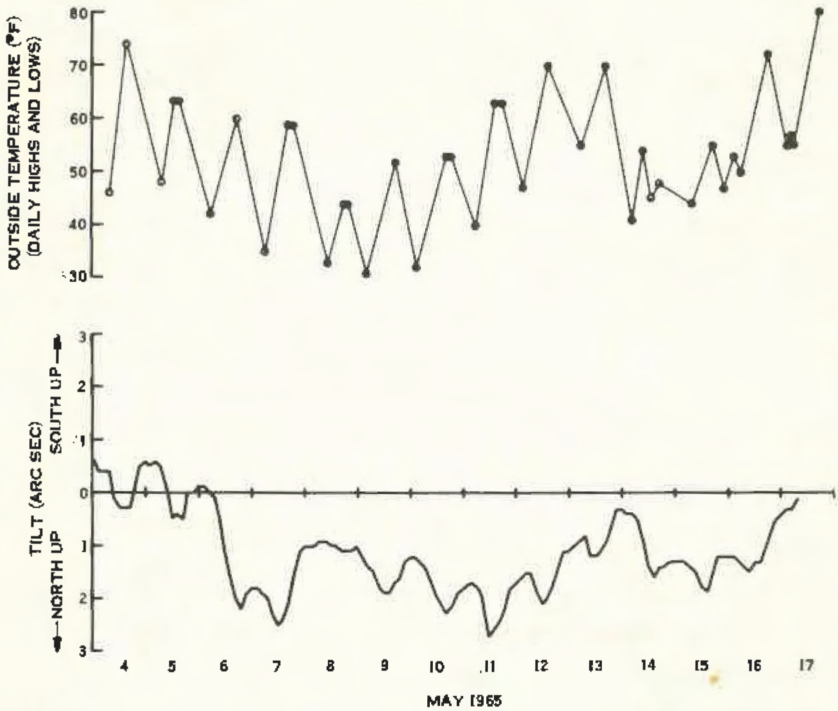


Fig. 10. Test Pad Tilt

HONEYWELL INERTIAL TEST SLAB STABILITY STUDY

A very extensive series of tests for measuring tilts of the ground has been carried out by Honeywell, Inc. of Minneapolis, Minnesota. The purpose of this study was to determine the principal causes of long period test slab motion. Results of these tests were presented (Ref. 2) by Ralph T. Berg, Honeywell Development and Evaluation Laboratory Engineer, at the AIAA Guidance and Control Conference of August 16, 1965.

In the Honeywell study, concrete slabs were positioned strategically on common geological formations of the upper midwest area, both on the surface and underground. Typical formations included sandstone, silica sand, granite, quartzite and sand over clay. The test sites of this study were: Roseville, a suburb of Minneapolis; Zimmerman, a rural area north of Minneapolis; St. Peter Sandstone, a sand mine tunnel in St. Paul, Minnesota; St. Cloud, Minnesota, northwest of Minneapolis; and Baraboo, Wisconsin, located between Milwaukee and Minneapolis.

The Honeywell investigations consisted of measuring low frequency ground tilts with precision level vials and recording the bubble positions photographically. Fig. 11 shows the construction of one of the precision level vials used in the study.

Fig. 11 Precision Level Vial

Fig. 12 shows the array of level vials, clock and counter assembled on a triangular plate. This plate rests on the concrete slab or on the surface to be measured. Fig. 13 shows how the level vial positions are recorded photographically.

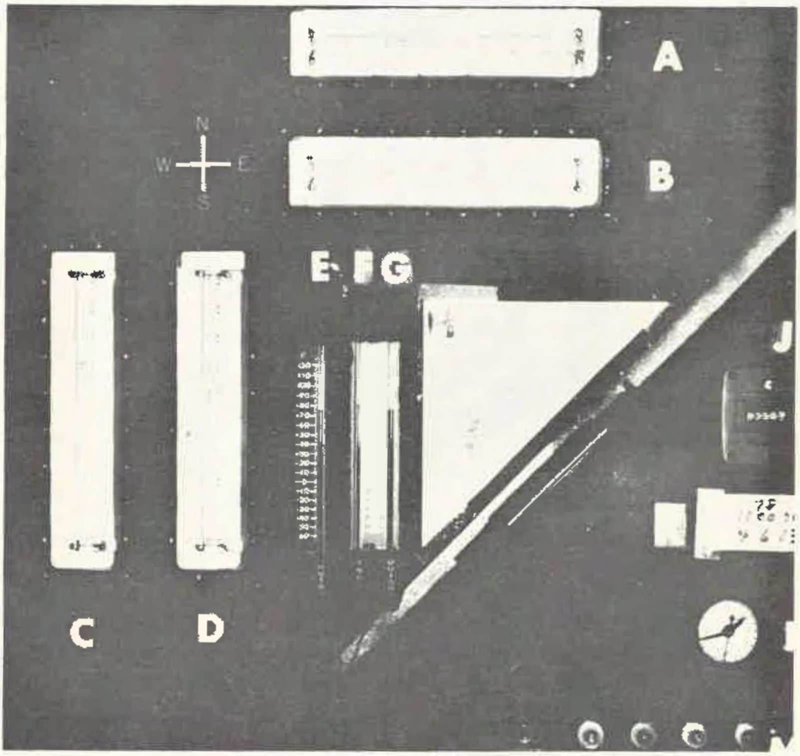


Fig. 12 Typical Data Frame

Roseville Sites

There were four slabs on ground in a building located at Roseville. They are shown in plan view as Fig. 14. Roseville No. 1 was a square slab. Roseville No. 2 was a square slab built for a concrete stability test. Stainless steel studs were inserted in the slab and measurements were made with a

Fig. 13 Camera Mount Set-Up

level to determine relative heights of the studs as the concrete aged. Roseville No. 3 was a square slab mounted on sand and clay. Roseville No. 4 was a circular slab mounted in clay surrounded by 4 ft. of sand.

Tests results for the period April, 1963, to January, 1964, are given in Fig. 15 for Roseville No. 1 and on Fig. 16 for Roseville No. 3. Fig 17 gives test results for Roseville No. 4 for the period May, 1963, to January, 1964. Fig. 18 gives results of the Roseville No. 2 concrete slab stability test. This graph shows the creep rate or distortion with age of this 12 ft. square slab.

Results of tests at Roseville stations 1, 3 and 4 show the slabs had daily oscillations of from 2 to 4 arc seconds, and long period tilts of 6 to 12 arc seconds. Roseville No. 2 tests showed that creep was rapid for the first two months after casting, but the creep rate was slower for the next eight

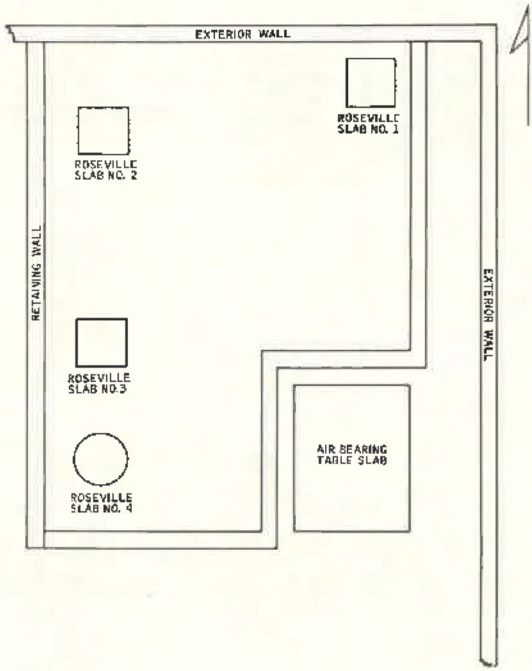


Fig. 14. Roseville Test Site

months. This seems to indicate that the concrete slab will continue to distort for a long time before reaching a dimensionally stable condition.

Zimmerman

At the Zimmerman test site, a housing was built around a concrete slab resting on silica sand. The depth of the sand was 225 ft., and the water table was at a level 8 to 10 ft. below ground surface. Ground temperature was measured by thermocouples placed at 2 ft., 4 ft. and 6 ft. below ground level. In addition to surface tests for tilts caused by natural disturbances, thermal tests and underground tests were conducted at this site.

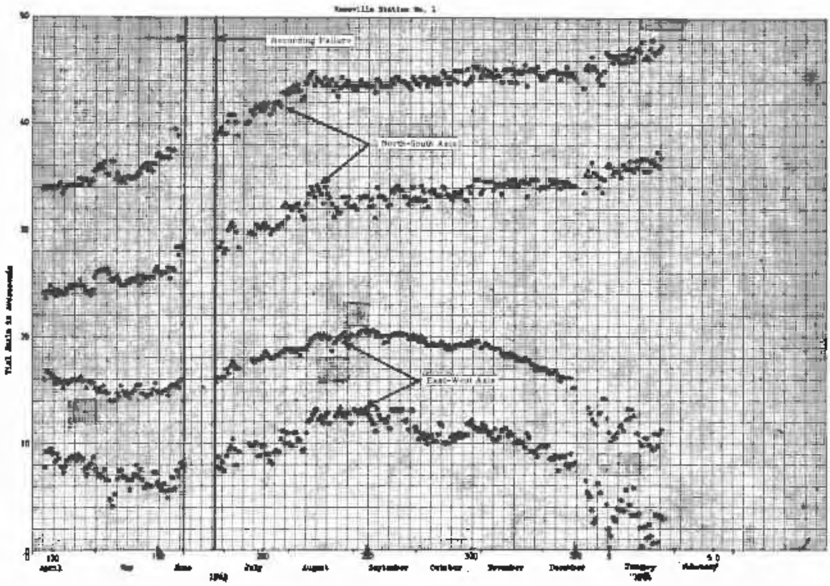


Fig. 15. Test Results Roseville #1

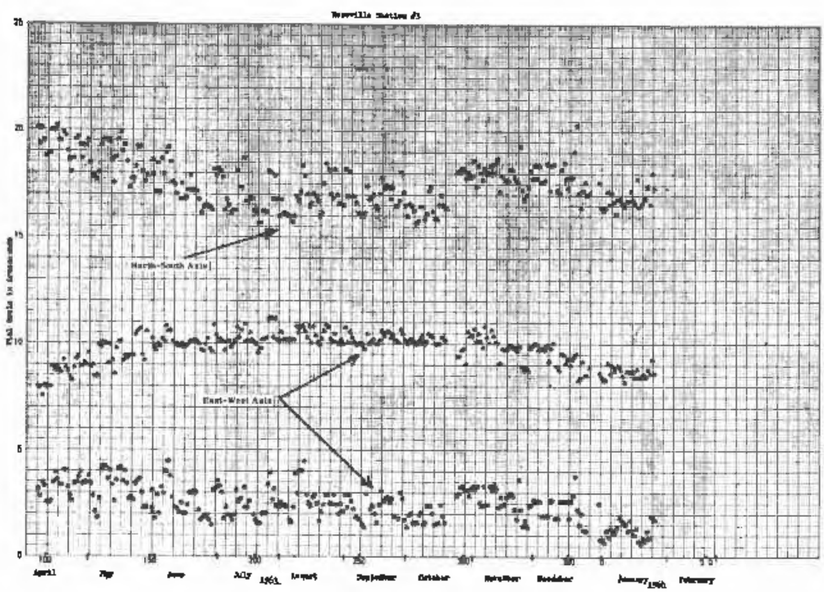


Fig. 16. Test Results Roseville #3

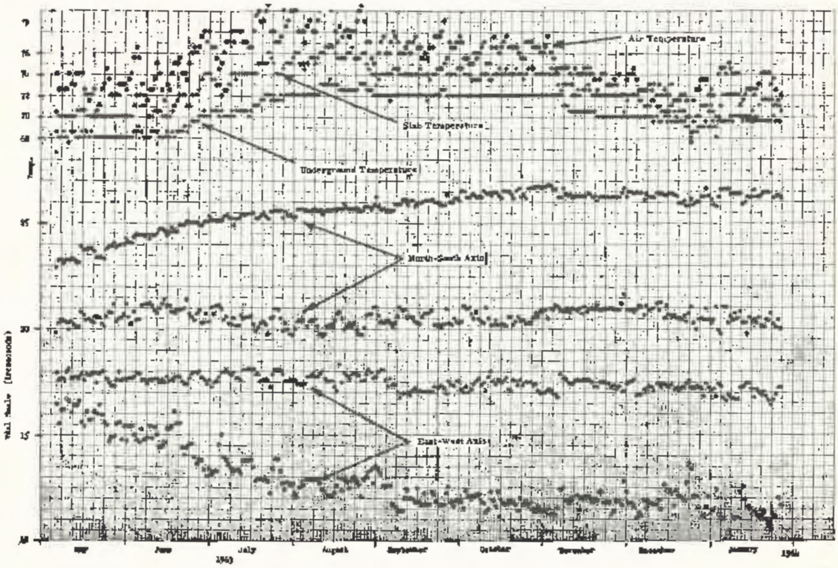


Fig. 17. Test Results Roseville #4

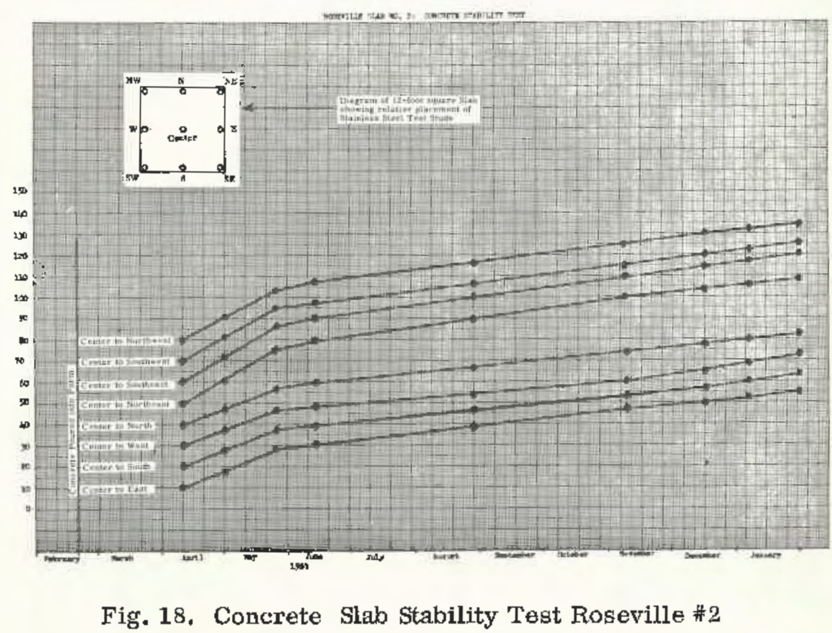


Fig. 18. Concrete Slab Stability Test Roseville #2

After approximately five and a half months of tilt measurement for natural causes, an artificial heat pulse was introduced into the supporting soil 4 ft. south of the slab and 2 ft. below ground level. In order to introduce the heat pulse rapidly into the soil, aluminum rods were driven 30 inches into the base of the pit. Eight hundred pounds of petroleum coke were burned in this pit over a period of four days.

At the time the fire was extinguished, there was little change in slab orientation. However, monitoring thermocouples indicated that a heat pulse was moving to the soil supporting the slab. The maximum deflection of the slab occurred nine days after the beginning of the pulse. The maximum slab tilt in the east-west axis was about 7 seconds, and the maximum tilt in the north-south axis was about 5 seconds. The purpose of this test was to confirm that horizontally stratified thermal gradients cause test slab tilts.

Since thermal gradients exist near the ground surface, it was decided to test for tilts below the ground surface. A test station was placed at a depth of 7 ft. below ground. This location was a foot above the water table at the site. It was theorized, and later confirmed, that the relatively high thermal conductivity of water would reduce thermal gradients to a minimum. The instrument mounting plate was placed directly on the sand. Test equipment indicated rotational and thermal gradient stability for two months, after which tilts of the instrument plate were observed. The cause of the tilts was determined to be the drying out of the surrounding sand. This test indicated that changes in moisture content of the soil also cause ground tilts.

In addition to the preceding tests at Zimmerman, soil temperature measurements were taken at depths of 2 ft, 4 ft. and 6 ft., from July, 1964, through December 1964. The temperature records show that a uniform ground temperature of 50° F was reached in December. This ground temperature of 50° F was noted at all of the underground test sites. It was also found that local thermal gradients occurred in the shadows of buildings and trees. Computations were made which indicated that yearly fluctuations in ground temperature will not be observed at a depth of 52 ft. or more. Fig. 19 is a record of the tilts measured at this site from December, 1963, to December, 1964. This record included the artificial heat pulse experiment. Fig. 20 shows Minneapolis temperatures which were similar to weather which occurred at Zimmerman, 45 air miles distant. Fig. 21 is a record of the test 7 ft underground at Zimmerman. This graph shows the effect of the change in moisture content of the surrounding sand at the test location. Fig. 22 is a record of the Zimmerman site ground temperature at depths of 2 ft, 4 ft and 6 ft for the period July, 1964, through December, 1964.

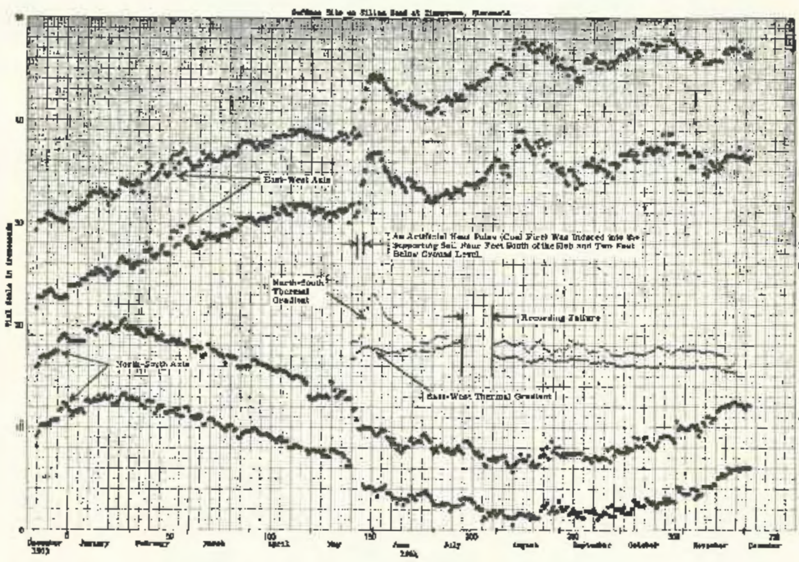


Fig. 19. Test Results Zimmerman Surface Site

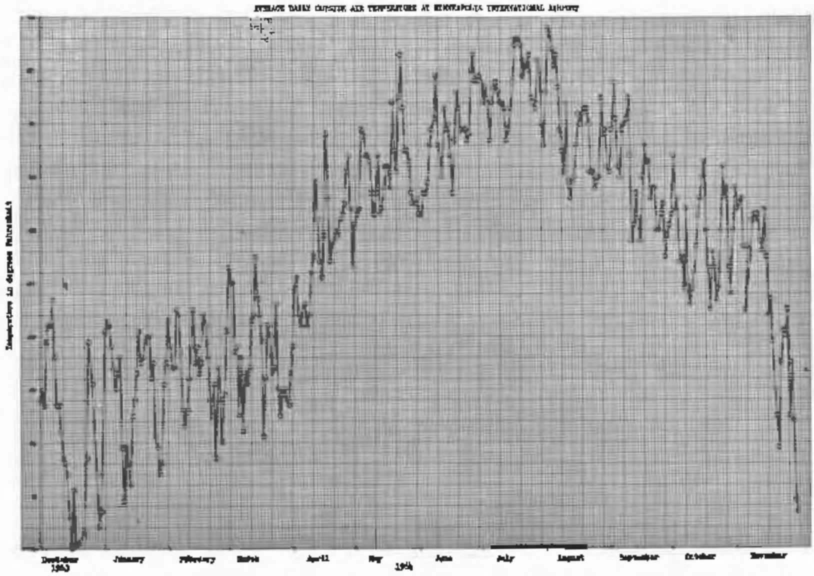


Fig. 20. Minneapolis Average Daily Outside Air Temperature

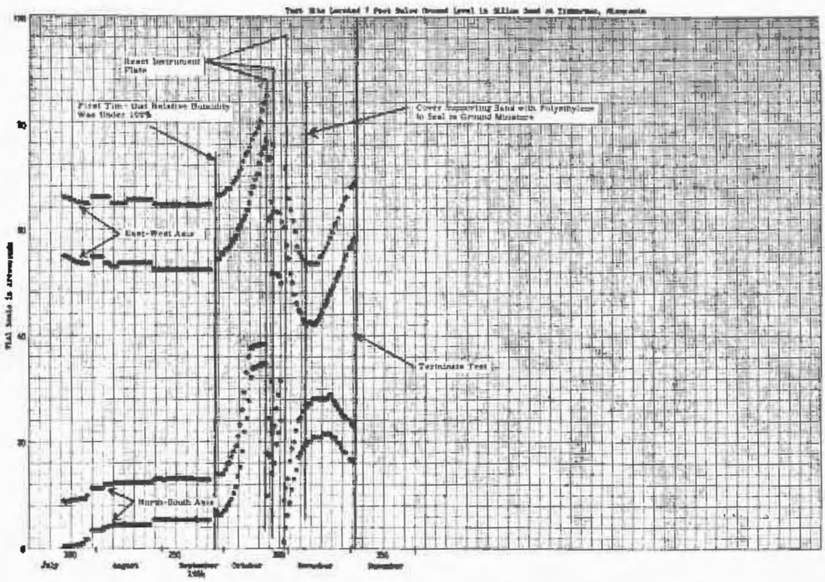


Fig. 21. Test Results Zimmerman Underground Test Site

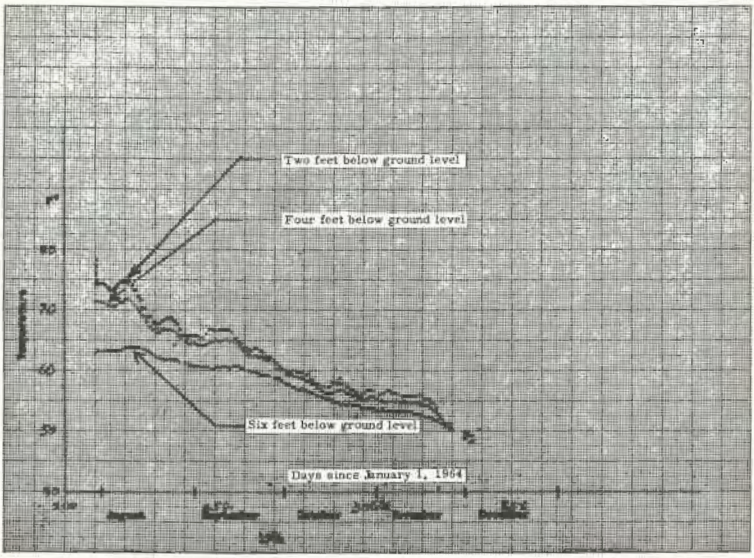


Fig. 22. Zimmerman Ground Temperature

St. Peter Sandstone

A test site was located 2500 ft. into a tunnel of sandstone mine at St. Paul, Minnesota, at a point 140 ft. below ground level. The geological formation here is called the St. Peter sandstone. The test station was isolated from climatological variables, and was found to be the most stable site in the Minneapolis area. Tilt records show that this site has a stability of one arc second for a one-year period, but it is subject to disturbances from large earthquakes. Fig. 23 is a photograph of the test site in the sandstone mine tunnel at St. Paul. Fig. 24 is a long period test record of this site. Fig. 25 is an enlarged section of this record showing the effect of the April, 1964, Alaskan earthquake.

Fig. 23. St. Peter Sandstone Test Site

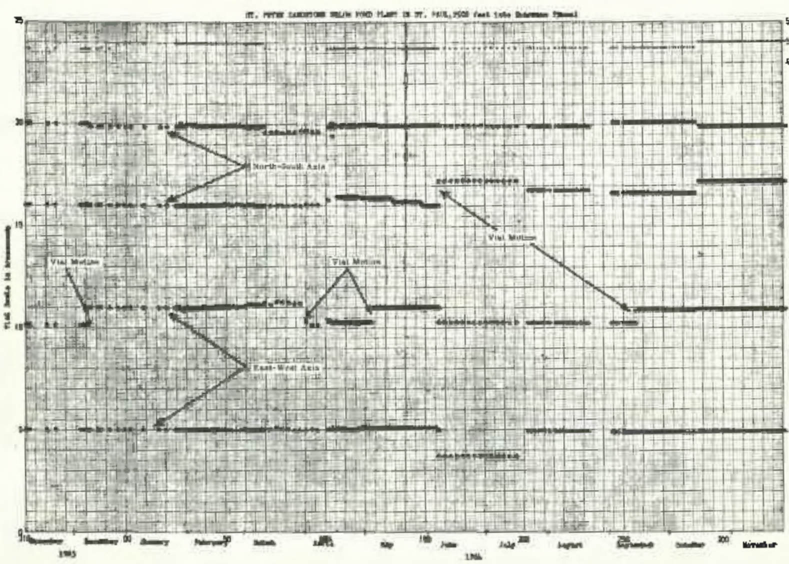


Fig. 24. Test Results St. Peter Sandstone Test Site

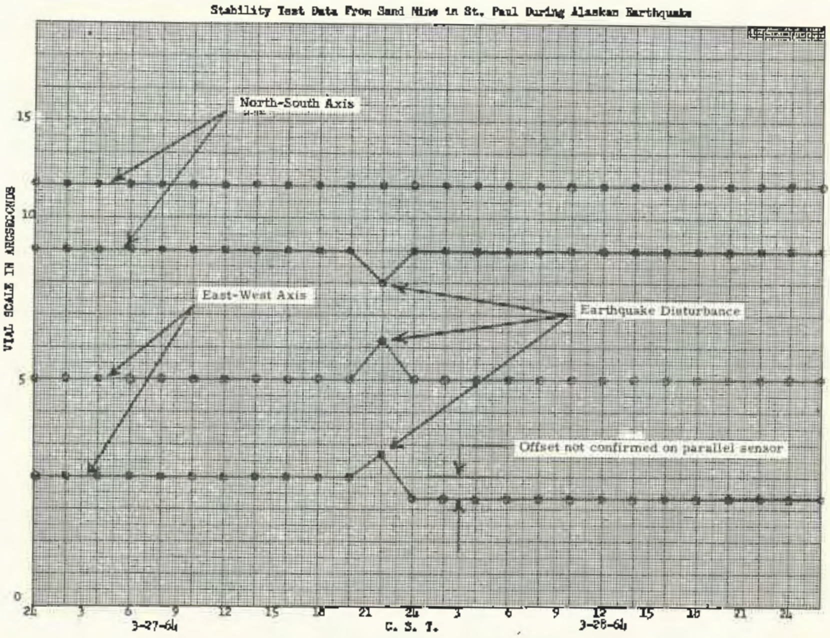


Fig. 25. Alaskan Earthquake Effects
at St. Peter Sandstone Test Site

Granite Outcrop at St. Cloud, Minnesota

Tilt measurements were taken at a granite outcrop in St. Cloud, Minnesota. The instrument mounting plate was placed directly on a horizontally ground surface of the rock. The tilt records showed a pronounced tilt about the east-west axis of approximately 17 arc seconds which varied in an annual cycle correlated to seasonal temperature changes. The tilt about the north-south axis did not have the pronounced cyclical variations. The maximum peak-to-peak tilt about this axis was approximately 3 arc seconds. The cause of the pronounced tilt about the east-west axis was found to be stratified thermal gradients in the rock. The irregular topography of the site caused differential solar heat absorption in the rock outcrop.

Fig. 26 is a photograph showing the irregular topography of the St. Cloud site, and Fig. 27 is a record of tilt measurements taken. It shows that a solid rock formation may not be the best location for a test pad.

Fig. 26. Granite Outcrop St. Cloud, Minnesota

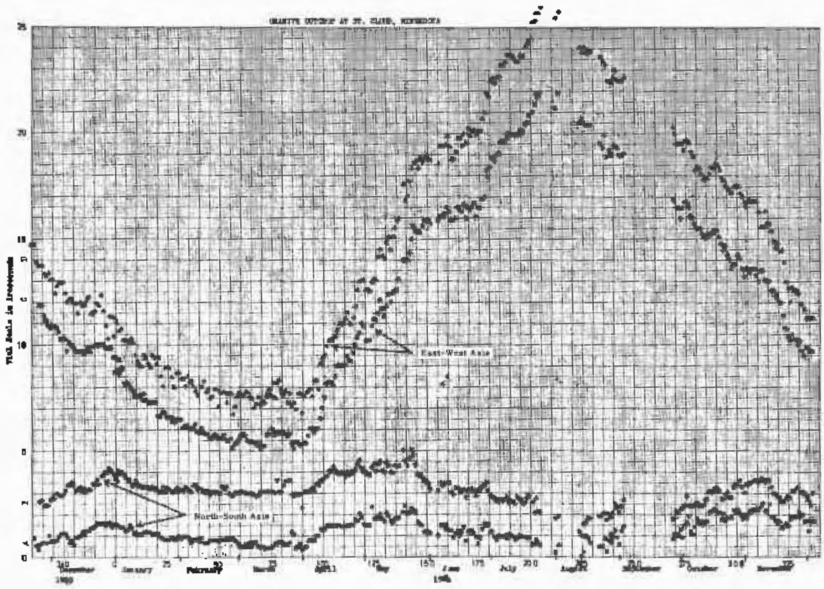


Fig. 27. Tilt Measurements at St. Cloud Granite Outcrop

Baraboo, Wisconsin Quartzite

Data was taken at Baraboo, Wisconsin, for a period of 9 months. The geological formation at this site is quartzite. Test readings showed a tilt of the slab between August, 1963, and February, 1964, of 5 arc seconds about the north-south axis, and 5 arc seconds about the east-west axis. The tilt about the north-south axis was gradual, while the tilt about the east-west axis peaked in January. It was again believed that the irregular topography of the site produced differential heat absorption, and that the consequent thermal heat gradients caused the tilts. Fig. 28 is a record of the tilts at Baraboo.

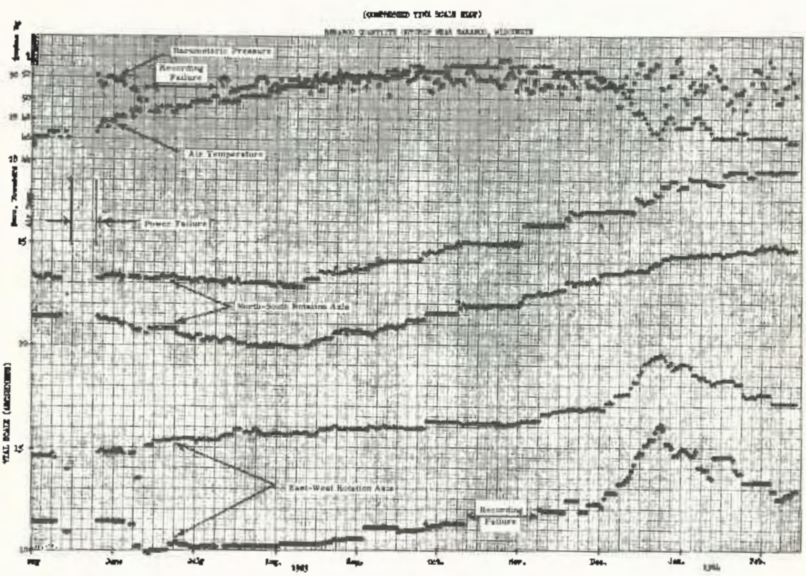


Fig. 28. Tilt Measurements at Baraboo,
Wisconsin Quartzite Outcrop

Recommendations in Honeywell Paper

The recommendations in Mr. Berg's paper for optimum conditions for a test pad can be summarized as follows:

Construct a circular concrete slab located in an underground chamber with at least a 50-ft-thick uniform overlay of unconsolidated, homogeneous soil of adequate load bearing capacity in a flat, isolated, treeless area; provide constant air temperature and humidity and a false floor for supporting operating personnel around the test pad.

TEST PAD IN AN URBAN AREA

Many inertial guidance test laboratory locations are not favored by geography and/or geophysical environment. One of these is the M.I.T. Instrumentation Laboratory in Cambridge, Massachusetts. Its location in a city on the Atlantic seaboard introduces extremely difficult environmental disturbances which prevent the construction of simple slab-on-ground test platforms for modern motion-sensing instruments.

In approaching the problem of providing a test platform for Cambridge, a study was made to determine all of the known physical sources of disturbances. (Ref. 3 through 23) This study places the sources in the following categories:

1. Motions of the earth
   - a. Earthquake waves
   - b. Earth tides
   - c. Microseismic waves
   - d. Earth tremors and local earth disturbances
   - e. Wandering of the poles
   - f. Precession of the polar axis
   - g. Change in speed of rotation of the earth
2. Thermal distortion of buildings and surrounding ground
3. Subsidence of building on soft foundation material
4. Acoustic noise
5. Local temperature changes
6. Variations in humidity, changes in barometric pressure, dust and wind
7. Stray electromagnetic fields

Some of the general characteristics of stray disturbances are discussed in a paper (Ref. 24) presented at the AIAA Test Pad Stability Symposium held in Dallas, Texas, on February 19 and 20, 1964. The multiplicity of disturbances and difficulty of isolation for all six degrees of motion of the ground made it necessary to narrow the isolation problem to the design of a stable test pad for calibration of gyroscopes. One of the studies made at the Instrumentation Laboratory to determine the general effects of stray disturbances on gyro tests was reported in Ref. 25. This study indicated that the:

1. high and low frequency linear stray motions had minor effects on the precision of gyro test results
2. high frequency angular stray motions also had minor effects
3. low frequency angular stray motions (ground tilts) were the most serious causes of errors in gyro test results.

Based on these results, the decision was made to build an experimental test platform isolated from low-frequency stray ground tilts. This specification required maintaining the platform level to within 0.2 seconds of arc (1 micro-radian) in order to minimize the effect of ground tilt on the results of gyro calibrations.

Geotilt Recorder

In order to design a stable platform within the required accuracy, measurments had to be made to determine the maximum amplitudes of low-frequency ground tilts. These measurements were made by a Geotilt Recorder, (Ref. 26) a new instrument developed at the Instrumentation Laboratory. The Geotilt Recorder, Fig. 29, has four major components:

1. Primary sensing unit and ionization transducer
2. Electronic galvanometer-amplifier
3. Esterline-Angus recording ammeter
4. Decker Delter transducer excitation source

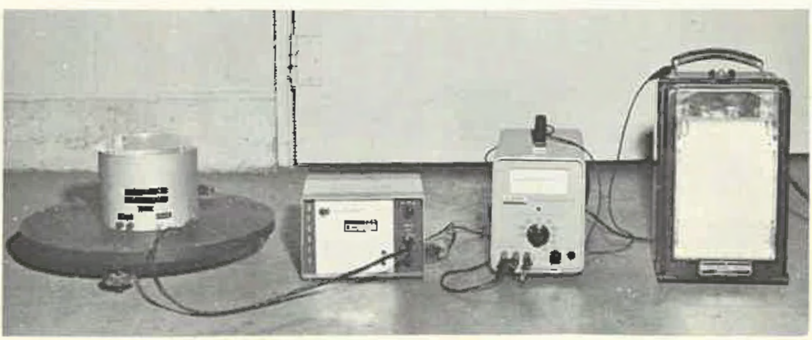


Fig. 29. Geotilt Recorder in Operating Position

The relationship of these major components is shown in Fig. 29. The primary sensing unit, Fig. 30, is a liquid level vial to which three capacitor electrodes are attached. These electrodes are connected in a differential mode to the ionization transducer, which is energized by the excitation oscillator. When a downward ground tilt occurs, the bubble of the level vial is displaced from null. This displacement is detected as a capacitance change between the electrodes, and causes the ionization transducer to produce a positive d-c voltage. An upward ground tilt will produce a negative d-c voltage. The d-c output voltage of the ionization transducer is directly proportional to the capacitance change at the electrodes, and is monitored by the galvanometer-amplifier. The output of the galvanometer-amplifier is an amplified signal that is used to drive the pen of the Esterline-Angus recording ammeter. The plot on the recording ammeter is a continuous record of the displacement of the level bubble caused by ground tilts.

Fig. 30. Geotilt Recorder Primary Sensing Unit Assembly

Floor Tilt Measurements

Measurements of floor tilts were taken at the proposed locations for gyro test facilities. Measurements were made with the level vials parallel to the building walls and also roughly along north-south and east-west directions. The Geotilt Recorder was placed directly on the floor as shown on Fig. 29. Measurements were made for periods varying from a few hours to several days. During these periods, no special efforts were made to divert personnel or other sources of disturbances from the vicinity of the test area. Fig. 31 is a record of one of these tests.

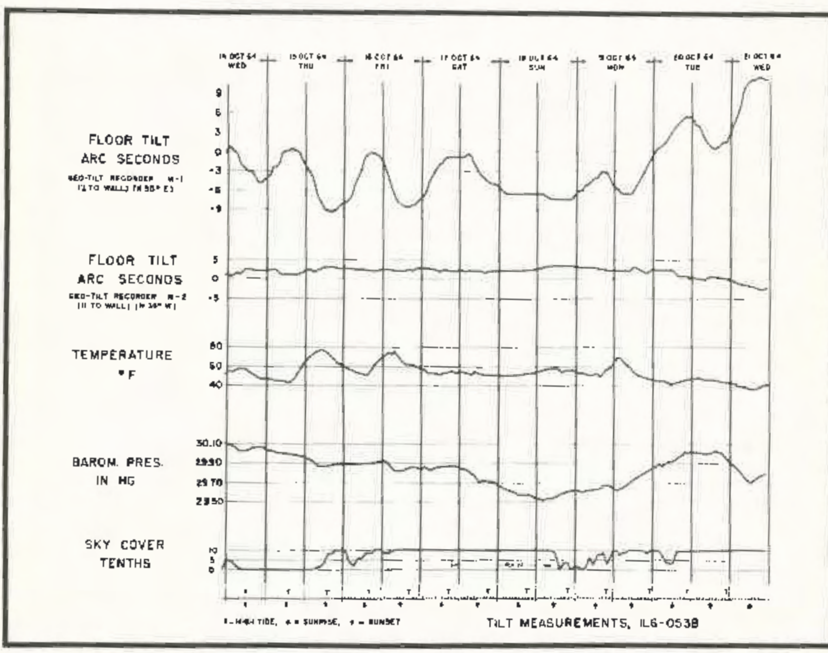


Fig. 31. Record of Floor Tilt Test

Ground tilts occurred at rates varying from several milliseconds of tilt per minute to several seconds of tilt per hour to many seconds of tilt per day. A summary of the measurements of the various test locations indicated that the maximum tilt angle to be expected was plus or minus 20 seconds of arc. However, for the purpose of designing the ground tilt isolation platform, an operating range of plus or minus 120 seconds of arc was chosen for the following reasons:

1. The measurements were taken over a relatively short period of time.
2. Knowledge of local soil conditions, Fig. 32, gained from test borings, indicated that the laboratory building rested on a thick layer of soft blue clay and the possibility of large tilt amplitudes existed between the seasons of the year.

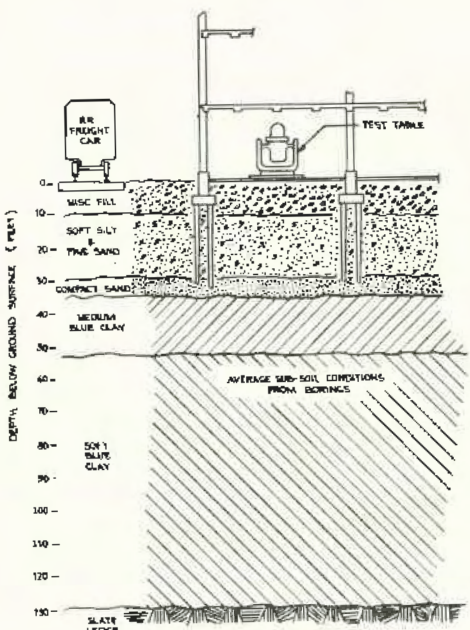


Fig. 32. Cross-section of Soil Conditions Under Instrumentation Laboratory

Experimental Platform

In order to fulfill the specification for ground tilt isolation, an experimental servo-driven test platform was built. It was triangular in design, and consisted of floor plates, micromotion drives, base triangle, intermediate triangle, and mounting plate (Fig. 33). The base triangle is composed of three equal-length aluminum I-beams held together at each corner with top and bottom apex plates. The intermediate triangle is a smaller but similarly unit bolted to the base triangle. The mounting plate is bolted to the intermediate triangle. During tests of the platform, lead blocks were placed on the mounting plate to simulate the weight of a gyro test table with its associated equipment. The two triangles and mounting plate form the superstructure that is supported by three highly-refined servo-driven hydraulic jacks, which are called micromotion drives. Two micromotion drives are actuated by electric servo motors, and the third is operated manually for initial calibration. Heavy floor plates support the micromotion drives.

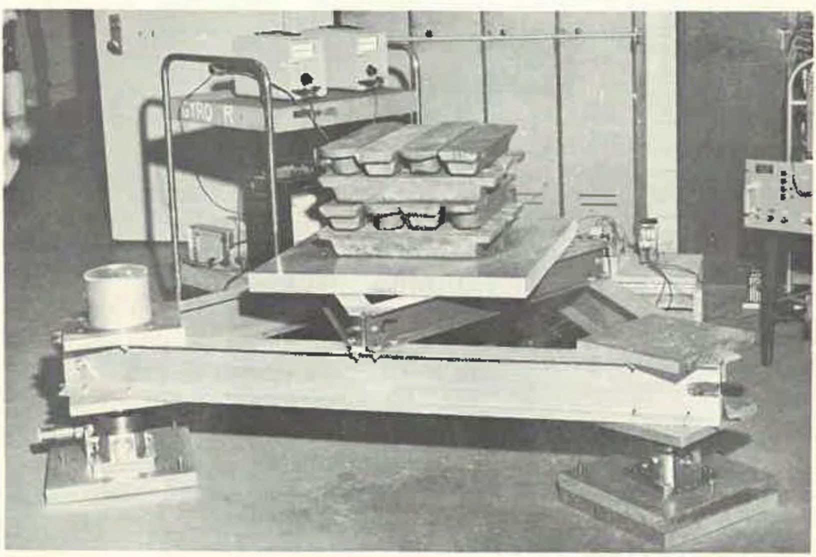


Fig. 33. Experimental Servo-Driven Test Platform

The platform has two horizontal axes of motion, 60 degrees to each other. When a ground tilt causes disturbances about these axes to tilt the platform, two separate servo systems activate the micromotion drives that return the platform to a level position. Each servo system consists of a Geotilt Recorder located on a top apex plate, a servo amplifier, and a servo motor that activates the corresponding micromotion drive. Fig. 34 is a block diagram of a servo system for one horizontal axis. One Geotilt Recorder is aligned with its level axis parallel to one horizontal axis of the test platform. This alignment makes the level vial insensitive to motions about this horizontal axis but sensitive to the other horizontal platform axis. The second Geotilt Recorder is aligned with its level vial axis parallel to the other horizontal platform axis.

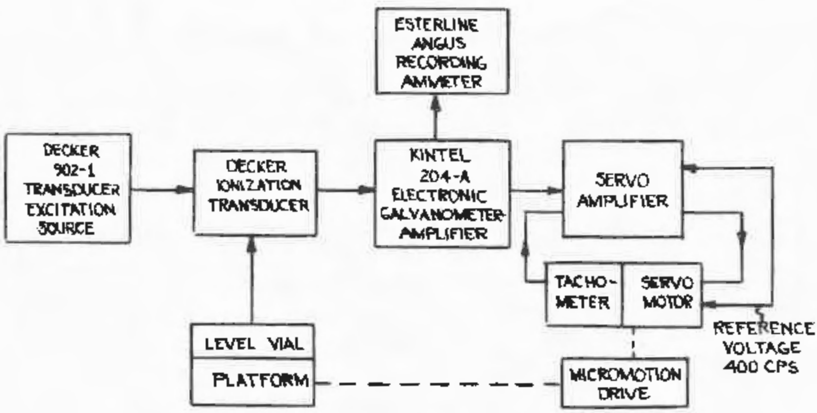


Fig. 34. Block Diagram of Servo Drive for One Horizontal Axis

Operation of Servo System to Control Platform

When a ground tilt occurs, the level bubbles of the Geotilt Recorders are displaced from null. These displacements cause both servo systems to act simultaneously but independently. The action of both servo systems maintains the platform in a level position. The action of one servo system is:

1. The displacement of the bubble in the level vial causes a change in the capacitance between the electrodes.
2. The capacitance change causes the ionization transducer

produce a d-c voltage proportional to the magnitude of the ground tilt.
3. The d-c voltage is monitored by the galvanometer-amplifier, recorded on the recording ammeter, and applied to the input of the servo amplifier.
4. The output voltage of the servo amplifier activates the servo motor of the micromotion drive.
5. The micromotion drive moves the platform back to a level position about one horizontal axis.

Platform Performance

The performance of the experimental ground tilt isolation platform about one horizontal axis of motion was determined by obtaining simultaneous recordings of variations in ground tilt and of variations of the platform position from null. The test setup is shown in Fig. 33. Length of tests varied from a few minutes to several hours. One of the results of these tests is shown in Fig. 35. Test results showed that the platform was stabilized to within 0.2 seconds of arc. It was assumed that stabilization about the other horizontal axis could be maintained simultaneously.

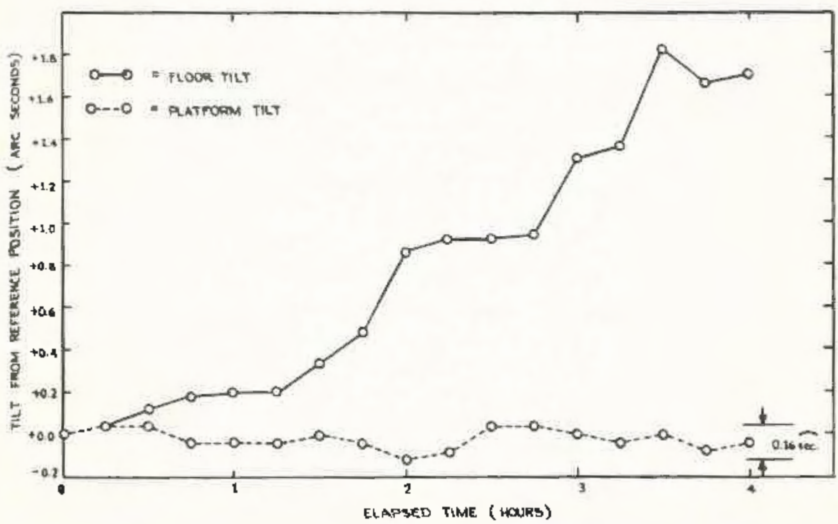


Fig. 35. Performance of Ground Tilt Isolation Platform Compared to Floor Tilt

Gyro Test Table Platform

The success of this experimental platform led to the construction of a ground tilt isolation platform for a large gyro test table which is shown on Fig. 36. This platform is circular in plan to support the circular test table. There are three brackets 120 degrees apart which rest on micromotion drives. Two of these brackets support Geotilt Recorders. The third bracket supports two Decker transducer excitation sources. The gyro to be tested is mounted on the top surface of the test table. A working platform for operating personnel which rests on the floor but is separated from the isolation platform is located along the periphery of the platform. The control and recording equipment for the entire system is housed in a short console.

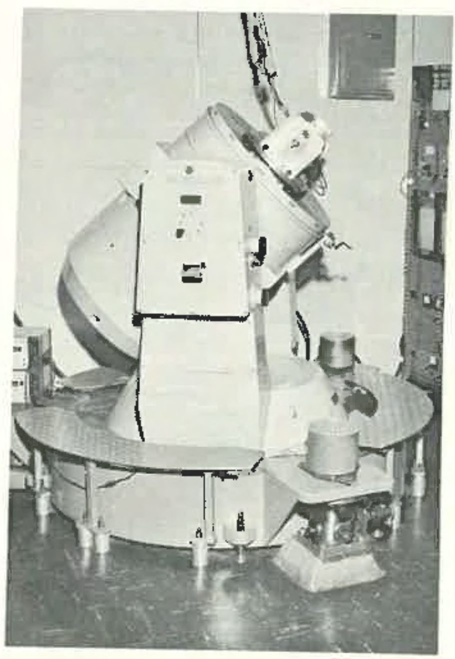


Fig. 36. Ground Tilt Isolation Platform Assembly

Fig. 37 is a close-up view showing the relationship of the Geotilt Recorder, the bracket of the isolation platform, the micromotion drive and the floor. The area between the bottom of the micromotion drive is the grouting that was required to bring the system to a level position because the floor slab was not level.

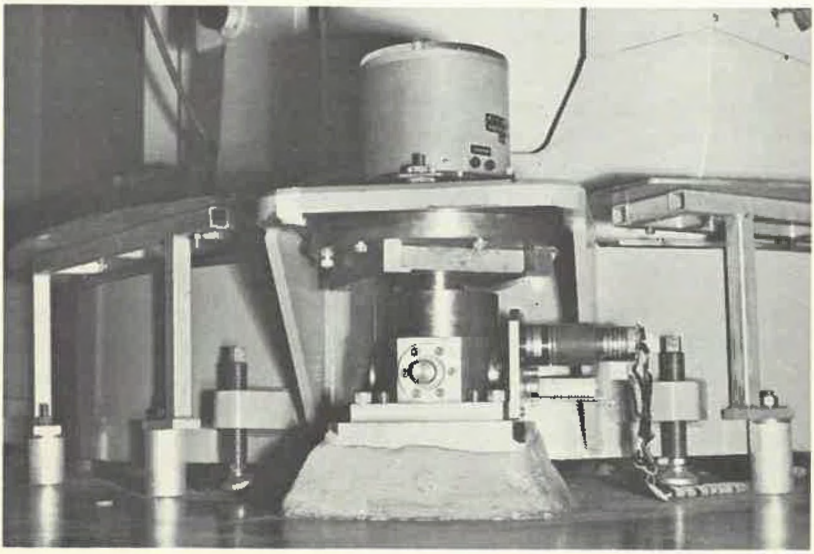


Fig. 37. Close-up View of
Geotilt Recorder and Micromotion Drive

Performance of Gyro Table Stable Platform

Short term tests of the ground tilt isolation platform with both servo systems operating have shown that the platform has remained level well within the specification of 0.2 arc second for long period ground tilts. Long period ground tilts are defined here as those oscillations which require 10 seconds or more for one cycle. Long term performance tests are still being conducted.

CONCLUSION

Since there is no known naturally vibration-free location on the earth, difficult problems are encountered in the construction of stable test platforms for the calibration of modern, highly precise motion sensitive instruments. Specifications for these test platforms vary with the instruments to be tested and the particular requirements of each test facility for geophysical environment and cost of construction.

Gyroscopes and accelerometers, which are the primary motion-sensing components in an inertial navigation system, require the most stable test pads for calibration. These test pads may also be used to calibrate the entire inertial navigation system.

The solutions described here required aid from personnel in many fields such as geology, seismology, soil mechanics and dynamics, materials, structures, meterology, astronomy, feedback control, electronics, fluid mechanics, thermodynamics, and applied mechanics and dynamics.

Although the test pads described in this paper are suitable for testing present-day instruments, they do not have the range of isolation necessary for testing the next generation of instruments. This means that more research with an interdisciplinary approach is required to develop sophisticated platforms for the calibration of projected instruments.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the generosity of Mr. L. O. Mathis of Martin Company, Denver, Colorado, and Mr. Ralph T. Berg of Honeywell, Inc., Minneapolis, Minnesota, in providing their illustrations for portions of this presentation. He also wishes to acknowledge the support received from many members of the M.I.T. Instrumentation Laboratory through helpful discussions and technical assistance.

REFERENCES

1. Mathis, L.O., Stephens, J.R. and Wright, S.C., "The Design and Construction of an Inertial Test Facility," Technical paper presented at American Institute of Aeronautics and Astronautics Guidance and Control Conference, Minneapolis, Minnesota, August 16-18, 1965.
2. Berg, R.T., "Inertial Test Stability Study," Technical paper presented at American Institute of Aeronatuics Guidance and Control Conference, Minneapolis, Minnesota, August 16-18, 1965.
3. Alsop, L.E., Sutton, G.H. and Ewing, M., "Free Oscillations of the Earth Observed on Strain and Pendulum Seismographs," Journal of Geophysical Research, Vol. 66, No. 2, pp. 631-641, February, 1961.
4. Benioff, H., Press, F. and Smith, S., "Excitation of the Free Oscillations of the Earth by Earthquakes," Journal of Geophysical Research, Vol. 66, No. 2, pp. 605-619, February, 1961.
5. Bullen, K.E., SEISMOLOGY, Methuen's Monographs on Physical Subjects, John Wiley & Sons, Inc., New York, 1954.
6. Byerly, P., SEISMOLOGY, Prentice-Hall, Inc., New York, 1942.
7. Creskoff, J.J., DYNAMICS OF EARTHQUAKE RESISTANT STRUCTURES, McGraw-Hill Book Company, Inc., New York, 1934.
8. Daly, R.E., STRENGTH AND STRUCTURE OF THE EARTH, Prentice-Hall, Inc., New York, 1940.
9. Gutenberg, B., "Seismological and Related Data," AMERICAN INSTITUTE OF PHYSICS HANDBOOK, pp. 2-201 to 2-114, McGraw-Hill Book Company, Inc., New York, 1957.
10. Gutenberg, B. and Richter, C.F., SEISMICITY OF THE EARTH AND ASSOCIATED PHENOMENA, Princeton University Press, 1949.
11. Krogdahl, W.S., THE ASTRONOMICAL UNIVERSE, Macmillan Company, New York, 1952.
12. Leet, L.D., PRACTICAL SEISMOLOGY AND SEISMIC PROSPECTING, Appleton-Century-Crofts, New York, 1938.
13. Lina, L.J. and Maglieri, D.J., "Ground Measurements of Airplane Shock-Wave Noise at Mach Numbers to 2.0 and at Altitudes to 60,000 feet," NASA Technical Note D-235, National Aeronautics and Space Administration, Washington, D.C., March, 1960.
14. MacDonald, G.J.F., "The Earth's Free Oscillations," Science, Vol. 132, No. 3491 November, 1961.

15. Maglieri, D.J. and Carlson, H.W., "The Shock-Wave Noise Problem of Supersonic Aircraft in Steady Flight," NASA Memorandum 3-4-59L, National Aeronautics and Space Administration, Washington, D.C., April, 1959.
16. Maglieri, D.J., Hubbard, H.H. and Lansing, D.L., "Ground Measurements of the Shock-Wave Noise from Airplanes in Level Flight at Mach Numbers to 1.4 and at Altitudes to 45,000 Feet," NASA Technical Note D-48, National Aeronautics and Space Administration, Washington, D.C., 1959.
17. National Bureau of Standards, "Time Adjustments in WWV and WWVH Standard Broadcasts," NBS Technical News Bulletin, Vol. 45, No. 9, p. 157, September, 1961.
18. National Bureau of Standards, "NBS and Navy Announce Change in Standard Frequency Broadcasts," NBS Technical News Bulletin,Vol. 46, No. 2, p. 24, February, 1962.
19. National Bureau of Standards, "Adjustment in Phase of Time Signals," NBS Technical News Bulletin, Vol. 47, No. 12, p. 219, December, 1963.
20. Ness, N.F., Harrison, J.C. and Slichter, L.B., "Observations of the Free Oscillations of the Earth," Journal of Geophysical Research, Vol. 66, No. 2, pp. 621-629 February, 1961.
21. Neumann, F., "Earthquake Investigation in the United States," Special Publication No. 282, Revised Editions (1953)(1958) (1962), Coast and Geodetic Survey, U. S. Department of Commerce, Washington, D.C., U. S. Government Printing Office, 1952.
22. Terzaghi, K. and Peck, R.B., SOIL MECHANICS IN ENGINEERING PRACTICE, John Wiley & Sons, Inc., New York, 1948.
23. Wilson, James T., "Underground Shocks," International Science and Technology, Vol. 1, No. 1, p. 47, January, 1962.
24. Tsutsumi, K., "A Ground Tilt Isolation Platform," E-1508, M.I.T. Instrumentation Laboratory Report, Cambridge, Mass., January, 1964.
25. Weinstock, H., "Specification for the Permissible Motions of a Platform for Performance Evaluation of Single-Degree-of-Freedom Inertial Gyroscopes," E-1267, M.I.T. Instrumentation Laboratory Report, Cambridge, Mass., December, 1962.
26. Hopewell, F., King, R., Merenda, F. and Tsutsumi, K., "Precision Level Vial Assembly - Preliminary Model," M.I.T. Instrumentation Laboratory Note C-719, Cambridge, Mass., February, 1962.

THE CHALLENGE
OF
CIVIL ENGINEERING

by
George R. Rich *

(Before a Student Meeting ASCE
Worcester Polytechnic Institute)

Dr. Hooper and Fellow Students -

I use this salutation advisedly, for not so long ago, in one of those office battles that are a normal feature of every live business, one of my partners (now safe in Heaven) referred to me with a shade of deprecation, as "just a damned student". It was a fair and just characterization; I have tried all my life to be just that, and I have never had a more flattering compliment - after fifty years of the toughest sort of in-fighting in the arena, still to have retained the gleaming vision and holy curiosity of golden youth.

You will rightly surmise that I am not a disappointed man. By my standards, I have had an abundant life in civil engineering, and if I were granted reincarnation, I should, without the least hesitancy, again select this profession, for my life work. Professor Hooper has asked me to tell you how I managed to have so enjoyable a career, and incidentally to pass on to you some of the lessons I have learned from my worst mistakes.

In the first place, men who have been so fortunate as to have had our advantages, training and qualifications, are (to borrow from Scripture) veritably a chosen priesthood, and as such, have a reciprocal moral obligation, always and from the very start to stand squarely on the side of professionalism, as contrasted with unionism. The criterion of a true professional is that after acquiring experience, he have his own office or

---

* Vice-President, Chas. T. Main, Inc. and
Partner, Uhl, Hall & Rich

be a principal in a larger firm. His income will then accrue to a large extent from profit, and profit in the economic sense is the reward of the entrepreneur for assuming business risks, hazarding his personal fortune or investing his own capital. Consulting engineering is a highly competitive business, and to obtain work your charges for professional services must fall within generally accepted limits prescribed by the various pertinent manuals of the engineering societies. If you do not control your office production costs efficiently, you will lose money out of your own pocket. There are numerous other risks also involved. To mention just one, no matter how excellent your work you must expect to defend yourself constantly against litigation, which is always expensive. In short, if you are to make any real money (as engineering goes) you must be a business-man and a salesman, as well as a top-flight engineer. But above all, because you are destined to assume business risks and to "shun delights and live laborious days", you are automatically committed from the outset to the side of management.

What of the opportunities in civil engineering today? Because of the magnitude and number of the projects involved, opportunities are unquestionably greater than ever before. Civil engineering, the parent of all but military engineering, is the profession in which you will have primary responsibility for the entire job, perhaps a large hydro or steam power plant, a major bridge, a large industrial plant or a large building. Because of this broad overall knowledge of heavy construction projects, it is the civil engineer who is entrusted with the commission, and the electrical and mechanical engineers more often work under his top direction. So, from your very first day out of school, while you are still serving your internship at the very bottom of the totem-pole, and of course with a decent regard for modesty, never lose sight of the fact that you are a young professional man in training, and let "neither foes nor loving friends, nor the fools that crowd youth" batter your perspective and aspirations down to the artisan level.

I next rush in where angels fear to tread to discuss the first step in your engineering career, your education. To introduce the subject with a decent regard to my own life and limb, I read from this editorial from a current issue of The Electrical World:

"Such Academic Nonsense is Dangerous

'We are completely fed up with the errant, academic nonsense peddled these days by a few professors in major colleges and universities across the country, who hold up the Ph.D. in science as the passport to technical progress and downgrade the engineering profession generally and power engineering particularly.

One professor makes this comment. 'Clearly our society will continue to need people who know how to build transformer and generator stations. There is, however, no room in the faculty and student body in the physical plant of our universities to train these craftsmen.

Vocational schools, two-year junior colleges, or technical four-year schools must take over this task. These schools will also supply those indispensable craftsmen, such as glassblowers, draftsmen, electronic technicians, chemical analysts, etc.'

He feels that our major universities and institutes of technology should concentrate on broad-based education for the most gifted who will assume positions of leadership. He sums up this way: 'The Ph.D. has become the standard terminal for those who aim for leadership in the engineering profession'."

Now to go to the other end of the spectrum. When I was teaching structures at the same professor's school, Harvard, senior students would ask my boss, the late lamented Dean Albert Haertlein, whether they should not proceed to their M. S. and Ph. D. degrees. Without the least hesitation he would invariably tell them that they had finished their school days but that for all the rest of their lives they must study day and night, books and men, in the wide-world school of practical experience.

I naturally incline to the viewpoint of Dean Haertlein. In my opinion the universities have been badgered into catering too much to the missile and nuclear industries with the result that the pendulum has now swung too far in the direction of pure science to the practical elimination of the traditional courses in basic practical engineering design. I am a great advocate of advanced applied mathematics. Get all of this you possibly can, but I still feel that after four years at college a graduate should be able to start earning his living by being able to design a railroad plate-girder, or to participate intelligently in the design of wind-bracing for a sky-scraper, or the design of an arch dam. I have interviewed candidates for research fellowships who were so befuddled with computer technology and system analysis that they could not state in simple intelligible English what research they proposed to conduct.

This brings us to still further controversial ground: the use of digital and analog computers in engineering. In this connection I always think of Sir Winston Churchill awaiting his turn to speak at the MIT convocation in Boston Garden some years back. The previous speaker in a moment of exuberance had stated that computers now could almost think. Sir Winson swallowed hard and said that when that day arrived he hoped to have departed to his eternal rest. Only a few days ago I saw a pertinent caricature in the New Yorker, done in gloomy India ink, showing a beetle-browed scientist with heavy black beard, horn-rimmed spectacles, and a Wellington pipe, frowning at the console of a mammoth computer. The sole decoration in the room was a framed motto in heavy black letters: "To err is unlikely; to forgive is unnecessary." Now in all seriousness, the two lessons are plain. No machine can ever think for us; and while such devices should by all means be used where there is iterative numerical labor, their abuse, if unrestrained, can become an obsession. They then breed a tendency to consign to pedestrian oblivion any study that does not involve the computer and system analysis. One of my bosses, the famous Robert Moses, has the following pertinent comment concerning his selection of the location for the Long Island Bridge:

> "I would be less than frank if I said my own conclusions rest solely on population, industry and

> travel figures, computers, origin and destination surveys, graphs, extensions of lines, and other accepted stereotypes, especially in locating a crossing where it must serve for centuries.
>
> No machine can take the place of the shrewd, prescient and clairvoyant practitioner. Machines do not measure the imponderables, the freaks of human nature, accidents, hell, high water and the King's enemies, time, chance and luck."

By far the greater number of problems confronting the more mature engineer are not iterative. Each one is more likely to be tailor-made and different from its predecessor. For many of these real problems, the time required to program the work would be greater than is available for the entire assignment. Furthermore, no engineer past his early youth would think of doing his own programming, anymore than he would of typing his own letters. So do not become too proficient at this sort of thing or you will never have a chance to do anything else.

I come at last to the most important thing I have to say to you, i.e., how to "stand the gaff". No less a figure than the great Benjamin Fairless, who rose after college from a civil engineering draftsman to Chairman of United States Steel, once wrote that he was certain that several promotions he richly deserved were given to others of lesser merit. Such experiences are the common lot of all of us. How then are we to build up our spiritual reserves to take such injustices in stride, to keep our eyes on the goal we have set and to attain the state of mind of Yeat's Irish Airman:

> "Nor law, nor duty bade me fight,
> Nor public men, nor cheering crowds.
> A lonely impulse of delight
> Drove to this tumult in the clouds."

I read recently a very interesting little book by Lord Wavell. It is called, "Generals and Generalship." It describes the training of British generals, and more than any other quality it emphasizes robustness. The general mustn't go to pieces when things are going against him. When the outlook is black, he has to fight harder than ever. When Lord Wavell was Colonel of a Field Artillery Regiment in a mountain district in India, the authorities would send out new field artillery pieces to be tested. The first step was to take these artillery pieces to the top of the highest cliff in the region and push them abruptly over the edge. If they were still able to fire at all after hitting the ground, the guns were given further and more refined tests. Robustness was what counted, and if your experience is to be anything like mine, you're going to be pushed over that cliff a great number of times, and you must come up fighting harder than ever.

The Iron Duke of Wellington said that the battle of Waterloo was won on the playing fields of Eton. While you are still at college by all means go in for sports of the rough-and-tumble variety, football, baseball, basket-ball and the like. Don't foul, don't flinch, but hit the line hard. You may be advised not to waste your time upon such plebian sports; but to learn instead the finesse of golf and bridge, which will later give you social entree to business success. You may be told to "cultivate the type of people who can get you somewhere." Like old Cyrano, "I mark the manner of these canine courtesies", I hate to see the gilt of an honorable idealism rubbed off at too early an age. My old baseball coach often "bawled me out" in front of my teammates. "You have all the physical attributes of a hitter, but because you go up there with the heart of a chicken, you strike out everytime!" I did not conquer my yellow streak in time to become a great athlete, but with the voice of that old Holy Cross second-baseman continually ringing in my ears, I just had to win out. You simply do not get this essential rough treatment in uppercrust pastimes like golf and bridge.

Another blade of Damacus steel stands ready for your grasp all the days of your life and its potency has best been certified by that great physician and humanist Sir William Osler:

> "Science is organized knowledge and knowledge is of things we see. Now the things that are seen are temporal; of the things that are unseen science knows nothing, and has at present no way of knowing anything.
>
> The man of science is in a sad quandary today. He cannot but feel that the emotional side to

which faith leans makes for all that is bright and joyous in life. Fed on the dry husks of facts, the human heart has a hidden want which science cannot supply; as a steady diet it is too strong and meaty, and hinders rather than promotes harmonious mental metabolism.

> To keep his mind sweet, the modern scientific man should be saturated with the Bible and Plato, with Homer, Shakespeare and Milton; to see life through their eyes may enable him to strike a balance between the rational and the emotional, which is the most serious difficulty of the intellectual life."

The engineers who really stand out in the profession, men like the late Boris Bakhmeteff of Columbia, or Abel Wolman of John Hopkins, have a spellbinding felicity and facility of expression that bespeak life-long love for, and intimacy with the classic world literature. Of course we read the classics principally for delight, not for material advancement; but as you progress in civil engineering you will be called upon more and more to serve as an expert witness in litigation. When you stand on your own resources at the mercy of the opposing lawyer in cross-examination, you will find this facility of expression no handicap.

In conclusion gentlemen, I wish you the best of good luck. May you too have the good fortune always to remain students in spirit for all your days, and in your inevitable hours of trial and testing, may you lift up your hearts to the great lines of Rupert Brooke, written when, homesick in a strange land, he draws renewed courage from the memories of his college days at Cambridge:

"Say do the elm-clumps greatly stand
    Still guardians of that holy land?
The chestnuts shade in reverend dream,
    The yet unacademic stream?
Is dawn a secret, shy and cold
    Anadyomere, silver-gold?
And sunset still a golden sea
    From Haslingfield to Madingley?

And after, ere the night is born,
    Do hares come out about the corn?
Oh, is the water sweet and cool,
    Gentle and brown, above the pool?
And laughs the immortal river still
    Under the mill, under the mill?
Say is there Beauty yet to Find?
    And Certainty, and Quiet kind?
Deep meadows yet, for to forget
    The lies, and truths and pain? - Oh! yet
Stands the church clock at ten to three?
    And is there honey still for tea?"

Do not be too surprised if your ultimate destination is not what you had earlier planned. Recall Cromwell's famous statement that nobody appears to go quite so far as he who does not quite know where he is going. Remember too the wisdom of Ecclesiastes: "I returned and I saw under the sun that the race is not to the swift, nor the battle to the strong, neither yet bread to the wise, nor yet riches to men of understanding nor yet favor to men of skill; but time and chance happeneth to them all."

By modern Madison Avenue standards, our old friend Cyrano de Bergerac with the long nose would have been an abysmal failure; but not even the Admiral at Trafalgar had so glorious a death:

"Hopeless you say
But a man does not fight merely to win!
Yes, all my laurels you have driven away
And all my roses; yet in spite of you,
There is one crown I bear away with me,
And tonight, when I enter before God,
My salute shall sweep all the stars away
From the blue threshold! One thing without stain,
Unspotted from the world, in spite of doom
Mine Own!

And that is ______________________

my white plume ____________________"

ERRATA

Getzler, Z., "The Virtual Differential Settlement Method," Vol. 53, No. 2, April, 1966, pp. 199-218.

1. page 200, line 23, missing "in", should be: "stress distribution in soil" . . . .

2. page 204, line 9, missing "see Fig. 1,"

3. page 207, formula (20), should be: $R_B^o a - R_A^o b = O$,

4. page 209, line 6, should be: $\sigma = \frac{R_B^o}{b^2} = \frac{R_A^o}{a^2}$

5. page 209, line 9, should be: "settlements for $\varkappa = 1$ will be equal."

6. page 210, left formula, line 3 from the bottom, should be:

$$\frac{\beta E_c J_c^{\Sigma}}{3} \left(\sqrt{\varkappa} + \frac{b}{a}\right) \text{ or } \ldots$$

7. page 213, line 10, should be: "the plan and a section with" . . .

8. page 217, Notation Nos. 7 & 8, should be: J_c and J_c^{Σ} (capital letters).

PROCEEDINGS OF THE SOCIETY

BOSTON SOCIETY OF CIVIL ENGINEERS

APRIL 14, 1966 - A joint meeting of the Boston Society of Civil Engineers and the Structural Section was held on this date at the Adams Room of the United Community Service Building.

President Biggs called the meeting to order at 7:10 P.M. After a short business meeting, he turned the meeting over to Chairman Fuller who introduced the speaker of the evening, Mr. Edward H. Barker of Sverdrup, Parcel and Associates, Inc., who spoke on "Techniques of Construction of the Chesapeake Bay Bridge and Tunnel Crossing."

The crossing consisted of four man-made islands, two tunnels, undership channels, causeways, trestles and bridges. The tunnel sections were prefabricated, towed to the site and sunk in dredged trenches. The talk was illustrated by movies.

The meeting adjourned at 8:30 P.M. after a short question and answer period. There were 46 members and guests present.

Respectfully submitted,
F. Hampe, Clerk

OCTOBER 19, 1966 - A joint meeting of the Boston Society of Civil Engineers and the Massachusetts Section of the American Society of Civil Engineers was held this evening at the Student Center Building, Massachusetts Institute of Technology, Cambridge, Massachusetts. The student chapters of the New England colleges were especially urged to attend.

At 6:30 P.M., a dinner was served in the Student Center Building, and delegates from Northeastern University, Tufts

University, Worcester Polytechnic Institute, Norwich University, University of Rhode Island, Merrimack College and the Massachusetts Institute of Technology were present.

President Biggs extended a cordial welcome to the students.

The secretary announced the names of applicants for membership.

President Biggs introduced John J. Cusack, President of the Massachusetts Section of the American Society of Civil Engineers and asked him to conduct any necessary business of ASCE at that time.

President Biggs then introduced the speaker of the evening, Professor Steven A. Coons, of Massachusetts Institute of Technology, who gave a most interesting talk on "Computer Graphics".

A question period followed the talk. Two hundred members and guests attended the dinner and meeting. The meeting adjourned at 8:45 P.M.

Respectfully submitted,
Charles O. Baird, Jr., Secretary

NOVEMBER 16, 1966 - A regular meeting of the Boston Society of Civil Engineers was held this evening at the United Community Service Building, 14 Somerset Street, Boston, Massachusetts, and was called to order by President John M. Biggs, at 7:00 P.M.

President Biggs stated that the minutes of the previous meeting held October 19, 1966, would be published in a forthcoming issue of the journal, and that the reading of minutes would be waived unless there was objection.

President Biggs announced the death of the following members --

Clarence A. Pethybridge, elected a member February 15, 1928, who died in May of 1965.

Frank Marcucella, elected a member May 17, 1948, who died July 11, 1966.

George L. Newman, elected a member April 14, 1949, who died September 20, 1966.

Louis I. Dexter, elected a member April 14, 1949, who died June 4, 1966.

The secretary announced that the following had been elected to membership November 16, 1966:

Grade of Member: A. M. Zahirul Alam, Robert A. Barrows, Armando P. Del Campo, Richard E. Cavanaugh, Jerome J. Conner, Jr., Carmin V. DiFilippo, David E. Heard, Frank E. Perkins, Eudell G. Whitten, Melvin W. Witham, Asaf A. Czilbash.

Grade of Junior Member: George R. Allen, Ernest R. Artus, Craig E. Barnes, Harold S. Costa, Neil K. Daykin, John M. Dewsnap, Mark T. Donohoe, Paul A. Fournier, Lawrence J. Hughes III, Thomas D. Hurley, Peter J. Majeski, Anthony J. Masse, William A. McKenzie, Paul M. O'Conner, Frederick J. Stavinski, Jr.

President Biggs requested the secretary to present a recommendation of the Board of Government to the Society for action. The President stated that this matter was before the Society in accordance with provisions of the by-laws and notice of such action published in the ESNE Journal dated November 14, 1966.

The Secretary presented the following recommendation of the Board of Government to the Society for action to be taken at this meeting:

MOTION: "To recommend to the Society that the Board of Government be authorized to transfer an amount not to exceed $7000 from the principal of the Permanent Fund to the Current Fund for current expenditures".

On motion duly made and seconded it was VOTED "that the Board of Government be authorized to transfer an amount not to exceed $7000 from the principal of the permanent Fund to the Current Fund for current expenditures."

President Biggs stated that final motion on this matter would be taken at the January meeting of the Society.

President Biggs introduced the guest speakers of the evening, Mr. Cruce Graham and Mr. E.A. Picardi, of Skidmore, Owings and Merrill, Chicago, Illinois, who gave a most interesting talk on "Design and Construction of the John Hancock Center in Chicago." The talk was illustrated with slides.

A discussion period followed the talk.

Thirty-three members and guests attended the dinner preceding the meeting and seventy five members and guests attended the meeting.

Respectfully submitted,
Paul A. Dunkerley,
Secretary (Pro-tem)

STRUCTURAL SECTION

DECEMBER 14, 1966 - A regular meeting of the Structural Section of the Boston Society of Civil Engineers was held in the society rooms, 47 Winter Street, Boston, Massachusetts. The meeting was called to order by Chairman Fuller at 7:20 P.M.

Chairman Fuller announced the minutes of the last meeting would be published in a forthcoming issue of the Journal.

The chairman introduced the speaker, Mr. Mace Bell, Coordinator of Research and Development, American Institute of Steel Construction, who spoke on "Structural Connections - Their Design and Economics." He gave examples and comparative costs of various types of connections. The talk was illustrated by slides.

The meeting adjourned at 8:50 after a question and answer period.

There were 46 members and guests present.

Respectfully submitted,
F. Hampe,
Clerk

SANITARY SECTION

OCTOBER 5, 1966 - A meeting of the Sanitary Section was held in the society rooms and was called to order at 7:00

P.M. by Chairman Robert L. Meserve. Reading of the minutes of the previous meeting (the June Annual Outing) was waived.

Chairman Meserve then introduced Dr. Alvin S. Goodman, Associate Professor of Civil Engineering at Northeastern University, who presented a very interesting paper entitled, "Mathematical Model For Water Pollution Control Studies." By use of an electronic computer, Dr. Goodman has set up a program of a hypothetical stretch of a river including several communities with water supply, sewage treatment, recreational use, population densities areas, river quality standards for each. Using this program, sets of assumptions are fed into the computer and the resulting effects on river quality, degree of treatment and other data are almost instantly obtained. Two methods have been used, (1) Trial and Modification -- whereby several sets of conditions are fed into the computer and the resulting effects of each compared, and (2) Automatic Optomizing Routine -- whereby the types of water and sewage treatment processes are varied in an attempt to arrive at the required degree of treatment that would satisfy the river quality standards at minimum cost. The value of such a program utilizing an electronic computer lies in its ability to almost instantaneously solve a host of problems with varying assumptions and data that by present long hand means would require months of work.

A lively question and answer period followed Dr. Goodman's presentation. Fifty-two members attended the meeting. Adjournment was at 8:30 P.M.

Respectfully submitted,
Charles A. Parthum,
Clerk.

HYDRAULICS SECTION

NOVEMBER 2, 1966 - A meeting of the Hydraulics Section of the Boston Society of Civil Engineers was held in the society room, 47 Winter Street, Boston, Massachusetts. The meeting was called to order by Mr. Athanasios A. Vulgaropulos, clerk of the section, substituting for the chairman, Mr. Nicholas Lally.

Mr. Vulgaropulos introduced the speaker of the evening, Mr. Peter A. Larsen, Associate Professor of Civil Engineering at the Worcester Polytechnic Institute. Professor Larsen spoke on "Head Losses Causes by an Ice Cover in Open Channels" and showed slides of prototype investigations in Sweden. The speaker discussed field observations and measurements of head losses in certain river sections with and without ice cover, the roughness of the underside of ice cover where a ripple pattern is formed and the results of the above investigations.

The meeting had an attendance of 16 and was adjourned at 9:00 P.M.

Respectfully submitted,
Athanasios A. Vulgaropulos,
Clerk.

NEW!!

UNIVERSAL
**DEMOLITION
HAMMER
& COMPACTOR**

Hits with 300,000 lbs. impact
Fully Hydraulic

One Man Operation

- **Break or Score Pavement**
- **Compact Ditches, etc.**
- **Break Rocks**
- **Demolition**

Strikes 30 to 60 times per minute, 120 degree work arc, creeps from 14 feet per minute up, breaks up to 4000 sq. ft. of 9" concrete pavement per hour, manual—semi automatic—full automatic control, fully roadable. Interchangeable tools for maximum usage.

FOR COMPLETE DETAILS

Earthdrill, Inc.

Phone (213) 698-3711
12121 Los Nietos Rd.
Santa Fe Springs, Calif. 90670

Woodward, Clyde, Sherard and Associates
Consulting Engineers and Geologists in Applied Earth Sciences
EXECUTIVE OFFICES
420 MONTGOMERY ST., SAN FRANCISCO
277 PARK AVENUE, NEW YORK
Engineering Offices
Oakland, San Jose, Los Angeles, San Diego, Orange California, Philadelphia, St. Louis, Kansas City Denver, Clifton, New Jersey

---

ALBRIGHT & FRIEL INC.
CONSULTING ENGINEERS
Water, Sewage, Industrial Wastes and Incineration Problems, City Planning Highways, Bridges and Airports, Dams, Flood Control, Industrial Buildings, Investigations, Reports, Appraisals and Rates
THREE PENN CENTER PLAZA
PHILADELPHIA 2, PA.

Hardesty & Hanover
Consulting Engineers
Bridges — Fixed & Movable
Highways
Special Structures
Design, Supervision
Inspection, Valuation
101 Park Ave. New York, N.Y. 10017

CHARLES R. VELZY ASSOCIATES, INC.
Consulting Engineers
Water Pollution Control — Water Supply
Drainage
Solid Waste Disposal — Industrial Wastes
500 Fifth Ave., New York, N.Y.
Mineola, N.Y. New City, N.Y.

Greeley and Hansen
Engineers
Water Supply, Water Purification, Sewerage, Sewage Treatment, Flood Control Drainage, Refuse Disposal
14 E. Jackson Blvd. Chicago, Ill. 60604
233 B'way, New York, New York 10007

JOHN J. KASSNER & CO., INC.
Consulting Engineers
250 BROADWAY
NEW YORK, N. Y., 10007
212-BA, 7-6445

COMMITTEES
1966-1967

NOMINATING COMMITTEE

Edwin B. Cobb

Harry P. Burden
Percival S. Rice
Marcello J. Guarino
James P. Archibald
(Term expires March, 1967)

George G. Bogren
Max D. Sorota
Darrell A. Root
Harry R. Feldman
(Term expires March, 1968)

PROGRAM

John M. Biggs, *Chairman*
Harry L. Kinsel
Harl P. Aldrich, Jr.
Charles O. Baird, Jr.
Ronald E. Bucknam
Robert L. Meserve
Robert L. Fuller
Casimir J. Kray
Nicholas Lally
Robert J. Van Epps

PUBLICATION

Simon Kirshen, *Chairman*
Ronald E. Bucknam
Robert L. Meserve
Robert L. Fuller
Casimir J. Kray
Nicholas Lally
Robert J. Van Epps

LIBRARY

Joseph Capone, *Chairman*
George W. Hankinson
John C. Adams, Jr.
Myle J. Holley, Jr.

HOSPITALITY

Arthur Quaglieri
William H. Parker, 3rd, *Chaidman*
Thomas Tsotsi

DESMOND FITZGERALD AND HERSCHEL AWARD

Edward C. Keane
George G. Bogren, *Chairman*
Saul Namyet

SANITARY SECTION AWARD

George M. Reece
Joseph C. Lawler, *Chairman*
George G. Bogren

STRUCTURAL SECTION AWARD

Saul Namyet
Max D. Sorota, *Chairman*
Harl P. Aldrich, Jr.

HYDRAULICS SECTION AWARD

Lawrence C. Neale
Richard F. Dutting, *Chairman*
Ronald T. McLaughlin

TRANSPORTATION SECTION AWARD

Thomas C. Coleman
Alexander J. Bone, *Chairman*
Edward C. Keane

CONSTRUCTION SECTION AWARD

Robert J. Hansen
Herman G. Protze, *Chairman*
Frank J. Heger

SUBSOILS OF BOSTON

Ronald C. Hirschfeld, *Chairman*
Rev. Daniel Linehan
Harl P. Aldrich, Jr.
Donald T. Goldberg
Floyd E. Brown

MEMBERSHIP CENTRAL COMMITTEE

Frank J. Heger, *Chairman*
Robert L. Fuller
Robert L. Meserve
Casimir J. Kray
Nicholas Lally
Robert J. Van Epps

AUDITING COMMITTEE

Myle J. Holley, Jr.
Frank J. Heger

INVESTMENT COMMITTEE

John M. Biggs
Paul A. Dunkerley, *Chairman*
Harry L. Kinsel

ADVERTISING COMMITTEE

Richard F. Dutting, *Chairman*
Ronald E. Bucknam
Walter M. Newman
Francis T. Bergin

JOINT LEGISLATIVE COMMITTEE

James L. Dallas, *Chairman*
Charles A. Parthum
Ralph M. Soule

COMMITTEE ON PROFESSIONAL CONDUCT

E. Sherman Chase, *Chairman*
William L. Hyland
Francis S. Harvey
John H. Hession

PUBLIC RELATIONS COMMITTEE

Richard F. Battles
John F. Flaherty, *Chairman*
Percival S. Rice

JOHN R. FREEMAN FUND COMMITTEE

Leslie J. Hooper, *Chairman*
Thomas R. Camp
George R. Rich
Clyde W. Hubbard
Lee M. G. Wolman

RALPH W. HORNE FUND COMMITTEE

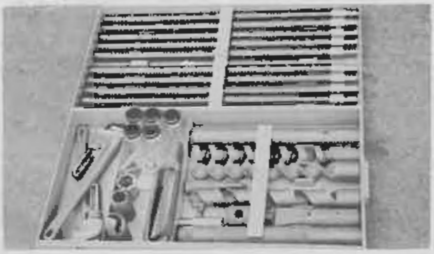
George G. Bogren
William L. Hyland, *Chairman*
Ernest A. Herzog

acker

KNOWN ROUND THE WORLD!

THE ACKER DRILL COMPANY

Manufactures a complete qualtiy line of drilling equipment such as: S
sampling Tools, Rotaty Earth Augers, Diamond Core Drills, Diamond
& Core Barrels, Drilling Accessories and Supplies.



ACKER SOIL SAMPLE KIT

Unequaled collection of tools for recovering samples from practically any meterials except rock. Hand carrying case is included. Write for bulletin No. 26.

ACKER VANE SHEAR

The vane shear is available in a kit or torque head sembly for obtaining accurate in-place shear value cohesive soils on the spot. Write for Bulletin No. 70

ACKER MOTORIZED CATHEAD

Aluminum tripod derrick with sheave and powerful 4-cycle cathead hoist. An ideal combination for: Soil sampling, Driving casting, Water well work, Preliminary survey, Standard penetration tests and any lifting operations up to 500 lbs. Write for Bulletin No. 20.

ACKER DENISON CORE SAMPLER

(Patent No. 2403,002) Acker Denison Soil Sample B obtains undisturbed quality cores from rare earths, c amic clays, soft but difficult formations. Accepted mining and soils engineers around the world. For de write for Bulletin No. 1100.

ACKER HILLBILLY CORE DRILL

The Hillbilly is a tried and proven core drill with unlimited versatility to perform a host of jobs. Acker core drills are available in a variety of models to meet your particular need. Write for Bulletin No. 29.

ACKER ALL PURPOSE AUGER

This versatile auger is capable of handling soil sampl mineral prospecting and earth augering to 100 ft. De Mounts on flat bed carrier. Easy to operate. Write fo free Bulletin No. 40.

ACKER DRILL COMPANY, INC.
P. O. Box 830
Scranton, Pennsylvania, 18501

FREE

Acker has available free of charge a complete series illustrated bulletins . . . write us on your needs.