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K-65 SILOS STUDY & EVALUATION REPORT

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DEPARTMENT OF ENERGY
FEED MATERIALS PRODUCTION CENTER
FERNALD, OHIO

K-65 SILOS STUDY & EVALUATION REPORT

FOR:
WESTINGHOUSE MATERIALS COMPANY OF OHIO

prepared by:
CAMARGO ASSOCIATES, LIMITED
CINCINNATI, OHIO

FEBRUARY 1986

CAMARGO ASSOCIATES, LIMITED
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February 25, 1986 W.O. 568-01

Mr. Steve Heisler
Westinghouse Materials Company
of Ohio
P.O. Box 398704
Cincinnati, Ohio 45239

Subject: Subcontract No. S-1161
K-65 Silos Study & Evaluation

Dear Mr. Heisler:

We are pleased to provide you with the final report on the condition of the K-65 Silos.

Included with this letter is the following:

Copies	Description
12	Report
4	Appendix A, B, C
4	Appendix D & E
4	Appendix F - 1 of 2
4	Appendix F - 2 of 2

This report includes all revisions and comments made during the review period and review meeting of January 17, 1986.

The purpose of our work was to indicate the structural condition of the two K-65 Silos. We have concluded that the base slab and walls have a maximum life expectancy of 5 to 10 years. A portion of the silo's dome is considered structurally defective and we have recommended remedial actions be undertaken.

The installation of a temporary self-supporting cover over the structurally defective area has already been accomplished.

Very truly yours,

CAMARGO ASSOCIATES, LIMITED

Edwin G. Hauggel
Project Engineer

John A. Sample
Project Manager

W. Christopher Luther, Jr.
Associate-in-Charge

enclosures

K-65 SILOS STUDY & EVALUATION
FOR THE
FEED MATERIALS PRODUCTION CENTER
VOLUME I
SECTIONS I THROUGH IX.

FEBRUARY 25, 1986

Prepared by:

CAMARGO ASSOCIATES, LIMITED
Architects-Engineers-Planners
Cincinnati, Ohio

K-65 SILOS

STUDY AND EVALUATION

FERNALD, OHIO

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SECTION I - INTRODUCTION

As part of the Feed Materials Production Center (FMPC) Five Year Environmental, Safety and Health Improvement Plan (FY-1986 through FY-1990), studies are being initiated to determine effective alternatives for the processing, removal and ultimate disposition of radium-bearing residues. These residues principally resulted from the processing of pitchblend ore for uranimum and are currently maintained in two concrete silos (K-65) onsite.

Essential to the implementation of the K-65 silo residues processing and disposal studies is a thorough testing and analysis of the two K-65 storage silos to determine their structural stability and to identify any potential structural problem areas that will require remedial action. In addition to performing a testing and analysis evaluation of the silos, it was necessary to determine the allowable stresses (dynamic and static) that may be applied to each of the silos without causing failure or significant damage.

Camargo Associates, Limited (CAL) in association with Muenow and Associates and Soil & Materials Engineers, Inc., was retained by National Lead of Ohio, Inc. (NLO) to provide inspection, testing, and evaluation of the two K-65 storage silos. The evaluation of the two silos is based upon existing static loads and probable dynamic loads. This work was authorized under NLO, Inc., Subcontract No. S-1161.

This report presents:

* Field and laboratory data collected during the geotechnical and nondestructive testing evaluation.
* Structural analysis and investigation of the K-65 Silos both as originally designed and as they currently exist.
* Finite element modeling and analysis results of the major components of the silos.
* Identification of problem areas of the silos.
* Recommended actions to remedy identified problem areas.

SECTION II - SUMMARY OF THE APPROACH

FMPC supplied CAL with copies of the original silo design drawings dated 1951. CAL was also given the maintenance file on the K65 storage silos, containing available historical documents on remedial and maintenance actions on the silos since 1963.

The silo investigation consisted of three phases. The first phase was a computer analysis of the storage silos depicting the original "designed" condition of the silos, based on the drawings supplied by FMPC. Only static loads were considered for this phase.

The second phase consisted of field work on and around the silos. This portion was divided into three areas. These were; the soil exploration study, the "Echo Pulse" system testing of the silo dome, wall and base slab, and the "Ground-Radar Survey" of the earth embankment surrounding the K-65 silos.

The third phase was a detailed computer analysis of the silos utilizing the results of the field work. Both static and probable dynamic loadings were considered during this phase.

A site plan (Figure 2-1) showing the location of the K-65 silos at the FMPC facility is shown on the following page.

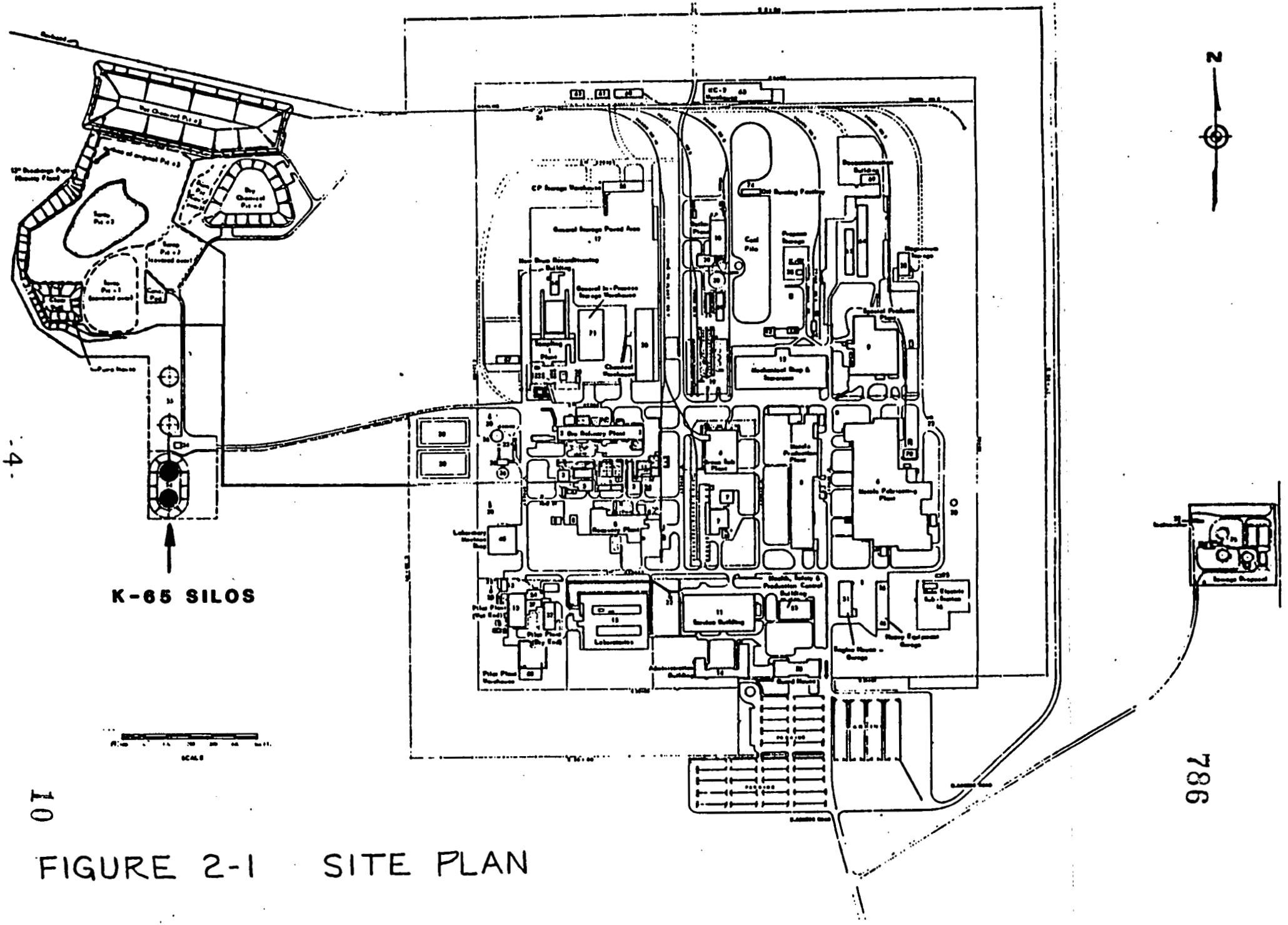


FIGURE 2-1 SITE PLAN

EXECUTIVE SUMMARY

SECTION III - CONCLUSIONS AND RECOMMENDATIONS

Conclusion

The structural stability of the K-65 storage silos can be subdivided into three separate areas: dome, wall, and base slab.

1. Dome

The dome is the most critical of the three areas since it is the one most highly exposed to tornado suction and external forces. The major portion of the domes on both the north and south silos are capable of supporting their own weight plus a live load of 20 psf. However, the structural load capacity of the center portion on each of the two domes is questionable for any load and the domes are considered to be defective due to the thinning and cracking of the concrete during the past 30 years. Therefore, considering the placement of three feet of earth cover over the dome should not be considered as a solution for radiation shielding.

2. Walls

The walls of the silo are in a stable state since the contents and the earth embankment counteract each other, therefore eliminating tensile stresses. If either the contents or the earth embankment are removed separately, the silo would be placed in an unsteady state with high stresses in the wall. This would be especially true for the case when the earth berm would be removed first. Therefore, it is recommended that any removal of the contents of the silos or removal of the earth berm be done silmultaneously.

3. Base Slab

Because of the earth embankment around the silos, the base slabs could not be directly investigated to determine their exact condition. The data from the field investigation of the silo walls indicates the condition of the footing directly under the silo walls. Therefore, it is concluded that the slab would show evidence of cracking and deterioration of the cement matrix.

Since the base slabs are supported by the earth embankment on the sides and the soil below, and since no evidence of settlement has appeared in the walls or domes, the

slabs seem to be in a stable condition. They should remain so as long as the earth embankment and the silo contents are removed simultaneously.

4. Earthquake

Based on the dynamic finite analysis performed in the event of an earthquake, the silos should experience some additional cracking at the base of the wall, and in the base slab. However, the dome being relatively non-rigid, would move with the ground motions with no resultant large stress increase and consequently would not suffer any additional distress. Therefore, it is our opinion that the silos would remain standing and still function as a container for the K-65 material.

Recommendations

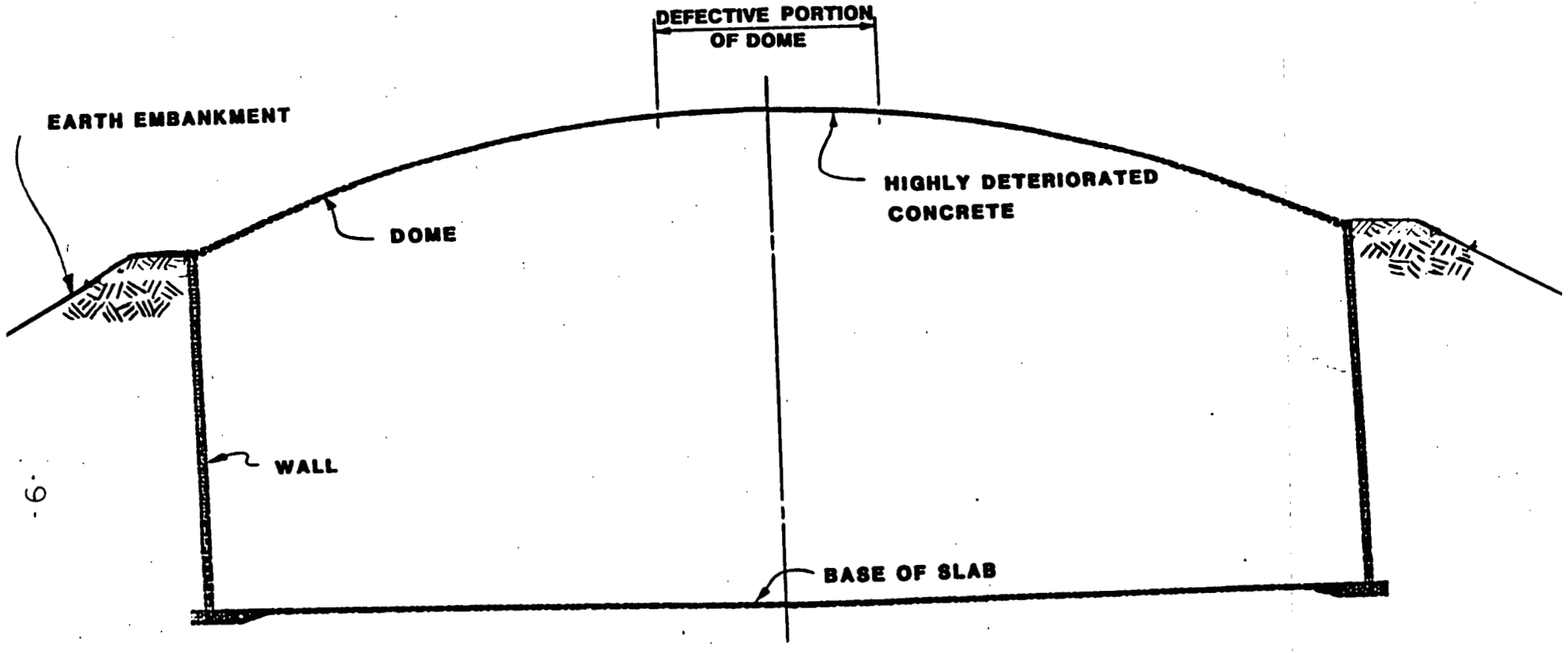
The condition of the walls and base slab of the two silos would be considered acceptable from a structural standpoint in that they are in a stable state and are unlikely to collapse at this time. However, during the field investigation the silo walls were found to contain many cracks and spalled areas. The life expectancy of the silo walls and slab is a maximum of five to ten years.

Based on the analyses and tests performed as part of this work, the condition of each of the silo domes must be considered structurally defective, with no life expectancy. It is recommended that one of two actions should be undertaken immediately as part of the remedial program.

1. Remove and properly dispose of the silos' contents and repair, replace or demolish the domes.

As stated previously in this report, the contents of the silos and the earthen embankments around the silos must be removed simultaneously to avoid damage or collapse of the silo walls and/or slab.

2. Pending the removal of the silos' contents, install a temporary self-supporting cover or enclosure over the structurally defective portion of the silos' dome (See Figure 3-1) to keep any load off this area.



TYPICAL SECTION THRU

K-65 SILO

FIG 3.1

SECTION IV - HISTORY OF THE K-65 SILOS

The two K-65 silos, located on the west side of the Fernald site, were constructed in 1955. The silos are used for storage of radium bearing residues, a byproduct of uranium ore processing. The silos are of cylindrical concrete construction, 80 feet in diameter and approximately 27 feet high. The silo domes were originally designated to be 4 inches thick at the center tapering to 8 inches at the dome-wall edge.

The walls were post-tensioned reinforced with 0.162 inch diameter wire stressed to 140,000 pounds per square inch (PSI) (with assumed 30% loss, for a design stress of 100,000 PSI). These post-tensioning wires were covered by a 3/4 inch thick gunite coating. The minimum 28-day compressive strength used was 4500 PSI for the dome and walls and 3000 PSI for the floor and footing. The maximum allowable soil pressure was 4000 pounds per square foot (PSF). The silo design information was supplied to CAL in the original silo design drawings dated 1951.

The silos were designed to be loaded with the metal oxides in slurry form at a maximum rate of 8000 gallons per day. The radioactive residues were allowed to settle and the water was

decanted, leaving a sludge with a density of 100 pounds per cubic foot (PCF) and angle of repose of 0 degrees. The maximum allowable height of solid material was 23 feet and the water level was limited to a maximum height of 25 feet.

In 1963, the silos were showing signs of exterior surface deterioration. Large areas of spalling occurred in the exterior surface gunite coating, particularly on the north silo, leaving post-tensioning wires exposed to weather. Subsequently, patches of the wires became severely corroded and broken. Various options were investigated as remedial actions for the silos. Repairs began in 1964 by first chipping away all loose gunite material and then patching the surface with a 3/4 inch coat of cement mortar. After the gunite was repaired and a waterproofing sealant was applied to the external silo walls, an earth embankment was built to the top of the wall on a one and one-half to one (1-1/2:1) slope (horizontal: vertical). The earthen embankment was to provide relief from tensile stress within the walls by counterbalancing the load from the internal contents, since the broken wires were not replaced. A soil was chosen with roughly the same density (125 PCF) as the contents of the silos (100 PCF). Two additional purposes of the embankment were to provide weather protection and to reduce the radon emission from the silos.

In subsequent years problems with soil erosion on the silo embankment were frequent. The eroded areas were repaired, but with heavy rains the problem reoccurred. In 1983, the embankment was enlarged to achieve a 3:1 slope. No further evidence of large scale erosion has occurred.

SECTION V - INITIAL SILO ANALYSIS PHASE DISCUSSION

The initial phase of the silo investigation was a computer analysis of the silo to be used as a tool to guide the field investigation. The original silo design was analyzed through a finite element computer program so that any critical points in the design of the silos could be given a more detailed investigation during the field investigation portion of the project.

The finite element program used in the initial analysis as well as the static analyses in the final analyses is a capability of the Micas Computer System produced by the Rand Corporation in Dallas, Texas. The program was run on a Wang 2200 minicomputer

There were two different finite element model types analyzed. One was an axisymmetric model, while the other was a three-dimensional shell subdivided into its three principle components, i.e., the arched dome roof titled 568DOME, the wall titled 568WALL, and the base slab titled 568SLAB. All models used the initial design of the K-65 silos as their respective design criteria. In this way, the actual history of the loading of these silos could be modeled.

The axisymmetric model titled 568TANK3 uses a two-dimensional model in the X-Y global coordinate system and rotates it 360 degrees about the Y axis, forming a shell of revolution. All components of the model are rotated, including nodes, elements and loads. Using data supplied by FMPC, the model shown in Figure 5.1 was generated.

The individual shell models were used as a back-up to the axisymmetric model. The axisymmetric model was a complete system therefore, the effect of the joints is analyzed in whole by the computer. When they are separated into individual components, the continuity of the connections are lost. The continuity can be estimated through the use of springs. The spring constraints were generated by applying a unit load to each reaction component and determining the amount of translation or rotation generated. Once the springs were applied, the results could be compared to the axisymmetric model.

Both silos are of cylindrical concrete construction, using normal weight concrete (150 PCF) with a 28-day compressive strength of 4500 PSI. A Young's Modulus of Elasticity of 3.82 X 10^6 PSI, Poissons Ratio of 0.2, and a thermal coefficient of 5.5 X 10^6 were the resulting properties used in the analysis.

The connections of the dome-to-wall and wall-to-base slab were modeled as originally designed. The wall is anchored to the base slab by #4 dowels (0.5" diameter) at 12 inches O.C. (on center). The dowels and a 7 inch X 2 inch deep shear ledge provide translational support, but the joint is free to rotate. This condition is effectively modeled as a hinge. The axisymmetric computer model connects the wall to the base slab at a single node, (Node #229, see Figure 5.2) producing the hinged condition. The dome-to-wall connection is more rigid, with #4 dowels at 12 inch O.C. at both the inside and outside face of the dome. This is modeled on the computer as a fixed condition, with the transition from wall to dome occurring over three nodes and eight elements (see Figure 5.3).

The loads on the silo model were dead load (DL), live load (LL), contents (C), and the earth embankment (E). The dead load consisted of the self-weight of the concrete and the compressive hoop force of the post-tensioned wires. The concrete is normal weight (150 PCF) and was modeled as a gravity load on every element in the model. The post-tensioning wires were modeled as inward acting loads on the outside face of the walls. These loads were based on the number of wires per 8 inches of wall height and it was assumed that every wire was stressed fully to its design stress of 100,000 PSI.

The live load on the model was a 20 PSF snow load over the dome surface.

The contents of the K-65 silos are modeled based on the following assumptions derived from the historical data supplied to CAL by FMPC:

1. The material was placed in the silo as a slurry, therefore, the angle of repose was taken as zero degrees.
2. The unit weight of the material was 100 PCF.
3. The contents over time have not shrunk back from the walls.

The contents were loaded at a maximum rate of 8000 gallons per day. However, for this load case the maximum possible loading was used. This was set at a maximum height of 23 feet above the floor slab for solids and 25 feet for water.

The embankment load was applied to the full height of the silos' walls and its density was taken as 125 PSF. The soil characteristics were assumed to be granular enough to prevent skrinkage away from the wall. A lateral pressure coefficient of 0.4 was used. This results in a silo wall lateral soil load at any point that is 40 percent of the total soil weight above the point.

The load combinations applied to the model paralleled the actual loading of the two K-65 silos. The first combination analyzed was DL (dead weight of structure) alone, resembling the as-built condition of the silo. Next the snow load was applied to the empty silo (DL + 20 PSF snow load (LL)). Then the silo was filled to its capacity (DL + Contents Properties (C)). Lastly, the embankment was built around the filled silos (DL + C + Soil Properties (E)).

The extreme events such as earthquake, tornado, and additional earth fill over the dome were left for the final analysis phase.

Results of the initial phase did not indicate any areas in the dome that were abnormal, therefore, no additional areas in the dome were investigated. Results of the analysis showed that the membrane stresses were very low and that all elements were in compression.

The results of the analysis of the silo walls under dead load plus contents showed that the elements in the bottom half of the silo were in tension. Normally, the post-tensioned wires would keep the concrete constantly under compression under the various load cases.

The initial phase analysis utilized the design stress value in the wires per the original drawings. The wires were initially stressed to a higher level, but the difference accounts for loss of pre-stressing force due to such factors as creep in concrete and steel, anchorage transfer loss, and initial deformation of the concrete. The stresses generated through the analysis indicate that the same wires were being stressed beyond the design stress value, therefore causing additional deformation of the wires and some cracking of the concrete since concrete in tension (modules of rupture) has approximately 10% of its compressive strength capacity.

Results for the base slab show low stress levels, with the top of the slab in compression and the bottom in tension, indicating bending in the base slab. The backup analysis was a flat plate model, (568SLAB) which was then analyzed by a design post processor to check the amount of steel reinforcement required. The results showed that the reinforcing steel provided in the original design was adequate.

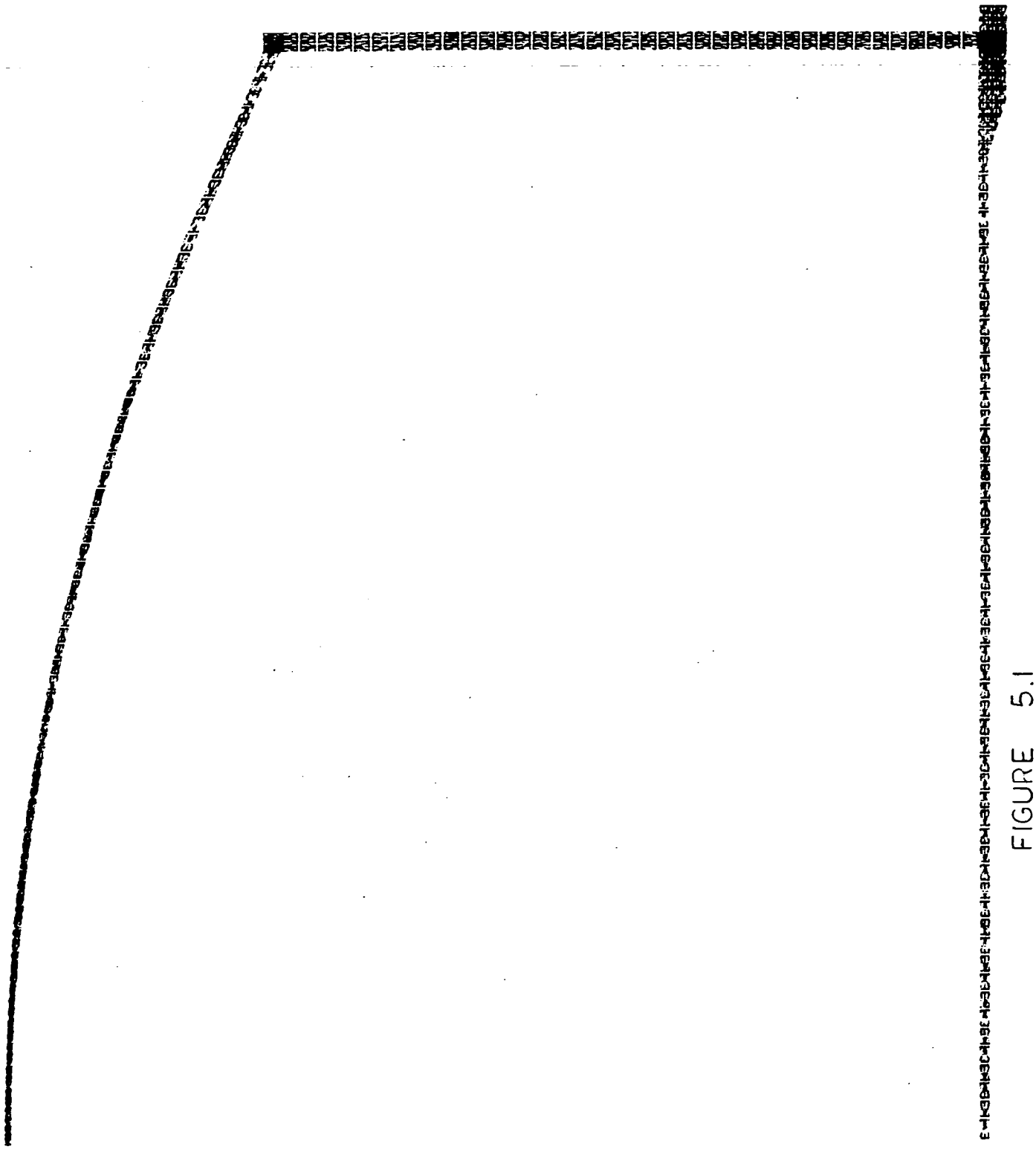


FIGURE 5.1

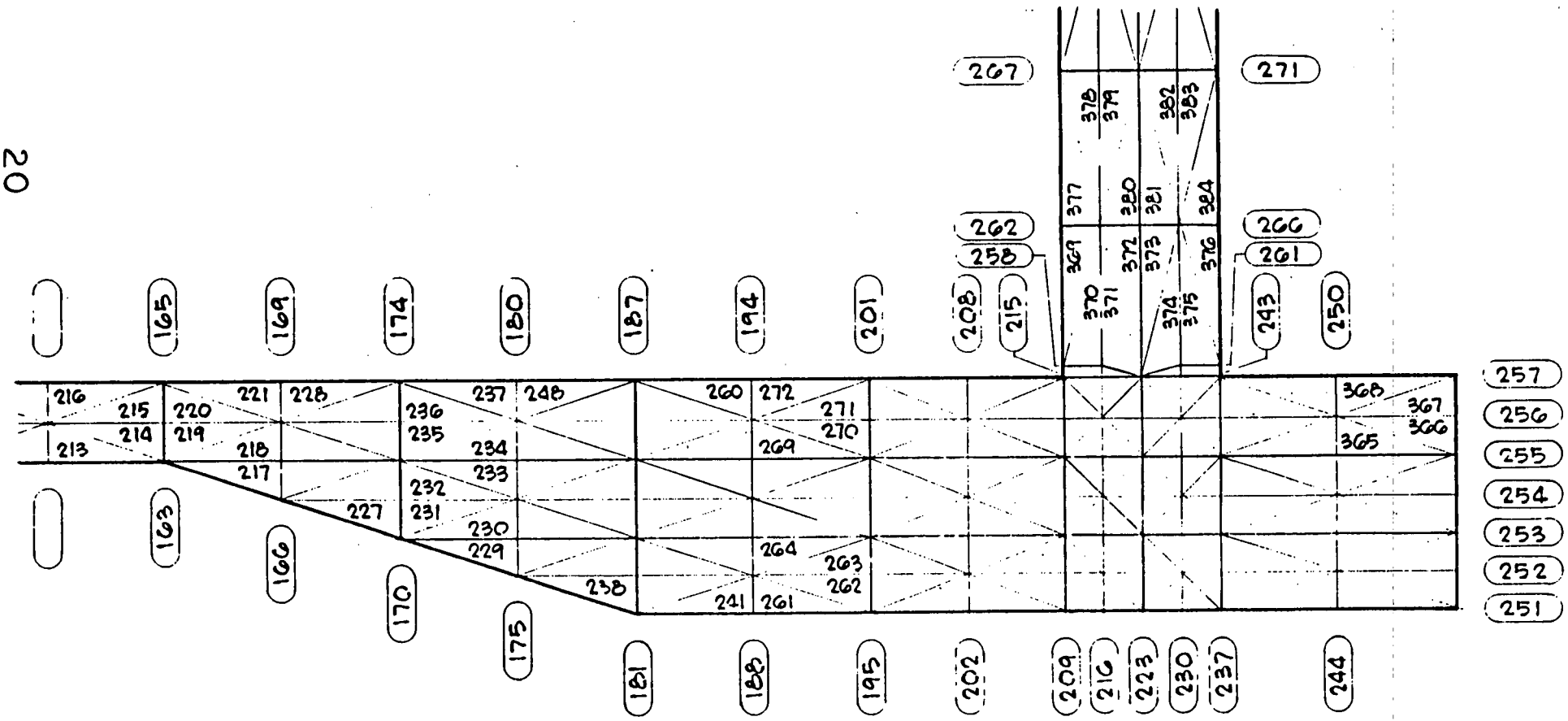

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FIGURE 5.2

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FIGURE 5.3

SECTION VI - SUMMARY OF FIELD INVESTIGATIONS

From August 19 to September 13, 1985, the field investigation of the K-65 silos was performed by personnel from Soil & Material Engineers, Inc. (S&ME), and Muenow and Associates, Inc. (M&A), under the supervision of Camargo Associates, Limited. The purpose of these tests was the determination of the existing condition of the silos. This information was necessary for assembling the third phase computer model reflect the current K-65 silos site conditions.

The field studies performed fall under two categories. First was the test necessary to determine the subsurface soil conditions around the silos. The second was the nondestructive tests needed to determine the physical condition of the silos.

S&ME performed standard analytical tests on samples recovered from test borings to determine the soil index and engineering properties of the soil. These tests included natural moisture content, Atterberg Limits, grain size analysis, dry density and unconsolidated and undrained one-point triaxial compressive strength.

S&ME also measured the dynamic properties of the subgrade using crosshole seismic techniques. These dynamic soil properties were used in the generation of a synthetic earthquake for the final model.

A detailed report on the investigation performed by S&ME is given in Appendix D.

M&A used subsurface ground radar to analyze the K-65 silos berm embankment. Ground radar utilizes reflected radio waves to determine the uniformity of the soil, locate and measure the size of voids, and locate and measure the size of any moisture concentrations and/or associated riverlets. This information can be used as an aid in determining if any leakage from the silos had occurred.

M&A also performed pulse-echo nondestructive testing on the silos for the purpose of determining concrete quality. The pulse echo technique uses reflected sound waves to locate and determine any changes in the concrete. From this data, the following information can be determined:

1. Compressive strength of the concrete.
2. Thickness of the dome and walls.
3. Percentage of reinforcement remaining in the dome and walls.
4. Condition of the remaining reinforcement.
5. Condition of the foundation directly under the wall.

A detailed report on the investigation performed by M&A is given in Appendix C.

SECTION VII - FINAL SILO ANALYSIS PHASE DISCUSSION

The final phase of the silo analysis focused on the silos in their present condition. Information obtained from the field investigations was utilized to model the silos in their present state. From this analysis, the structural integrity of the silos could be determined based on normal loading patterns as well as extreme cases such as earthquake and tornado.

There were two different types of axisymmetric finite element models analyzed during this phase. The first was a static model using a triangular element type. This model is similar to the one described in Section V. The following information, taken from the M&A report (Appendix C) was used to model the actual physical characteristics of both the north and south silos:

1. Actual dome thicknesses and concrete compressive strengths.
2. Actual wall thicknesses and concrete compressive strengths.
3. Percentage of post-tensioned wires remaining in the walls.

The M&A report showed general thinning of the silo domes with sharply undulating interior surfaces. Associated with the thinner dome sections was concrete that was cracked and exhibiting various stages of deterioration. Widely varying silo characteristics cannot be modeled axisymmetrically, because any small inconsistency (i.e., a hole) will be revolved 360 degrees about the Y axis. This would, if it were a hole, form a discontinuity in the silo structure at the location of the hole. To avoid this, average values were computed for various wall heights and dome radial points. The following trends were observed from M&A's report:

1. The silo dome thickness and general concrete quality deteriorated progressively, moving radially from the dome-wall edge to the dome top.

2. The silo wall thickness, concrete quality, and remaining percentage of horizontal preload wires have deteriorated progressively, moving from wall top to bottom.

3. The silo base slab could not be measured by the techniques used in the field investigation. Therefore, the slab was modeled as it was originally designed.

Tabulation of field data is presented in Appendix E and values used for the final model are shown in Figures 7.1 and 7.2.

Load combinations applied to the final model for analysis are:

1. DL
2. DL + LL
3. DL + LL + CONT. (C)
4. DL + LL + EMBANKMENT (E)
5. DL + LL + C + E
6. DL + [FILL (F) + E (Rev.)] + C
7. DL + TORNADO PRESSURE (TP) + CONT (C) + EMBANKMENT (E)
8. DL + TORNADO SUCTION (TS) + CONT (C) + EMBANKMENT (E)
9. DL + C

Where DL = Dead weight of structure

LL = 20 PSF snow load

C = Contents Properties: = 100 PCF, Angle of Repose = 0 degree

E = Soil Properties: = 130 PCF, Ka = 0.5 (value from S&ME report, Appendix D)

Fill (F) = 3 ft. soil cover over dome: = 390 PCF

E (Rev.) = E + surcharge produced by extra 3 ft. of soil

TP = 401 PSF acting downward on exposed dome surface

TS = 432 PSF acting upward on exposed dome surface

The second axisymmetric model analyzed was a dynamic model using the ASHSD2 program. ASHSD2 is a finite element computer program written by Sukumar Gosh and Edward Wilson in 1969 at the University of California, Berkley. The program was designed to be used for the dynamic stress analysis of axisymmetric structures under arbitrary loading. The current version was modified and improved by C. J. Lin of United Engineers & Contractors, Inc. in 1975. The program may be used to analyze axisymmetric structures subjected to arbitrary axisymmetric or non-axisymmetric loadings. The axisymmetric continum may be idealized either as a shell of revolution or as an axisymmetric solid or a combination of both. This program will therefore enable one to evaluate the dynamic response of soil-structure system. The program will also perform a time history analysis (i.e., a step-by-step integration through the time domain).

The dynamic model was run using the same silo model data as the static model and the dynamic stresses developed would be added to the static loads to obtain the following load combination:

10. DL + LL + C + E + Dynamic

The dynamic properties were taken from S&ME report in Appendix D. The stresses from the earthquake analysis are tabulated in Appendix F, and must be added to the static stresses.

SECTION VIII - DISCUSSION OF RESULTS

CONCLUSIONS

The final phase of this report is concerned with the structural condition of the silos and the recommended allowable loads the silos can support at the time of this report.

The following finite element programs were used as the computer models of the north and south K-65 silos. The north silo model is labeled "568TANK7" and the south silo model is labeled "568TANK8." Both of the above models are based on the axisymmetric technique. The above models were used for the static load cases as well as the tornado pressure and suction loadings. The time-history earthquake loadings were modeled on an axisymmetric shell program called "ASHSD2."

Static Results - Dome

The domes of the silos are the most critical part of the entire silo structure system. Under the static loads, as they exist at the time of this report, Load Combination 5, the

maximum X or hoop stress is 41 PSI tension near the center, and 190 PSI compression near the edges.

Another area that will limit the capacity of the dome would be its buckling stability or the ability to withstand the compressive loads without bending out of plane. Since the domes have thinned down, and the Modulus of Elasticity has also reduced, (concrete quality), the critical buckling load will also be reduced. The critical buckling load of the domes would be approximately 284 PSF for the existing concrete outside of the critical area. Using a safety factor of 4, the allowable load would be 71 PSF. This value would be valid for the 3 inch thick area of both domes.

Inside the critical 20 foot diameter area, the buckling load of the domes would be approximately 104 PSF for the existing concrete. Using a safety factor of 4, the allowable load would be 26 PSF. This is based on a 2 inch thick area of the concrete. As is readily evident, the allowable buckling load is nearly the same as the dead weight of the dome itself (25 PSF for a 2" thick dome).

Results indicate that the portion of the dome outside the center 20 foot diameter area is not capable of supporting much more than its own self weight and snow load. This amounts to 38 PSF for the slab and snow load at 20 PSF for a total of 58 PSF. Since the buckling load capacity of the 3" slab is 71 PSF, this leaves a net of 13 PSF. Ignoring the center 20 foot section, the total additional load that could be supported by the 3" (and greater thickness) dome area is 11,000 pounds.

Since the center 20 foot diameter portion is only rated at a load equal to its own dead weight, and since the critical buckling load of the dome would be greatly affected by irregularities in the concrete surface, it is recommended that all additional live loads should be kept off that area.

Based on the computer Load Combination 6, which is for the 3 feet of earth cover over the dome (390 PSF), the stress is 1000 PSI tension near the center and 300 PSI tension near the edge. Based on the low buckling capacity, the additional earth load should not be considered.

Combined Static and Dynamic Results - Dome

Based on the results given in Appendix F, the domes are not stiff enough to resist movement. Therefore, the domes will move along with the ground motions induced by the synthetic earthquake (Appendix D). From the results tabulated in Appendix F there are very little additional stresses induced (2 psi) into the concrete domes from the "defined" synthetic earthquake.

Static Results - Wall

The static load combinations indicated the following:

1. Under dead load, contents, and earth embankment, Load Combination 5, the stress levels indicate that the wall is under compression and as such, is in a stable condition. The maximum stress is 515 PSI compression. This case represents the present state of the silo.

2. Under dead load and contents, Load Combination 9, the stress levels indicate that the silo would be in a dangerous state with stress in the remaining post-tensioning wires at 114,000 PSI. This case would

depict the removal of the earth berm with the contents of the silos left in place. The design stress in the wires was 100,000 PSI and the above value of 114,000 PSI would be above the original design limits.

3. Under dead load, live load, and embankment, Load Combination 4, the stress levels are 1200 PSI compression. This combination would represent the removal of the silo contents without removal of the earth berm. Because of the magnitude of the stresses, this combination should not be utilized.

4. Therefore, based on Load Combinations 4 and 9, the removal of the contents and/or embankment should both be accomplished simultaneously. This case would represent Load Combination 1.

Combined Static and Dynamic Results - Wall and Base Slab

Under the dynamic loading (Appendix F), two areas are in the greatest amount of stress. These locations are at the dome-wall intersection, Node 121, and at the base of the wall (approximately) 1'-0" above the footing, Node 50.

At the dome-wall intersection, the combined element stresses show the wall is in compression or just slightly into tension. The tension value is less than the modulus of rupture, and therefore will not introduce additional cracking. At the base of the wall, the stress levels even under combined static and dynamic loads would indicate that cracking and load redistribution would occur. Since the wall is surrounded on either side by the soil or the silo contents, the wall cannot collapse. Therefore, the wall would still function as a containment for a dry material. The combined results show that the slab experiences enough tension to crack the concrete during the dynamic loading. The result of this cracking would be to redistribute the loads since:

1. The slab is surrounded by soil and the silo contents and a collapse could not occur.

2. The program calculates the stress in the direction of the applied load. Therefore, additional stress can be taken at other points along the wall outside of maximum point shown.

CONCLUSIONS

In conclusion, our analysis indicates the following:

1. The base slab and walls at this time are structurally stable under the existing static loads being applied to them and should continue to be so for approximately 5 to 10 years.

2. If either the contents of the silos or the silo embankment are removed, they must be removed simultaneously or failure of the walls or base slab could result.

3. The center 20 foot diameter portion of the dome top is structurally unsound for a load greater than the existing static dead load and no life expectancy can be assigned to it.

4. The application of a three foot thick earth fill load on the dome or the application of tornado suction or pressure loads, would cause a structural failure of the dome.

5. The application of the "defined" synthetic earthquake would induce some additional cracking in the base slab and at the base of the silo wall, but the silo would still be in a serviceable condition. The dome would be unaffected, and should simply move with the ground motions.

6. If a cover is chosen to cover the center portion of each silo dome, the weight should be as light as possible and in no case should it be greater than that allowed by the buckling capacity of the dome (approximately 11,000 pounds).

SECTION IX - ANALYSIS VERIFICATION

The original report including calculations, analysis and methods used was reviewed by a registered professional engineer on our staff, who was not originally associated with this project.

Another finite element program was run using the same type of elements (membrane stress only, no bending) as used in the original work. This program was "SAP IV" which is "A Structural Analysis Program For Static and Dynamic Response of Linear Systems" by Bathe, Wilson and Peterson 1973, Revised, from Earthquake Engineering Research Center.

Results of the verification confirmed the condition of the silo, and the findings stated within. The following references were used during the investigation:

ACI-318-83 - "Building Code Requirements for Reinforced Concrete".

ACI SP-28 - Concrete Thin Shells

ACI SP-67 - Concrete Shell Buckling

Design of Prestressed Concrete Structures T.Y. Lin & Ned H. Burns - 3rd Edition - Wiley.

Theory of Prestressed Concrete by Michael Chi & Frank A. Biberstein. 1963 Prentice-Hall, Inc.

Thin Shell Concrete Structures by Billington. 1965 McGraw-Hill.

Finite Element Computer Programs

MICAS - Produced by the Rand Group, Dallas Texas

ASHSD2 - "Dynamic Stress Analysis of Axi-Symmetric Structures Under Arbitrary Loading" by Ghosh & Wilson - Revised 1975 - Earthquake Engineering Research Center.