Seismic Reevaluation and Retrofit Criteria

For: Yankee Atomic Electric Company
By: Cygna Energy Services

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SEISMIC REEVALUATION AND RETROFIT CRITERIA

For
Yankee Nuclear Power Station
Rowe, Massachusetts
Prepared for
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1.0 INTRODUCTION

Yankee Nuclear Power Station was designed before the current technology and codes had fully evolved. In the last decade, the state-of-the-art of earthquake engineering has progressed considerably. During this same period, new codes and regulations governing the design of nuclear power plants have been developed and have undergone significant changes. This evolution, while not changing the basic design concepts, has yielded more detailed information concerning the seismic behavior of structures, systems and equipment.

Yankee Atomic Electric Company (YAEC) has requested Cygna Energy Services to perform a seismic evaluation of the plant's critical structures and piping systems in accordance with the NRC Systematic Evaluation Program (SEP).

This document establishes the seismic criteria and the seismic evaluation approaches to be used in the investigation.

2.0 SCOPE

The purpose of this document is to establish the methodology and criteria for the seismic evaluation of piping systems and structures for the Yankee Nuclear Power Station.

Within the scope of this program, Cygna Energy Services will:

(a) Perform static analyses for thermal, dead weight, anchor movement and pressure loads, and dynamic analyses for seismic inertia loads. These analyses will be based on the as-built geometry of the piping systems and structures.

(b) Evaluate the critical piping systems and structures to withstand the loading conditions specified herein.

(c) Design retrofits for critical piping systems and structures to meet the loading conditions specified herein.

The piping systems and the structures included in the scope of this effort are summarized in Appendices A and B, respectively.

3.0 CODES AND STANDARDS

The following codes and standards apply to the appropriate sections of this document (except where noted otherwise).

(a) American National Standard Code for Pressure Piping, ANSI B31.1, 1977.

(b) Nuclear Regulatory Guides: 1.60 Rev. 1, 1.61 Rev. 0, 1.92 Rev. 1 and 1.122 Rev. 1.

(c) American Institute of Steel Construction (AISC), "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," 8th Edition.

(d) American Concrete Institute (ACI), "Building Code Requirements for Reinforced Concrete" (ACI 318-77), including 1977 commentary.

(e) American Iron and Steel Institute (AISI), "Specification for the Design of Cold Formed Steel Structural Members," 1968 Edition with 1970 commentary and 1971 supplement.

(f) American Welding Society (AWS), "Structural Welding Code," D1.1-75.

(g) American Society of Mechanical Engineers (ASME), "Boiler and Pressure Vessel Code," 1977 Edition.

(h) U. S. Nuclear Regulatory Commission (NRC), "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," NUREG/CR-0098, May 1978.

(i) International Conference of Building Officials, "Uniform Building Code," 1979 Edition.

(j) U.S. Nuclear Regulatory Commission (NRC), "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," NUREG-75/087, Section 3.7, Washington, DC, Office of Nuclear Reactor Regulation, September, 1975.

(k) American Concrete Institute (ACI), "Code Requirements for Nuclear Safety Related Concrete Structures" (ACI 349-76), including supplements.

(l) American Concrete Institute (ACI), "Code for Concrete Reactor Vessels and Containments" (ACI 359-77), including 1977 commentary.

(m) Newmark, N. M., et al., "Seismic Review of Dresden Nuclear Power Station--Unit 2 for the Systematic Evaluation Program," U.S. Nuclear Regulatory Commission, NUREG/CR-0891. 1979.

(n) ANSI B16.10, "Face-to-Face and End-to-End Dimensions of Ferrous Values," 1973.

(o) American Concrete Institute (ACI), "Building Code Requirements for Concrete Masonry Structures," (ACI Committee 531-79).

(p) U. S. Nuclear Regulatory Commission (NRC), LS05-82-02-068 (Docket No. 50-206), February 17, 1982.

(q) U. S. Nuclear Regulatory Commission (NRC), Regulatory Guide 1.92, Rev. 1, February, 1976.

(r) ITT Grinnell Catalog PH81.

(s) Hilti, Architects and Engineers Anchor and Fastener Design Manual, File No. H2189-S1, Report No. 8738R.

(t) U. S. Nuclear Regulatory Commission (NRC), "Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components," Regulatory Guide 1.122, Rev. 1, February, 1978.

(u) Pacific Scientific, Kin-Tech Division, Snubber Information Catalog, 1982.

4.0 REFERENCE DOCUMENTS

The following reference documents shall be used in carrying out the piping stress and structural analysis efforts:

(a) Yankee Atomic Electric Company, "Final Hazard Summary Report, Yankee Nuclear Power Station, Rowe, Massachusetts."

(b) Specification for Piping. YS-497 (S & W, JO-9699) July 15, 1959. Yankee Atomic Electric Company, Yankee Nuclear Power Station, Rowe, Massachusetts.

(c) Hot Service Thermal Insulation for Yankee Atomic Electric Plant. YS-2304 (S & W, JO-9699) June 1, 1959. Yankee Nuclear Power Station, Rowe, Massachusetts.

(d) Piping Flow Diagrams, Yankee Nuclear Power Station, Rowe, Massachusetts (Drawing Sets E through S).

(e) Piping Drawings (Drawing Nos. 9699-FP-1 through 77).

(f) Stone and Webster Contract Drawings for Yankee Nuclear Power Station, 1958, 1959.

(g) Cygna Energy Services, "Steel Vapor Container, Yankee Nuclear Power Station, Structural Analysis Report, YCS and NRC Spectra Loadings," EY-YR-80023-5, 1981, Rev. 0.

(h) Cygna Energy Services, "Reactor Support Structure, Yankee Nuclear Power Station, Structural Analysis Report, YCS and NRC Spectra Loadings," 1981, Rev. 0.

(i) Weston Geophysical Corporation, "Geology and Seismology, Yankee Rowe Nuclear Power Plant," January 29, 1979.

(j) Wiegel, R. L., Earthquake Engineering, Prentice-Hall, Inc. (Englewood Cliffs, N.J.), 1970.

(k) Housner, G. W., "Dynamic Pressures on Accelerated Fluid Containers," Bulletin, Seismic Society of America, 47(1), January, 1967.

(l) U.S. Atomic Energy Commission, Nuclear Reactors and Earthquakes, TID-7024, Washington, DC, Office of Technical Services, 1963.

(m) Bleich, F., Buckling Strength of Metal Structures, McGraw-Hill Book Company, New York, 1952.

(n) Bresler, B., T. Y. Lin, and J. B. Scalzi, Design of Steel Structures, John Wiley & Sons, Inc., New York, 1968.

(o) Popov, E.P., Introduction to Mechanics of Solids, Prentice - Hall Inc., Englewood Cliffs, New Jersey, 1968.

(p) Yankee Rowe Project Memoranda #002 SF, dated 12/22/81 for Job No. 8160.

(q) Newmark, N.H. and E. Rosenblueth, Fundamentals of Earthquake Engineering, Prentice-Hall, Inc., 1971.

(r) Powell, G.H., "Inelastic Dynamic Analysis of Tall Buildings," Earthquake Resistant Design of Engineering Structures, ASCE, Conference, July 19-30, 1972.

(s) Clough, R. W. and K. L. Benuska, "FHA Study of Seismic Design Criteria for Hi-rise Buildings," Report HUD IS-3, Federal Housing Administration, Washington, DC, August 1966.

(t) Kobori, Takuji, et al., "Elastic-plastic Earthquake Response of Frames with Shearwall," Proceedings, World Conference of Earthquake Engineering, Meerut, India, Vol VIII, 1977, pp 3037-3042.

(u) Takeda, T., "Study of Load-Deflection Characteristics of Reinforced Concrete Beams Subjected to Alternating Loads," Transactions, Architectural Institute of Japan, Vol. 76, September 1962.

(v) Earthquake Engineering Systems, Inc., "SIMQKE - A Computer Program for Acceleration Time-History Generation," Version 1, July 1981.

(w) "Structural Building Response Review, Vols. 1 and 2," reports prepared by Lawrence Livermore Laboratory for the US Nuclear Regulatory Commission, NUREG/CR-1423, Vol. 1 and SL-3759, Vol. 2, May 1980.

(x) "Nonlinear Structural Dynamic Analysis Procedures for Category I Structures," report prepared by John A. Blume & Associates for the US Nuclear Regulatory Commission, NUREG/CR-0948, July 1979.

(y) Western Geophysical Corporation, "Site Dependent Response Spectra, Yankee Rowe," February 1970.

(z) Timoshenko, S. P., and J. M. Gere, "Theory of Elastic Stability," 2nd Ed., McGraw-Hill Book Company, 1961.

(aa) Brush, D. O., and B. O. Almroth, "Buckling of Bars, Plates and Shells," McGraw-Hill Book Company, 1975.

(bb) Schmidt, H., "Ergebnisse von Beulversuchen mit doppelt gekrummten Schalenmodellen aus Aluminum, " Proceedings of the Symposium on Shell Research, North Holland Pub. Co., Amsterdam (1961).

(cc) Cox, H. L., "The Buckling of Plates and Shells," Pergamon Press, N. Y. (1963).

(dd) Cygna Energy Services, "Primary Auxiliary Building, Yankee Nuclear Power Station, Structural Analysis Report, YCS and NRC Spectra Loadings," EY-YR-80023-7, December 1981, Rev. 0.

(ee) Cygna Energy Services, "Turbine Building Turbine Pedestal, Yankee Nuclear Power Station, Structural Analysis Report, YCS and NRC Spectra Loadings," EY-YR-80023-9, December 1981, Rev. 0.

(ff) Cygna Energy Services, "Diesel Generator Building, Yankee Nuclear Power Station, Structural Analysis Report, YCS and NRC Spectra Loadings," EY-YR-80023-8, December 1981, Rev. 0.

(gg) Cygna Energy Services, "Spent Fuel Pool and Chute, Yankee Nuclear Power Station, Structural Analysis Report, YCS and NRC Spectra Loadings," EY-YR-80023-10, June 1982, Rev. 0.

5.0 STRUCTURAL LINEAR PERFORMANCE CRITERIA

This section describes the criteria to be used in the analysis, evaluation and design of modifications of the structures listed in Appendix B.

5.1 Material Properties

The following material specifications govern unless superseded by field tests.

5.1.1 Concrete

See Table D-1.* These values are obtained from Reference 4(f).

5.1.2 Structural Steel

See Table D-1.

5.1.3 Masonry

See Tables 7.1 and 7.2 in Section 7.

5.1.4 Soils

Bearing capacity for the soil underneath all footings shall be 8 ksf if wind or earthquake loads are not considered, and 10.6 ksf if they are. Compacted backfills shall have a bearing capacity of 4 ksf. For reference, see Stone & Webster Drawing No. 9699-FC-59B.

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* Letters refer to Appendix

5.2 Loads Description

5.2.1 Dead Loads

Dead loads and their related internal moments and forces, including fixed equipment loads, will be included in the analysis. Equipment weights up to 500 lbs. will be considered as distributed loads; equipment weights more than 500 lbs. will be applied as concentrated loads.

5.2.2 Live Loads

Live loads and their related internal moments and forces, including any moveable equipment loads, will be included in the analysis.

5.2.3 Earth Pressure and Groundwater Table

Loads due to earth pressure will be included in the analysis. Lateral soil load factors to be used in the analysis shall reflect the physical properties of the soil as described in Table 4 of Reference 4(y). Hydrostatic loads due to groundwater table will be included in the analysis.

5.2.4 Fluid Loads

Fluid loads will be treated as hydrostatic except under seismic conditions. For this case, fluid loads will be computed using the Housner method. See References 4.1(j), (k) and (l).

5.2.5 Seismic Loads

All structures shall be evaluated for the Safe Shutdown Earthquake (SSE) as defined by the NRC Spectra.

5.2.6 Thermal Loads

Thermal loads will be considered for extreme environmental and operating conditions. The stress-free temperature is assumed to be 50° F. The coolest outside temperature considered is -20°F and the warmest outside temperature is +120°F, which will both produce a differential temperature of 70° F for the extreme environmental condition. Under the accident condition, the LOCA temperature inside the steel vapor container is assumed to be 249° F. In this case, the differential temperature is 199° F.

5.3 Analysis Methodology

5.3.1 Analysis Procedure

Figure C-1 shows the general structural analysis steps. These steps are described in the following paragraphs:

The SSE ground response spectra will be used to generate artificial time histories using Cygna's NRCQUAKE program. These time histories will then be verified by checking their generated response spectra (plotted using INSPEC program) against Reference 3(t).

In a parallel effort the structural models are developed.

The basic linear elastic analysis technique will be the response spectrum modal superposition method of dynamic analysis.

Based on information obtained from Reference 4(y) the plant structures are founded on very dense glacial till. The in situ seismic velocity values range from 6,700 to 7,000 ft./sec. and 1,700 to 2,200 ft./sec. for compressional and shear waves, respectively. The average values of the

Young's Modulus of Elasticity and Poisson ratio are 3×10^5 psi and 0.46, respectively.

By using the formulas recommended by Newmark & Rosenblueth [Ref. 4.1(q)], the stiffness of the equivalent rocking and horizontal soil springs of the Reactor Support Structure are calculated as 6.93×10^9 k-ft/rad and 5.27×10^6 k/ft, respectively. Using the same reference, the virtual rocking and horizontal soil masses are calculated as 41355 K-sec^2-ft and 54.9 K-sec^2/ft, respectively. A modal analysis performed for the model with soil springs and soil masses yielded a natural frequency of 1.15 Hz compared to 1.27 Hz for the model without soil springs and soil masses. For all events considered, the inertial forces applied to the model with soil springs and soil masses are lower than those considering the model without soil springs and soil masses. Therefore, the effect of the soil-structure interaction for the Reactor Support Structure can be neglected. The same conclusion can be made for all the other plant structures.

The inter-connected buildings will be studied case-by-case. If buildings are coupled, they will be considered appropriately connected and analyzed as one unit. With a few possible exceptions (where the floor slabs have substantial openings) the floor diaphragms will be treated as rigid in plane. Three-dimensional beam elements will be used to describe columns and other beam-type components. The models will describe the stiffness and mass relationship in three-dimensional space. Torsional effects due to asymmetric characteristics are automatically considered in this procedure. In buildings where the potential for further accidental torsion is considered likely, accidental torsion considerations as per NUREG/CR-0098 will be included. The superstructure in the reactor concrete pedestal is very stiff and consequently it responds dynamically in a rigid body type motion. The majority of the structural deformations will take place in the base columns and especially at their connections with the superstructure and foundation. Based on this behavior, a simplified model will be developed representing the superstructure as a vertical cantilever with multiple lumped masses. This cantilever will be connected to the base columns with a rigid beam system. The superstructure will subsequently be analyzed for the effects generated from the above analysis in addition to its own loads.

The steel vapor container will be modeled using shell elements to represent the sphere, and beam elements to represent the columns. In developing the spherical model, care will be taken to generate a finer mesh at the locations of high stress. The thin shell elements will be modeled to account for the actual thickness of the plate used in the structure which ranges from 7/8 to 3 inches.

Additional masses will be applied to selected node points in the model to account for concentrated masses such as hatches and platforms.

The structural dynamic properties obtained from the dynamic analyses will be used to perform a modal superposition analysis using the program MOST.* Amplified time histories will be generated at designated locations in the structures. These amplified time histories will be used to generate the Amplified Response Spectra (ARS) using the program INSPEC. The ARS will be broadened per Regulatory Guide 1.122. These ARS will be used as input for the piping and equipment analyses.

The resulting stresses and deformations will be obtained from the modal responses using the SRSS method, except for closely spaced modes where Regulatory Guide 1.61 method will be used.

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* See Reference K for Computer Programs

The stresses and deformations will be evaluated for their compliance with Section 5.4. The piping between buildings will be reviewed for these displacements.

5.4 Acceptance Criteria

5.4.1 Load Combination

The analyses will be performed assuming that the seismic event is initiated with the plant at normal full power condition. The following load combinations will be considered in evaluating the structures and performing structural design of modifications.

$$U = D + R_o + P_o + (E \text{ or } W \text{ or } M) \qquad (5.1)$$

or

$$U = D + L + T_o + R_o + P_o \qquad (5.2)$$

where:

U = Total load to be resisted.

D = Dead loads or their related internal moments and forces, including any permanent equipment loads, hydrostatic loads, and lateral soil pressures. It also includes operating static and dynamic heads and fluid flow effects.

L = Live loads or their related internal moments and forces, including snow loads, any moveable equipment loads and crane loads. For equipment supports, it also includes loads due to vibration and any support movement effects. Note that crane loads and wind loads are not simultaneous.

T_o = Thermal effects and loads during startup, normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

R_o = Pipe reactions during the startup, normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

P_o = Pressure equivalent static load within or across a compartment generated by normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

E = Loads generated by the SSE. Three earthquake directions will be considered as per NUREG/CR-0098 except for special condition as discussed in NUREG/CR-0891.

W = Wind load, or its related internal movements and forces in accordance with the provisions of Reference 3(i). (Use greater value of W, E or M only.)

M = Loads associated with missiles other than those tornado-generated.

5.4.2 Allowable Stresses

This section is specifically developed for linear elastic dynamic analysis. The allowable stresses for reinforced concrete portions of the structures will be per Reference 3(d). In lieu of the code load factors, the factors shown in Section 5.4.1 will be used.

The member stresses for steel structures will be evaluated against Part 1 of Reference 3(c). Stresses up to 0.95 of critical buckling will be allowed only for straight members in the steel vapor container. This buckling stress is calculated as follows:

$$F_{cr} = \frac{\pi^2 E}{(K\ell/r)^2} \quad \text{for } (K\ell/r) > C_c \; [\text{or } F_{cr} < 0.5\, F_y]$$

$$F_{cr} = \left[1 - \frac{(K\ell/r)^2}{2C_c^2}\right] F_y \quad \text{for } K\ell/r < C_c$$

where: $C_c = \left(\frac{2\pi^2 E}{F_y} \right)^{1/2}$

The allowable buckling stress of the vapor container steel shell elements is based on studies made on the structural stability of shell structures. (Refer to Appendix E-2 of this criteria.)

Existing brace connections will be evaluated per Part 2, Section 2.8 of Reference 3(c).

5.4.3 Allowable Deformation

Deformations will be limited to existing clearances to prevent impact of adjacent structures.

5.4.4 Damping

The damping in structures is a function of the type of material, type of construction, and the state of stress produced by the excitation. Table D-3 in the Appendix lists recommended damping values.

5.4.5 Alternate Criteria

In cases where stresses exceed the allowables given in Section 5.4.2, the Structural Nonlinear Performance Criteria given in Section 6 may be used.

5.5 Modifications

Design of modifications shall be based on conventional methods of structural analysis of linear elastic materials.

The load combinations and allowable stresses in Section 5.4.1 and 5.4.2 shall apply. Connections will also be designed in accordance with Part 1 of the AISC Specifications, Reference 3(c).

Selection of concrete anchors shall be in accordance with Table D-2 of Appendix D.

For material properties to be used in the design of modifications, refer to Table D-1 of this criteria.

6.0 STRUCTURAL NONLINEAR PERFORMANCE CRITERIA

The load categories to be considered in evaluating structural performance include severe and extreme environmental conditions such as "a maximum credible earthquake," which has a small probability of occurrence during the lifetime of the structure. Under these circumstances, it is reasonable to allow the structural system to undergo stresses and deformations beyond the linear elastic limit as long as safety is not compromised.

The nonlinear behavior of a structural system is generally due to material nonlinearity and/or geometric nonlinearity. For a typical nuclear reactor structure, the effect of geometric nonlinearity due to large deformations are usually small and only material nonlinearity is presented.

6.1 Material Behavior

The nonlinear behavior of a multiple degree-of-freedom system is generally complicated. For this reason, nonlinear material characteristics are modeled into simple idealized force-deformation curves.

6.1.1 Lumped Plasticity

For a ductile model of a structural member (or connection) subject to bending moment, a plastic hinge will form when the maximum bending moment reaches its yield value. For beam-column members, the interaction of bending and axial forces should be considered.

For a structure constructed of ductile materials, plastic deformation may spread under increasing loads. As a result, plastic hinges will form in the structural model and stresses will be redistributed. Details about this phenomena are found in References 4(r) and 4(s).

6.1.2 Distributed Plasticity

For strain-hardening materials and/or sections with a shape factor larger than 1.0, the plastic deformation will be distributed over a finite length of the member. The distribution is, in general, not uniform over the member and it plays an important role in stress distribution through the structure. For more accurate analysis, the spread of plastic zones near the yield location must be followed (References 4(s) and 4(t)).

6.1.3 Stiffness Degradation

For concrete structures subjected to monotonically increasing loads, reduction of stiffness will appear due to concrete cracking. Under repeated loads resulting from strong seismic excitations, however, stiffness degradation of reinforced concrete elements can be substantial.

A beam element with degrading stiffness is available in the "DRAIN-TABS" computer program. Yielding may take place only in concentrated plastic hinges at the element ends. Plastic deformation, strain-hardening, and flexural stiffness degradation are modeled by nonlinear rotational springs at each end. The moment-rotation relationship for each spring is an extended version of Takeda's model (Reference 4(u)).

6.2 Earthquake Loading

A key element in the analysis of nuclear power plants is the description of the maximum credible earthquake that might affect the site under construction. This earthquake has only a small probability of occurrence during the lifetime of the plant. The Nuclear Regulatory Commission defines the maximum credible earthquake as Safe Shutdown Earthquake (SSE). Unfortunately, the historical record of earthquakes is often insufficient to define such an SSE. In cases of incomplete earthquake data, accelerograms may be generated artificially by a stochastic process (Reference 4(v)).

Because earthquake ground motions are stochastic in nature, they can not be represented mathematically by a single continuous function with a closed form solution. Two common methods for computing the dynamic response of a structure are: (i) Time-History Analysis (Reference 4(w)), and (ii) Response Spectrum Method (Reference 4(q)). The Time-History Analysis requires the earthquake complete time-history and it computes the complete structural response. The Response Spectrum method, however, requires the earthquake spectrum and yields maximum structural response.

6.3 Analysis Procedures

It is found that nonlinear analysis is often required for structures subjected to high seismic excitations. The approaches for nonlinear analyses are broadly categorized as: (i) Time-History Analysis; (ii) Equivalent Linear Approach, and (iii) Static Nonlinear Analysis.

6.3.1 Time-History Analysis

A number of commercially available computer programs suitable for determining the seismic response of nonlinear structures utilizing step-by-step numerical integration are available (Reference 4(w)). At each time step, changes in material or geometry or connectivity properties of the structure are made. This implies a deterministic approach since a deterministic time-history is involved. By use of several time histories independently considered, one can arrive at an average or conservative upper bound of response at the expense of a considerable increased amount of calculations. For a system with large number of degrees-of-freedom, this analysis is time consuming and costly.

Important general characteristics of inelastic response of structures can often be understood through study of the response of a single degree-of-freedom model. This model is particularly adequate for a structure where the response indicates that most of the mass is excited in one fundamental response mode. An example of this structure is the Reactor Support Structure (Reference 4(h)).

6.3.2 Equivalent Linear Approach

For simple hysteretic structures, the equivalent linear approach is quite viable. In this approach, an equivalent linear system is developed such that its response is matched, by various alternative measures, to the response of the subject nonlinear system. This method has the following attractive features: (i) it is more economical than detailed nonlinear dynamic computations; (ii) such equivalent linear calculations provide important insights into the nature of structural system response; (iii) it is compatible with the response spectrum input; and (iv) it is useful in the development of practical response spectrum method for multiple degree-of-freedom systems (Reference 4(q) and Standard 3(h)).

6.3.3 Nonlinear Static Analysis

An earthquake-resistant structure should be capable of deforming in a ductile manner and must be strong enough to survive the SSE. The structural response under earthquake excitations is a dynamics problem. Their effect, however, may be expressed as an equivalent static load once the dynamic characteristics of the structure have been determined or estimated. A nonlinear pseudo static analysis should be performed to compute the ultimate structural lateral resistance.

6.3.4 Computer Programs

Several computer programs are available for nonlinear dynamic and static analysis. Appendix K summarizes the computer programs which have been reviewed in Reference 4(x).

6.4 Acceptance Criteria

6.4.1 Load Combination

The load combinations in Section 5.4.1 shall apply.

6.4.2 Resistance to Extreme Environmental Conditions

Under extreme environmental conditions, the structure may be allowed to stress and deform beyond the elastic limit as long as safety is not compromised.

The structure shall safely withstand the SSE.

6.4.3 Damping

Table D-3 recommends damping values for various types of structures and piping.

7.0 MASONRY WALL LINEAR PERFORMANCE CRITERIA

This section provides the technical basis for the qualification of reinforced and unreinforced masonry block walls. It is specifically intended for the analysis of walls considered critical for hot shutdown. The criteria uses conservative and simplified assumptions to reflect the qualities of materials, give proper considerations to boundary conditions and to the dynamic behavior of the walls.

7.1 Assumptions

- Stress-Strain model is linear elastic.
- Damping values are 5% and 7% of critical damping associated with YCS and NRC spectra motions, respectively, in accordance with NUREG-0098.
- All walls can be represented by an assembly of thin plate elements.

7.2 Analysis and Design

7.2.1 General

- Linear elastic analysis will be performed for all walls.
- The aspect ratio for all rectangular grid elements shall not exceed 2.0.
- Support conditions for all walls shall be considered pinned on a minimum of two (2) sides unless otherwise noted in field survey information.

* Multi-wyth walls shall be analyzed as multiple single-wyth walls connected by cross ties. No credit shall be taken for collar joint mortar shear capacity.

* Section properties of unreinforced and reinforced block walls are shown in Tables I and II, respectively.

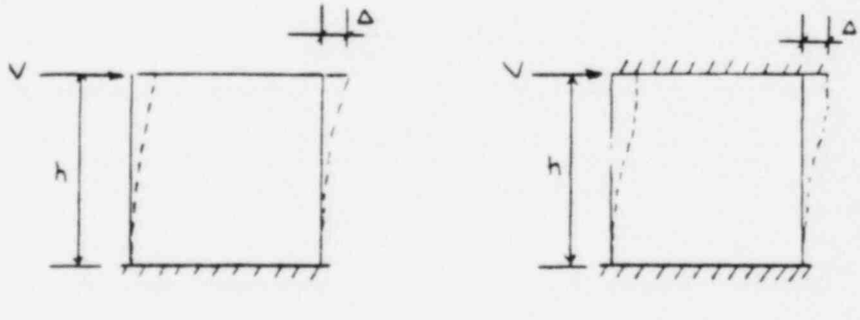
7.2.2 Seismic Loads

7.2.2.1 Inertia Loads

All walls shall be analyzed using uncracked section properties in the two major axes. A response spectrum analysis shall be performed using curves for 5% (YCS) and 7% (NRC) critical damping. The response spectra at the top will generally be used to determine the wall inertia forces.

7.2.2.2 Interstory Displacement

The total relative displacement due to flexure plus shear can be calculated as follows:



CANTILEVER FIXED-END

$$\Delta = \Delta_f + \Delta_s$$

Due to Flexure: Due to shear:

$$\Delta_f = \frac{vh^3}{3E_m I} \qquad \Delta_s = \frac{1.2\ vh}{AG} \quad \text{(Cantilever)}$$

$$\Delta_f = \frac{vh^3}{12E_m I} \qquad \Delta_s = \frac{1.2\ vh}{AG} \quad \text{(Fixed-end)}$$

where:

I = moment of inertia of the wall panel

G = shear modulus of rigidity

A = shear area

Shear force, v, shall be calculated considering in-plane inertia force of the wall itself and tributary load from the wall(s) perpendicular to the wall being analyzed.

7.2.2.3 Equipment Attachments and Pipe Penetration

- Attached equipment shall be modeled as lumped masses.
- In case of pipe penetrations, effective lumped masses shall be obtained at the point of penetration.

7.2.3 Wind Loads

A static uniform wind pressure based on the latest edition of the Uniform Building Code shall be applied to each exterior wall. Exterior and interior walls shall be analyzed for depressurization loads as obtained from YAEC.

7.3 Acceptance Criteria

7.3.1 Flexure

The maximum moment shall be calculated for the following load cases:

1. YCS Spectrum
2. NRC Spectrum
3. Wind load

Allowable stressses to be used in the analysis are in Table 7-3. Concrete tensile stresses will be resisted by steel only. Working stress design method will be used to calculate moment capacity of the reinforced section.

7.3.2 In-Plane Shear

The maximum shear force shall be calculated per Section 7.2.2.2. The allowable shear stress shall be as shown in Table 7-3.

7.3.3 Interstory Displacements

The total relative displacement calculated in Section 7.2.2.2 shall not exceed 0.1% of the wall height.

7.4 Modifications

Section 5.5 of this criteria shall apply where applicable.

7.5 References

1. Reference 3(i).

2. Reference 3(o).

3. Reference 3(p).

TABLE 7.1

UNREINFORCED MASONRY WALL SECTION PROPERTIES

(UNCRACKED)

THICKNESS (INCHES)			MOMENTS OF INERTIA (IN^4/IN)	MASS DENSITY	
NOMINAL	ACTUAL	EFFECTIVE	$I_{gx} = I_{gy}$	($lb\text{-}sec^2/in^4$)	REMARKS
8	7.625	6.76	25.73	1.4622×10^{-4}	
12	11.625	9.76	77.45	1.4731×10^{-4}	
16	15.25	8.52	51.46	2.3203×10^{-4}	Two 8" block walls
20	19.25	8.52	51.46	2.9004×10^{-4}	Two 8" block walls
24*	23.25	12.30	154.90	2.3378×10^{-4}	Two 12" block walls
24*	23.25	9.75	77.19	3.0414×10^{-4}	Three 8" block walls

* Section properties shall be based on supplied field data.

Table 7.2

REINFORCED WALL PROPERTIES

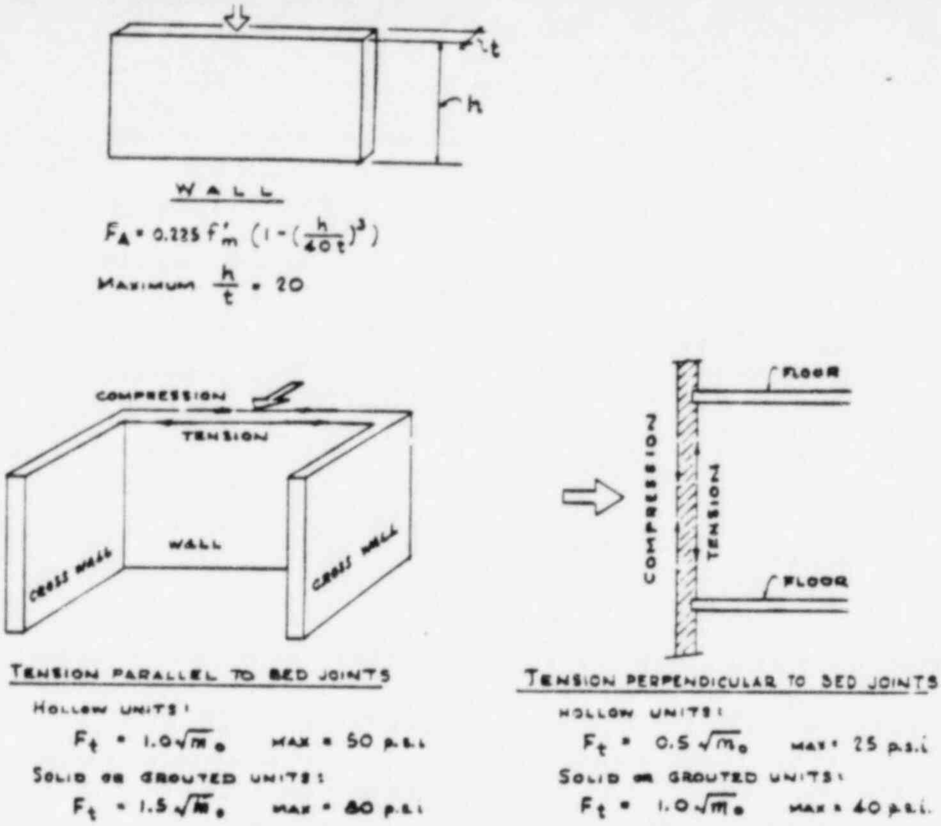
Nominal Thickness (inch)	Moment of Inertia in^4/in I_{tx}	I_{ty}	Mass-density $lb\text{-}sec^2/in^4$	Remarks
8	0.0	3.62	1.83443×10^{-4}	Reinforcement #6 @ 2'-8"c/c vertical direction only

TABLE 7-3 ALLOWABLE STRESSES

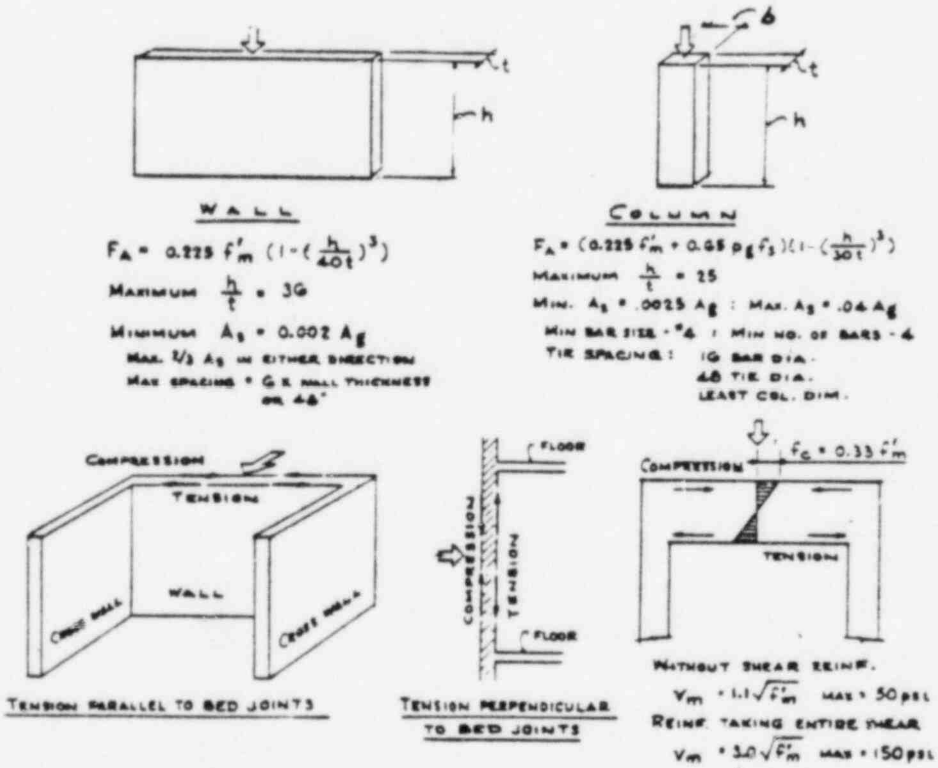
Description		NRC Criteria	ACI 531		CYGNA		
		f'_m = 875	Related to f'_m	Maximum		C-90 Exterior walls $f'm$ = 1000 m_o = 750	C-129 Interior walls $f'm$ = 600 m_o = 750
Compressive			See Sections 10.1.3 and 10.1.4				
Axial	F_a			1000		N/A	
Flexural	F_m	350	$0.33f'_m$	1200	$0.33\ f'_m$	330	198
Bearing							
On full area	F_a		$0.25f'_m$	900	$0.25\ f'_m$	250	150
On one-third area or less	F_a		$0.375f'_m$	1200	$0.375\ f'_m$	375	225
Shear		10					
No shear reinforcement							
Flexural members	v_m		$1.1\sqrt{f'_m}$	50	NRC	10	10
Shearwalls							
$M/VD_v > 1$	v_m		$0.9\sqrt{f'_m}$	34		N/A	N/A
$M/Vd_v < 1$	v_m		$2.0\sqrt{f'_m}$	40 $(1.85 - M/Vd_v)$		N/A	N/A
Reinforcement taking entire shear							
Flexural members	v		$3.0\sqrt{f'_m}$	150			
Shearwalls							
$M/Vd_v > 1$	v		$1.5\sqrt{f'_m}$	75		N/A	N/A
$M/Vd_v < 1$	v		$2.0\sqrt{f'_m}$	45 $(2.67 - M/Vd_v)$			
Tension							
No tension reinforcement							
Tension normal to bed joints							
Hollow units	F_t	0	$0.5\sqrt{m_o}$	25	$0.5\sqrt{m_o}$	14	14
Solid and/or grouted units	F_t	0	$1.0\sqrt{m_o}$	40			
Tension parallel to bed joints in running bond							
Hollow units	F_t	18	$1.0\sqrt{m_o}$	50	$1.0\sqrt{m_o}$	27	27
Solid and/or grouted units	F_t	36	$1.5\sqrt{m_o}$	80			
Modulus of elasticity	E_m		$1000\ f'_m$	2,500,000		810,000	810,000
Modulus of rigidity	E_v		$400\ f'_m$	1,000,000	$G = \frac{E}{2(1+v)}$	375,000	375,000
Steel Stress	f_s	48,000(Gr 60)	20,000(Gr 40)			20,000(Gr 40)	

Notes:

1. N/A = Not applicable
2. Actual strengths of C-90 & C-129 block walls will have to be justified by field inspection.
3. See Table 7.4 for NRC approved increases in allowable stresses.
4. See Figure 7.1 for illustrations of allowable stresses.



Nonreinforced concrete masonry—Allowable stresses



Reinforced concrete masonry—Allowable stresses. f_s = 20,000 psi (Grade 40 steel); f_s = 24,000 psi (Grade 60 steel); f_s = 30,000 psi (Grade 50 joint reinforcement)

FIGURE 7.3 Illustration of Allowable Stresses

TABLE 7-4

STRESS FACTOR FOR LOAD COMBINATION

When tornado, wind or SSE loads combined with other prescribed loads (D, L, Ta and Pa, etc.) are considered, the allowable working stresses may be increased by the factors shown:

TYPE OF STRESS	FACTOR
Axial or Flexural Compression(1)	2.5
Bearing	2.5
Reinforcement stress except shear	2.0 but not to exceed 0.9 fy
Shear reinforcement and/or bolts	1.5
Masonry tension parallel to bed joint	1.5
Shear carried by masonry	1.0
Masonry tension perpendicular to bed joint	0

8.0 PIPING ANALYSIS CRITERIA

This section describes the criteria to be used in the stress analysis of the piping systems listed in Appendix A.

8.1 Load Description

The following load cases shall be considered for the piping stress analysis. In addition, local stress concentration due to integral support shall be evaluated.

8.1.1 Thermal Load

Loads due to steady state temperature effect, including thermal anchor movements.

8.1.2 Weight Load

Loads due to pipe, content and insulation.

8.1.3 Pressure Load

Loads due to steady state internal pressure.

8.1.4 Seismic (SSE) Load

Loads due to earthquake excitations, including both seismic inertia effect and seismic anchor movements.

8.1.5 Occasional Load

Loads due to relief valve blow-off.

8.2 Analysis Methodology

8.2.1 Geometry and Computer Modeling

For the purpose of computer analysis, the piping systems will be idealized by three-dimensional linear elastic models. All supports and anchors are assumed rigid. The direct stiffness method will be used.

(a) Each problem shall be considered from anchor to anchor. If an anchor to anchor problem exceeds program limitations, multiple computer run results may be bracketed to assess boundary or loading conditions.

(b) The geometry and restraint conditions shall be modeled in accordance with as-built isometrics.

(c) The pipe material properties and analysis conditions shall be considered as per YAEC's approved information such as Yankee Piping Specification (YS-497), YAEC flow diagrams, Yankee Insulation Specifications (YS-2304) and Grinnel catalog data.

(d) Branch lines with a nominal diameter greater than 2 inches: The system may be decoupled if the ratio of moment of inertia (pipe/branch) is 25:1 or greater.

Also, branch lines with a nominal diameter 2 inches and less: The system may be decoupled if the ratio of moment of inertia (pipe/branch) is 10:1 or greater.

In both the cases above, an anchor shall be considered at the point of decoupling on the branch lines for the purpose of analysis only. Also, the main-line deflections and rotations shall be input as anchor movements for the branch line analysis.

(e) Equipment nozzles and penetrations shall be considered as anchor points in the analysis. All equipment is assumed to be properly supported. Loading shall be summarized and compared to allowables when available. When allowable loads are not available, the calculated loads shall be submitted to YAEC for their review. Thermal anchor movements at nozzles and penetrations shall be indicated on the as-built isometrics. Or, if necessary, they shall be calculated by conventional methods based on system design temperature. Refer to Appendix I for details of nozzle allowables information.

(f) Valves shall be modeled as follows:

- Thickness of the valve body shall be considered as twice the connecting pipe wall thickness.

- Manually operated valves and check valves shall be modeled with the mass of the valve concentrated at the centerline of the pipe at the valve node points.

- Motor and air operated valves shall be modeled as eccentric mass points. The total weight of the valve shall be concentrated at a point one-third (1/3) the distance from the valve assembly to the centerline of the operator (one-third of the "stem length" measurements as noted on the valve data form).

- If not available, body length of the valve shall be as per ANSI B16.10, 1977.

- Seismic accelerations of the valves shall be summarized. The allowable valve accelerations shall be:
  Resultant horizontal acceleration $\leq$ 4.25 g
  Vertical accelerations $\leq$ 3.0 g (refer to Appendix G for details)

(g) Flanges shall be considered as additional lumped weights. Flange thickness shall be assumed to be the same as that of pipe for purposes of modeling stiffness.

(h) Stress intensification factors for tees, reducers, flanges, elbows and couplings (half and full) shall be considered as per Reference 3(a).

(i) Penetrations shall be analyzed as follows:

- Grouted penetrations: A bilateral restraint condition shall be considered to exist on either side of the penetration for all load cases. Axial restraint of the pipe shall not be considered unless a welded collar is indicated on the pipe and embedded in the penetration.

- Ungrouted penetrations: At ungrouted penetrations, deflection of the pipe $\leq$ 1/4" shall be considered acceptable. Where deflections exceed 1/4", further review of actual penetration clearances shall be initiated. Deflections shall be based on the combined thermal, deadweight and seismic conditions.

(j) The modulii of elasticity at various temperatures for ferrous and non-ferrous materials shall be taken from Appendix C, Tables C-1 and C-2 of the Reference 3(a).

(k) The Poisson's ratio shall be taken as 0.3 for all metals at all temperatures.

(l) The hot modulus of elasticity (E_h) values shall be used for seismic and SAM (Seismic Anchor Movements) analyses and the cold modulus of elasticity (E_c) values shall be used for deadweight, thermal and TAM (Thermal Anchor Movements) analyses.

(m) The flexibility of the equipment shall be considered in the pipe stress analysis where necessary.

(n) The computed stresses of various load cases shall be summarized.

(o) Rod hangers uplift condition shall be taken into account in the analysis as per "Rod Hanger Uplift Study." Refer to Appendix J for details.

8.2.2 Weight Analysis

The following considerations shall be made for dead weight analysis:

Weight analysis shall be performed considering weight of the pipe, content, insulation and concentrated masses (such as pipes supported off pipe, flanges and valves).

8.2.3 Thermal Analysis

Thermal analysis of the piping system shall be performed based either on the maximum design temperatures designated on YAEC flow diagrams or stress isometric drawings or on pipe operating temperatures as supplied by YAEC. Effects of thermal movements from equipment nozzles, anchors, penetrations and connecting piping shall be analyzed. The Thermal Anchor

Movement stress (TAM) shall be added to thermal expansion stress to obtain the total thermal stress.

8.2.4 Seismic Analysis

(a) The basic dynamic analysis technique will be the Response Spectrum, Modal Superposition method using lumped mass models.

For rod hanger type of supports, when the uplift due to seismic load (include Thermal Load if it is upward) is larger than 90% of weight load, the rod hanger support shall be assumed noneffective. Consequently the particular rod hanger support will not be included in the computer modeling.

Seismic Inertia analysis and Seismic Anchor Movement analysis shall be performed for the SSE.

(b) Application of Spectra:

For each earthquake condition, three directions of earthquake will be considered. (Two horizontal components and one vertical component.) The total response due to each of the three (3) components of earthquake shall be calculated first. These individual responses shall then be combined by the SRSS method (Square Root of the Sum of Squares). The procedures to be used in combining the modal responses and responses due to spatial components of earthquake shall be as follows:

1. The modal responses for each component of earthquake shall be combined by taking into consideration the modes with closely spaced frequencies in accordance with Reference 3(q), Subsections 1.2.1, 1.2.2, or 1.2.3.

2. The total systems' responses to the three (3) spatial components of earthquake are then combined by the SRSS method.

The responses of the Yankee Site Specific load case shall be used to evaluate the piping and its support. For piping systems spanning several floors or with pipe supports connected to support structures attached to different floors, the response spectra for the analysis of the piping system shall be the envelope of the floor response spectra of all the floors involved.

(c) Cut-off frequency and minimum number of modes:

A cut-off frequency of 33 cps and with no less than 10 modes shall be considered in the analysis. An equivalent Static-Seismic analysis based on a constant acceleration from the spectra at 33 cps cut-off frequency shall be performed when the contributions of higher modes (>33 cps) are significant. The results of the static analysis shall be combined in an SRSS fashion with the dynamic results.

(d) Damping values:

For the seismic SSE condition, a damping value of two percent (2%) of critical damping shall be used for piping with outside diameter less than or equal to 12" and a damping value of three percent (3%) of critical shall be used for piping with outside diameter larger than 12". Reference 3(g), Page 1.61-2, Table 1, and Reference 3(h), Table 1, Page 44, shall govern.

8.2.5 Seismic Anchor Movement Analysis (SAM)

The SSE Seismic Anchor Movement load condition shall be considered for both stress and support load evaluations.

8.2.6 Pressure Effect

The effect of internal pressure shall be considered in computing longitudinal stresses.

8.2.7 Effect Due to Relief Valve Blow-Off

The effect due to relief valve blow-off shall be calculated as external forces.

8.3 Acceptance Criteria

8.3.1 Stress Equations

Stresses in the piping system must not exceed the allowable stress limits of Reference 3(a). The Acceptance Criteria shall be considered satisfied when the requirements of the following equations are met.

(a) The effects of pressure, weight, and other sustained loads must meet the following requirements:

$$\frac{PD_o}{4t_n} + \frac{0.75i}{Z} M_A < KS_h \qquad \text{(Eq. 8.3.1-A)}$$

where:

P = Internal Design Pressure, psi

D_o = Outside Diameter of Pipe, in.

t_n = Nominal wall thickness of components, in.

i = Stress intensification factor. The product of 0.75i shall never be taken as less than 1.0.

Z = Section modulus of the pipe, in^3.

M_A = Resultant moment loading on cross section of the pipe due to weight and other sustained loads, in pounds.

K = 1.0 for Dead Weight Loading

S_h = Basic material allowable stress at maximum temperature from allowable stress tables, psi.

Stress Intensification Factors "i" shall be as per ANSI B31.1 code 1977 edition.

(b) The effects of pressure, weight, other sustained loads and occasional loads including earthquake must meet the following requirements:

$$\frac{PD_o}{4t_n} + \frac{0.75i}{Z} M_A + \frac{0.75i}{Z} M_B < KS_h \qquad \text{(Eq. 8.3.1-B)}$$

where:

K = 1.8 for Yankee Composite Spectra applied to Hot Shutdown Systems.

= 2.4 for NRC Spectra (refer to Appendix H for details) applied to Hot Shutdown Systems and YCS or NRC Spectra for remaining systems.

M_B = Resultant moment loading on cross section due to occasional loads such as earthquake. Other terms same as 6.3.1-A.

See 8.3.1(a) for terms not listed.

(c) Thermal Expansion Stress (S_E):

$$S_E = \frac{i\ M_c}{Z} < S_A \qquad \text{(Eq. 8.3.1-C)}$$

where:

M_C = Range of resultant moments due to thermal expansion. Also include moment effects of anchor displacement due to earthquake if anchor displacement effects were omitted from Eq. 8.3.1-B. Since the YCS and NRC Spectra represent an SSE, for which stress analysis is not required by the ASME code, 60% of the SAM effects can be used in the M_C. This 60% is intended to limit the SAM results to the same levels as an OBE code analysis would.

S_A = Allowable stress range for expansion stress.

= $f\ (1.25\ S_c + 0.25\ S_h)$

where:

S_c = Allowable stress of the specific material at 70°F (psi)

S_h = Allowable stress of the specific material at maximum temperature in degrees Fahrenheit (psi)

f = Stress range reduction factor

See 8.3.1(a) and (b) for terms not listed.

(d) Sustained Plus Thermal Expansion Stresses:

The effects of pressure, weight, other sustained loads and thermal expansion must meet the requirements of the equation 8.3.1-D

$$\frac{PD_o}{4t_h} + \frac{0.75i}{Z} M_A + \frac{i}{Z} M_C \leq (S_h + S_A) \qquad \text{(Eq. 8.3.1-D)}$$

Terms as previously described.

(e) The requirements of either Equation 8.3.1-C or Equation 8.3.1-D must be met.

(f) Even though only the Response Spectrum Analysis method is considered in this criteria, we do not preclude the possibility of using time history analysis method, if the situation warrants its application. Specific criteria for time history analysis will be provided when the need arises.

8.3.2 Allowable Stresses

Allowable stress values to be used for power piping systems are given in Appendix A of Reference 3(a). Those values shall be used for piping stress analyses.

For material allowable stress values not available in Appendix A of Reference 3(a), reference should be made to Reference 3(g), Section III, Division 1. The appropriate allowable stress values shall be taken from tables contained in Appendix I of the reference

8.4 Small Pipe Stress Analysis

This section applies to piping with a nominal outside diameter of 2" or smaller.

8.4.1 Detailed Stress Analysis

For detailed stress analysis the same procedures and methods as those for large pipe stress analysis shall be followed (Sections 8.1 through 8.3). In addition:

- Connections at elbow, tee, reducer, coupling and nozzle shall be considered as socket welded. However, a threaded connection shall be considered where noted in the drawing.

8.4.2 Simplified Stress Analysis

This is an alternative method to the Detailed Stress Analysis method. Each span of a piping system (spans are generally separated by guides) is evaluated by simplified thermal, seismic and weight stress analyses. Span lengths and support locations are investigated to ensure the requirements of piping flexibility and high natural frequency are met.

(a) Weight stress - weight stress is kept to a predetermined level by using specified support spacings. When concentrated loads such as valves or risers exist, the support span shall be suitably modified, if required.

(b) Thermal stress - thermal stress shall be kept to an acceptable level by providing a minimum offset to absorb thermal movement. Offset is defined as the length of piping in a plane perpendicular to the direction of movement. The offset piping shall be unrestrained in the direction of movement.

(c) Seismic stress - Seismic pipe spans shall be generated by simplified analysis method so that the actual stress will be less than the predetermined maximum stress. These seismic pipe spans and restraint loads are defined as a function of unique spectra curves and pipe sizes. The basic approach is to keep the seismic acceleration of the system low and to keep the natural frequencies in the "Rigid Range." The seismic spans shall generally be separated by guides at each change of direction, at all extended massses and at each tee.

(d) Pressure stress - longitudinal pressure stress shall be computed as per ANSI B31.1 code requirements.

(e) Acceptance requirements - the piping system meets the stress acceptance requirements if each span satisfies the span length, offset and pressure stress requirements mentioned above. Span length shall be adjusted to account for the effect of stress intensification factor applicable to the component under consideration. If any of the above requirements cannot be satisfied, a detailed stress analysis shall be performed for the portion of piping involved.

9.0 PIPE SUPPORT DESIGN REVIEW

9.1 Introduction

The following criteria shall be used to evaluate or redesign pipe supports for the Yankee Rowe Piping Analysis project.

9.2 Codes, Standards and References

The following codes shall be used for the design of pipe supports:

9.2.1 ASME, Reference 3(g), Section III, Subsection NF, 1977 edition.

9.2.2 ANSI, Reference 3(a).

9.2.3 AISC, Reference 3(c).

9.2.4 ITT, Reference 3(r).

9.2.5 PSA-3, Mechanical Shock Arrestors, Pacific Scientific Company, 1977.

9.2.6 Hilti, Reference 3(s).

9.3 Loading Description

All loadings obtained from piping stress analysis shall be used for support design.

9.3.1 Normal Operating Loads

9.3.1.1 Dead Weight, D - includes all gravity loads, such as weight of pipe, its contents and insulation, weight of supporting members and loads due to steady internal pressure.

9.3.1.2 Thermal, TH - Loads generated by restrained thermal expansion.

9.3.1.3 Thermal Anchor Movement, TAM - loads applied at supports due to thermal anchor movement.

9.3.1.4 Friction, FL - Friction loads are to be applied in the opposite direction of thermal movement. Its magnitude shall be the friction coefficient times the summation of the pipe deadload, thermal load and loads due to thermal anchor movement, D+TH+TAM. The friction coefficient for steel-on-steel shall be at least 0.3 and for Teflon 0.07.

9.3.2 Emergency/Faulted Loads

9.3.2.1 Seismic Loads - Only Safe Shutdown Earthquake (SSE) will be considered. Values obtained from SRSS (square root of sum of squares) of the individual responses of each of the three components of earthquake shall be used to design the support. In the case of anchors, the higher component between the two for each piping analysis shall be added to those from the other side of the anchor to arrive at the resultant design load. Seismic loads shall be considered to act in both the positive or negative restraining directions. For example, for anchors,

$$F_{1L} = DW_L + SEIS_L \qquad F_{1R} = DW_R + SEIS_R$$

$$F_{2L} = DW_L - SEIS_L \qquad F_{2R} = DW_R - SEIS_R$$

If $|F_{1L}| > |F_{2L}| \qquad |F_{1R}| < |F_{2R}|$

Then to design the anchor, $F_1 = |F_{1L}| = |F_{2R}|$

9.3.2.1.1 Seismic loads considered are (a) Yankee Composite Spectra (YCS), SSE_{YCS} and (b) NRC, SSE_{NRC}.

9.3.2.1.2 Seismic Anchor Movement, SAM - Loads obtained from seismic anchor movement of both YCS and NRC (SAM_{YCS} and SAM_{NRC}) shall be considered. For supports, the full SSE SAM loads must be used, not 60% as allowed in the pipe stress criteria (Section 8.3.1-c)

9.4 Loading Combinations

Load Case	Service Level	Load
1	NORMAL	(D) or (D+TH+TAM) + FL
2	EMERGENCY/FAULTED	(D) or (D+TH+TAM) + SSE_{YCS}+SAM_{YCS}
3	EMERGENCY/FAULTED	(D) or (D+TH+TAM) + SSE_{NRC}+SAM_{NRC}

9.5 Frequency

The natural frequency of a seismic restraint with its tributary pipe mass must be greater than 33 Hertz in the pipe's restrained direction. The mass used to calculate the natural frequency shall include the weight of the restraint, restrained pipe, pipe insulation, fluid, pipe attachments, and valves. Any rational analysis may be used to calculate the natural frequency. The natural frequency calculations of pipe restraints do not have to include the flexibility of the building structure.

For the purpose of determining the natural frequency of snubbers and their frames, consider the snubber to exhibit stiffness qualities which would make them a rigid link between the pipe and the supporting structure. The supporting structure, from the building's frame to the snubber, shall be designed such that the natural frequency is at least 33 Hertz.

9.6 Allowable Stress

Stress		SERVICE LEVEL			
		NORMAL		EMERGENCY/FAULTED	
		Value	KSI	Value	KSI
Tension		$\emptyset.6\ F_y$	19.8	$\emptyset.9\ F_y$	29.7
Shear		$\emptyset.4\ F_y$	13.2	$\emptyset.6\ F_y$	19.8
Web Crippling		$\emptyset.75\ F_y$	24.8	$\emptyset.9\ F_y$	29.7
Compression		F_a (Table A)		Smaller of 1.5 F_a or 2/3 F_{cr} (Table B)	
Bending		$\emptyset.6\ F_Y$	19.8	$\emptyset.9\ F_Y$	29.7
Bearing		$\emptyset.9\ F_Y$	29.7	N/A	
Bolts 307	Tension	Allowable Tension per AISC		1.5 X (Allowable Tension per AISC)	
	Shear	Allowable Shear per AISC		1.5 X (Allowable Shear per AISC)	
Anchor Bolt		See Note 1		See Note 1	

9.6 Allowable Stress, cont'd.

Welds (Fillet)	Shear	0.3 F_v (Weld Metal)	21.0	0.45 F_v (Weld Metal)	31.5
		.4 F_y (Base Metal)	13.2	.6 F_y (Base Metal)	19.8
(Full or Partial Penetration)	Tension	0.6 F_y (Base Metal)	19.8	0.9 F_y (Base Metal)	29.7
Combined Stress		Per AISC		Per AISC	
Catalog Items		Catalog Values		1.5 X (Catalog Values)	

NOTE 1: The allowable Hilti expansion anchor loads are taken from Page 1 to 9 of Ref. 2.6 with a safety factor of 4.

TABLE A

ALLOWABLE STRESS (KSI)

FOR COMPRESSION MEMBERS OF 33 KSI SPECIFIED YIELD POINT STEEL

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* l/r 121 to 200			
$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{l}{r}$	F_{as} (ksi)	$\frac{l}{r}$	F_{as} (ksi)
1	19.77	41	17.64	81	14.32	121	9.96	161	5.76	121	10.01	161	7.25
2	19.73	42	17.57	82	14.23	122	9.84	162	5.69	122	9.94	162	7.20
3	19.69	43	17.50	83	14.13	123	9.72	163	5.62	123	9.87	163	7.16
4	19.66	44	17.43	84	14.03	124	9.59	164	5.55	124	9.79	164	7.12
5	19.62	45	17.36	85	13.93	125	9.47	165	5.49	125	9.71	165	7.08
6	19.58	46	17.29	86	13.84	126	9.34	166	5.42	126	9.63	166	7.04
7	19.54	47	17.22	87	13.74	127	9.22	167	5.35	127	9.55	167	7.00
8	19.50	48	17.14	88	13.64	128	9.09	168	5.29	128	9.47	168	6.96
9	19.46	49	17.07	89	13.53	129	8.96	169	5.23	129	9.38	169	6.93
10	19.41	50	16.99	90	13.43	130	8.83	170	5.17	130	9.30	170	6.89
11	19.37	51	16.92	91	13.33	131	8.70	171	5.11	131	9.21	171	6.85
12	19.32	52	16.84	92	13.23	132	8.57	172	5.05	132	9.12	172	6.82
13	19.28	53	16.76	93	13.13	133	8.44	173	4.99	133	9.03	173	6.79
14	19.23	54	16.68	94	13.02	134	8.32	174	4.93	134	8.94	174	6.76
15	19.18	55	16.60	95	12.92	135	8.19	175	4.88	135	8.86	175	6.73
16	19.13	56	16.52	96	12.81	136	8.07	176	4.82	136	8.78	176	6.70
17	19.08	57	16.44	97	12.71	137	7.96	177	4.77	137	8.70	177	6.67
18	19.03	58	16.36	98	12.60	138	7.84	178	4.71	138	8.62	178	6.64
19	18.98	59	16.28	99	12.49	139	7.73	179	4.66	139	8.54	179	6.61
20	18.93	60	16.20	100	12.38	140	7.62	180	4.61	140	8.47	180	6.58
21	18.88	61	16.12	101	12.28	141	7.51	181	4.56	141	8.39	181	6.56
22	18.82	62	16.03	102	12.17	142	7.41	182	4.51	142	8.32	182	6.53
23	18.77	63	15.95	103	12.06	143	7.30	183	4.46	143	8.25	183	6.51
24	18.71	64	15.86	104	11.95	144	7.20	184	4.41	144	8.18	184	6.49
25	18.66	65	15.78	105	11.83	145	7.10	185	4.36	145	8.12	185	6.46
26	18.60	66	15.69	106	11.72	146	7.01	186	4.32	146	8.05	186	6.44
27	18.54	67	15.61	107	11.61	147	6.91	187	4.27	147	7.99	187	6.42
28	18.48	68	15.52	108	11.50	148	6.82	188	4.23	148	7.93	188	6.40
29	18.42	69	15.43	109	11.38	149	6.73	189	4.18	149	7.87	189	6.38
30	18.36	70	15.34	110	11.27	150	6.64	190	4.14	150	7.81	190	6.36
31	18.30	71	15.25	111	11.15	151	6.55	191	4.09	151	7.75	191	6.35
32	18.24	72	15.16	112	11.04	152	6.46	192	4.05	152	7.69	192	6.33
33	18.18	73	15.07	113	10.92	153	6.38	193	4.01	153	7.64	193	6.31
34	18.11	74	14.98	114	10.80	154	6.30	194	3.97	154	7.59	194	6.30
35	18.05	75	14.89	115	10.69	155	6.22	195	3.93	155	7.53	195	6.28
36	17.98	76	14.80	116	10.57	156	6.14	196	3.89	156	7.48	196	6.27
37	17.92	77	14.70	117	10.45	157	6.06	197	3.85	157	7.43	197	6.26
38	17.85	78	14.61	118	10.33	158	5.98	198	3.81	158	7.39	198	6.24
39	17.78	79	14.51	119	10.21	159	5.91	199	3.77	159	7.34	199	6.23
40	17.71	80	14.42	120	10.09	160	5.83	200	3.73	160	7.29	200	6.22

* K taken as 1.0 for secondary members.

TABLE B

BASIC COLUMN BUCKLING STRESS, F_{cr}, (KSI)

FOR STEEL OF 33 KSI YIELD STRESS

Effective Slenderness Ratio, $K\ell /r$	Buckling Stress, F_{cr} KSI
5	32.98
10	32.90
15	32.79
20	32.62
25	32.41
30	32.14
35	31.83
40	31.49
45	31.07
50	30.62
55	30.12
60	29.58
65	28.98
70	28.34
75	27.65
80	26.91
85	26.13
90	25.29
95	24.42
100	23.49
105	22.51
110	21.49
115	20.42
120	19.30
125	18.14
130	16.92
135	15.70
140	14.60
145	13.61
150	12.72
155	11.91
160	11.18

APPENDICES

APPENDIX A

PIPING SYSTEMS WITHIN SCOPE

A. The following piping systems are in scope:

1. Main Steam
2. Feed Water
3. Reactor (Main) Coolant
4. Pressure Control & Relief
5. Charging & Volume Control
6. Safety Injection
7. Shut Down Coolant
8. Sample and Drain System
9. Primary Plant Purification
10. Emergency Feedwater System
11. Vapor Containment Heating System
12. Control Rod Drive
13. Emergency Core Cooling System
14. Component Cooling System
15. Service Water System

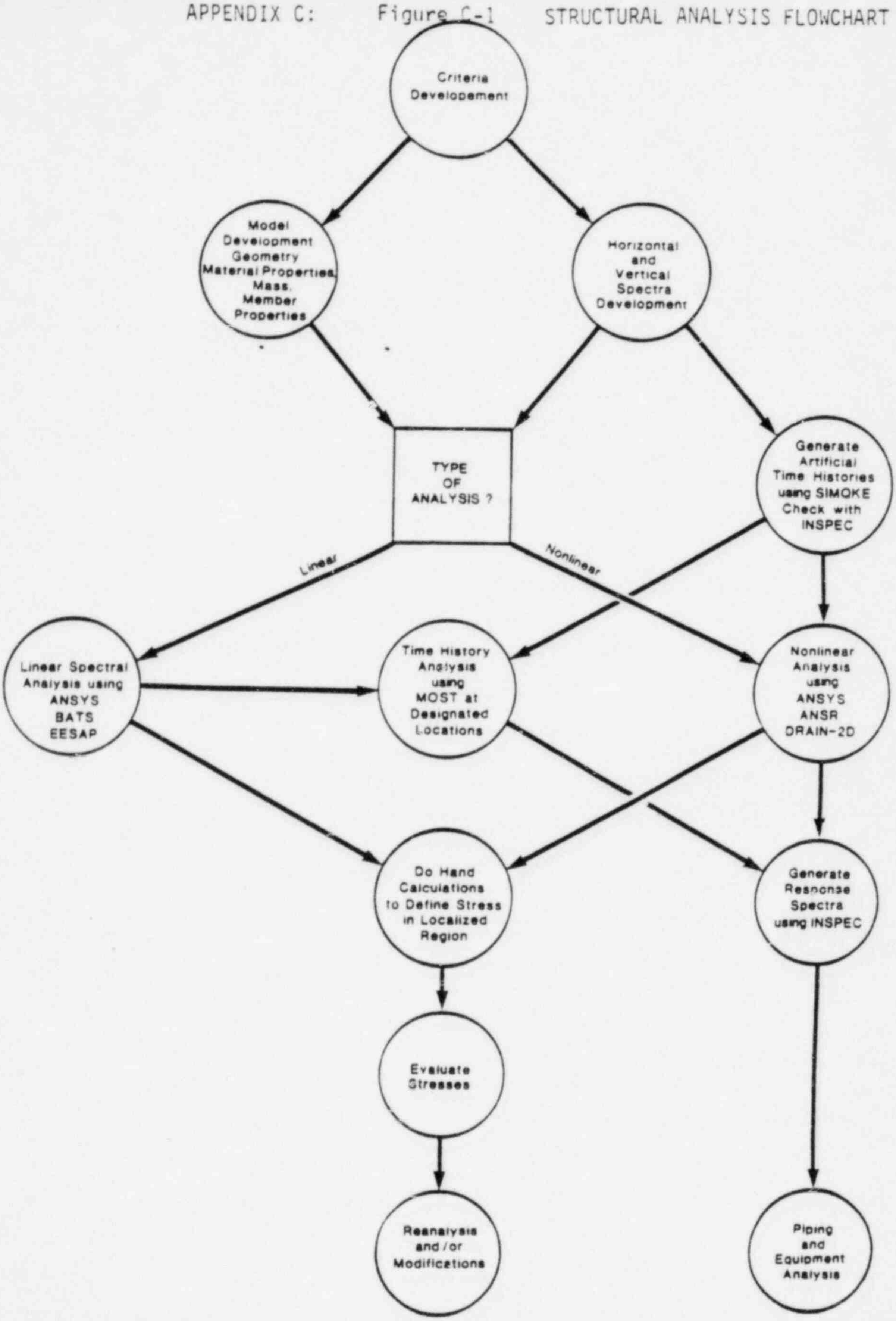
APPENDIX B

STRUCTURES WITHIN SCOPE

B. The following structures are in scope:

1. Concrete Reactor Support Structure
2. Steel Vapor Container Structure
3. Diesel Generator Building, Accumulator Tank Enclosure and Annex
4. Turbine Building and Turbine Pedestal
5. Ion Exchanger Building
6. Primary Auxiliary Building and Radioactive Tunnel
7. Screen Well and Pump House
8. Spent Fuel Pool and Spent Fuel Chute
9. Main Steam/Feed Water Support Structure
10. Fire Tank

APPENDIX C: Figure C-1 STRUCTURAL ANALYSIS FLOWCHART



APPENDIX C

FIGURE C-2

PIPING STRESS ANALYSIS FLOWCHART

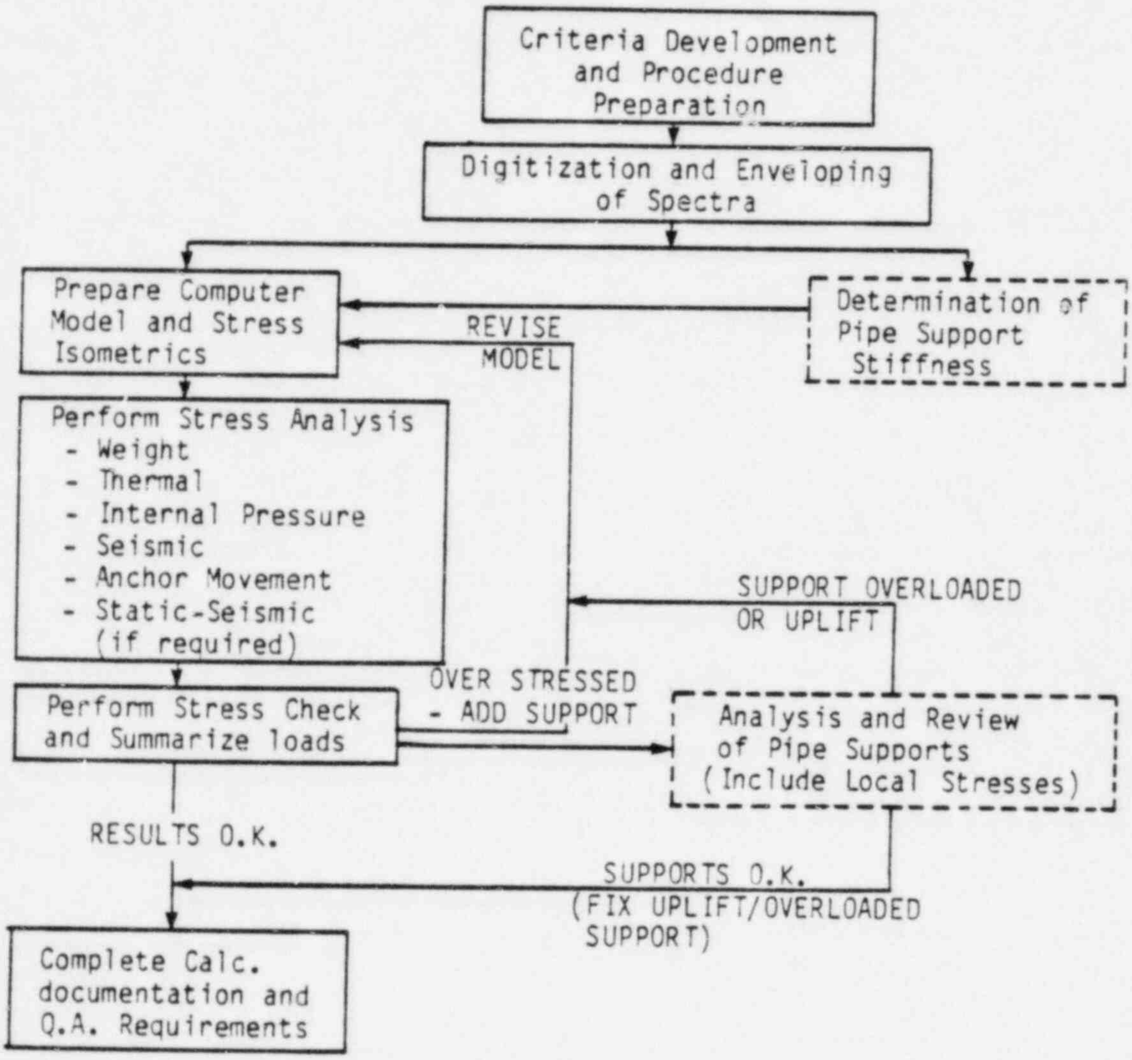
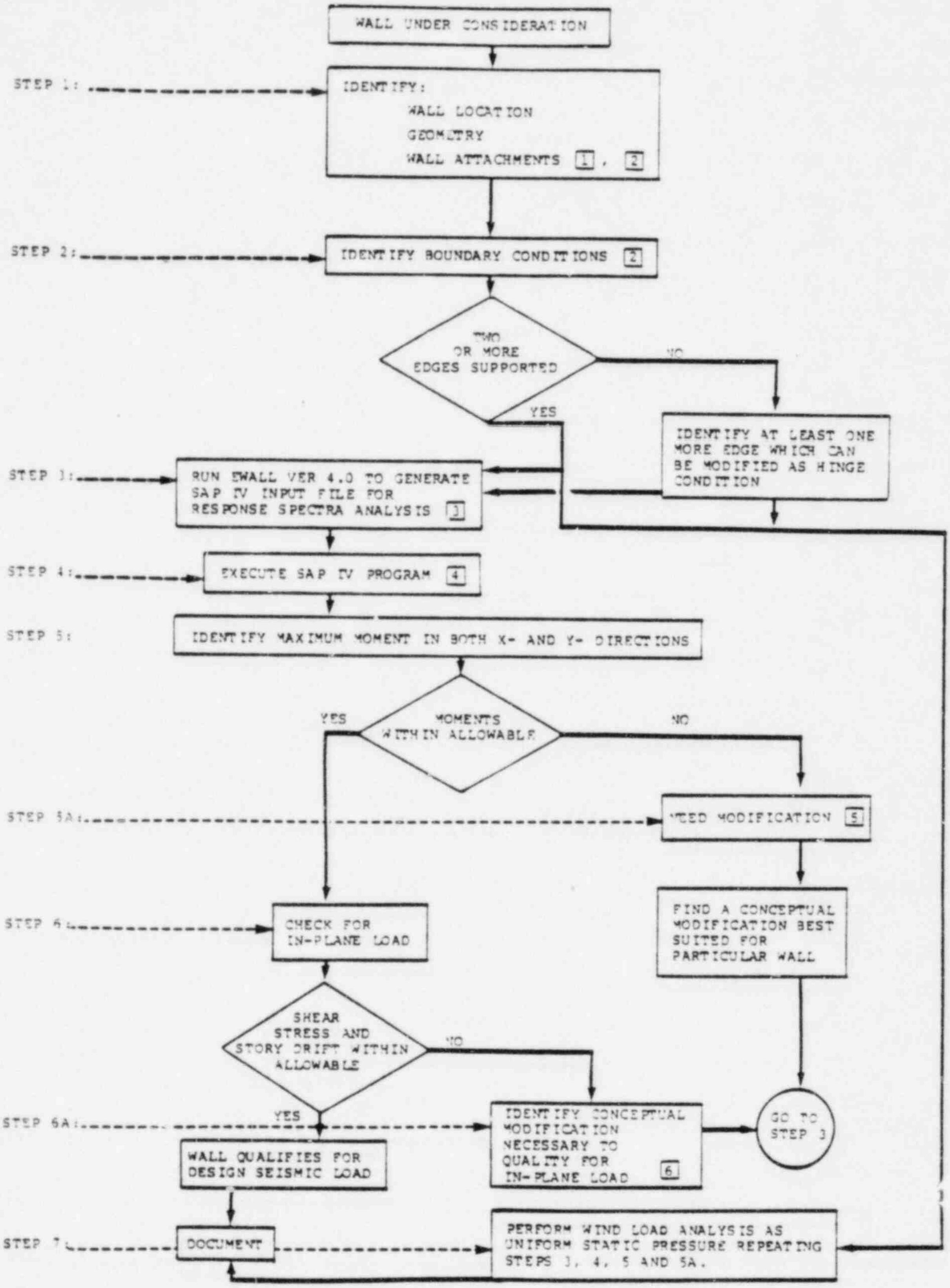


FIG. C-3

FLOWCHART FOR MASONRY BLOCK WALL ANALYSIS



NOTES:

1. Nominal wall attachments are included in mass density of wall. Attachment weight 100 lbs. or more shall be considered as concentrated load at that location.
2. Refer to field survey information binder 81033-1F.
3. Use 12 modes in dynamic analysis.
4. Use "SAPRD" execution command.
5. Controlling stress, either seismic or wind load, will govern modification design.
6. Modification for in-plane load may affect out-of-plane load carrying capacity of wall.

APPENDIX D: TABLE D-1

MATERIAL PROPERTIES

BUILDING I.D.	MATERIALS STRUCTURAL STEEL	CONCRETE	REINFORCING STEEL	SOIL
1. Diesel Gen. Bldg. & Accum. Tank Enclosure	ASTM A7 (Fy = 33 ksi)	f'_c = 3,000 psi.	ASTM A15 & A 305 Int. Gr. (Fy = 40 ksi)	
2. Turbine Building & Pedestal	ASTM A7 (Fy = 33 ksi)	A) Footings & Grade Beams, f'_c = 2500 psi. B) Precast BMS & Wall Shield, f'_c = 2500 psi. C) All Other Cast In-place, f'_c = 3000 psi. D) Turbine Support Mat & Pedestal, f'_c = 3000 psi.	ASTM A15 & A 305 Int. Gr. (Fy = 40 ksi)	
3. Spent Fuel Pool/ Spent Fuel Chute	ASTM A7 (Fy = 33 ksi)	f'_c = 3000 psi	ASTM A15 & A 305 Int. Gr. (Fy = 40 ksi)	
4. Screen Well and Pumphouse	ASTM A7 (Fy - 33 ksi)	f'_c = 3000 psi	ASTM A15 & A 305 Int. Gr. (Fy = 40 ksi)	
5. Steel Vapor Container	A) Plate Material, ASTM A-300, Class A-201, Grade B, Fy = 32 ksi. B) Steel Columns ASTM A-283, Grade C Fy = 30 ksi C) Tie Rod Assembly AISI C-1020 & 4320 Fy = 30 ksi (Assume same as columns) D) Base Plate*, ASTM A284, Grade B, FY=27 ksi E) Anchor Bolts* F_t = 20 ksi (Fy≥32ksi) F_v = 10 ksi F_b = 20 ksi (S.S.) 25 ksi (D.S.)	f'_c = 3000 psi (Pedestal and Footings)	ASTM A15 & A 305 Int. Gr. (Fy = 40 ksi)	
6. Reactor Support Structure	ASTM A-300, Class A-201 Grade B, Fy = 32 ksi	A) Footing and Grade BMS, f'_c = 3000 psi B. Pedestals, Cols, Walls & All Others f'_c = 4000 psi	ASTM A15 & A 305 Int. Gr. (Fy = 40 ksi)	
7. Primary Aux. Building and Radioactive Tunnel	ASTM A7 (Fy = 33 ksi)	f'_c = 3000 psi	ASTM A15 & A 305 Int. Gr. (Fy = 40 ksi)	
8. MS/FW Support Structure	ASTM A7 (Fy = 33 ksi)	f'_c = 3000 psi	ASTM A15 & A 305 Int. Gr. (Fy = 40 ksi)	

Continued next sheet

TABLE D-1 (Continued)

9. Modifications*	ASTM A36 (F_y = 36 ksi); E70-XX Electrode; Bolts: ASTM A325F	Hilti Kwik-Bolts f'_c = 3000 psi	ASTM A615 Grade 40	
10. Fire Tank	A) Plate Material, ASTM A-283, Grade C B) Bracing System A-36 C) Anchor Bolts AISI (114-1)	f'_c = 4000 psi	ASTM A615 Grade 60 (F_y = 60 ksi)	ϕ = 36 ° q_{allow} = 6 KSF (static) q_{allow} = 8 KSF (static + seismic)

*Applicable to all structures

APPENDIX D: TABLE D-2
Allowable Loads for Hilti Bolts

KWIK-BOLT
AVERAGE ULTIMATE TENSILE & SHEAR LOADS*

Concrete Strength		2000 PSI		4000 PSI		6000 PSI	
Diameter	Embedment	Tension	Shear	Tension	Shear	Tension	Shear
5/8"	2 3/4"	54[illegible]0	11198	6600	11562	7700	13500
	3 1/2"	6250	11198	9100	11562	9560	13500
	4 1/2"	7000	11198	12000	11562	14500	13500
	5 1/2"	7550	13378	14300	15437	20300	15437
	6 1/2"	8025	13378	16000	15437	21000	15437
	7 1/2"	9000	13378	17000	15437	21000	15437
3/4"	3 1/4"	8155	13257	10159	17133	10860	18102
	4"	9700	13257	13400	17133	13700	18102
	5"	11700	13257	16500	17133	17600	18102
	6"	13800	15195	18000	18466	22500	21009
	7"	15800	15195	21000	18466	23600	21009
	8"	16000	15195	23000	18466	23600	21009
	9"	16000	15195	23500	18466	23600	21009
1"	4 1/2"	14000	27355	6000	26879	20500	32112
	5"	15500	27355	8900	26879	23441	32112
	6"	17600	27355	3441	26879	23441	32112
	7"	18200	27355	3441	26879	23441	32112
	8"	18200	27355	3441	34491	23441	36394
	9"	18200	27355	3441	34491	23441	36394
	10"	18200	27355	3441	34491	23441	36394
1 1/4"	5 1/2"	19000	36750	23000	35680	31200	45195
	6 1/2"	21600	36750	27100	35680	36500	45195
	7 1/2"	23600	36750	31100	35680	42000	45195
	8 1/2"	25100	39843	34600	35680	44400	47098
	9 1/2"	26200	39843	37800	35680	44400	47098
	10 1/2"	26800	39843	40900	35680	44400	49596

Actual Concrete Strengths

2178 psi 4027 psi 6119 psi

* Tension values obtained from best fit curve through mean values of test data. Curves and test data contained in A. A. Hanks Report No. 8784 (HILTI No. TR-111A).

Shear values are minimum mean values at each embedment based on failure across threaded section of the anchor.

ABBOT A. HANKS, TESTING LABORATORIES, SAN FRANCISCO, CA 94107

APPENDIX D

TABLE D-3
RECOMMENDED DAMPING VALUES*

Stress Level	Type and Condition of Structure	Percentage Critical Damping
Working stress, no more than about ½ yield point	a. Vital piping	1 to 2
	b. Welded steel, prestressed concrete, well reinforced concrete (only slight cracking)	2 to 3
	c. Reinforced concrete with considerable cracking	3 to 5
	d. Bolted and/or riveted steel, wood structures with nailed or bolted joints.	5 to 7
At or just below yield point	a. Vital piping	2 to 3
	b. Welded steel, prestressed concrete (without complete loss in prestress)	5 to 7
	c. Prestressed concrete with no prestress left	7 to 10
	d. Reinforced concrete	7 to 10
	e. Bolted and/or riveted steel, wood structures, with bolted joints	10 to 15
	f. Wood structures with nailed joints	15 to 20

* Source: NUREG/CR-0098

APPENDIX E

E.1. Buckling Stress for Straight Members in the Steel Vapor Container (see Figure E-1).

The procedure follows AISC Guidelines and it has built-in conservativeness in the assumptions:

A) The value $K\ell/r$ is calculated using K=1. Up to K=0.7 could be applicable in this case due to the end restraining conditions.

B) $K\ell/r$ falls in the range where the buckling phenomena is more sensitive to material strength than to Euler buckling.

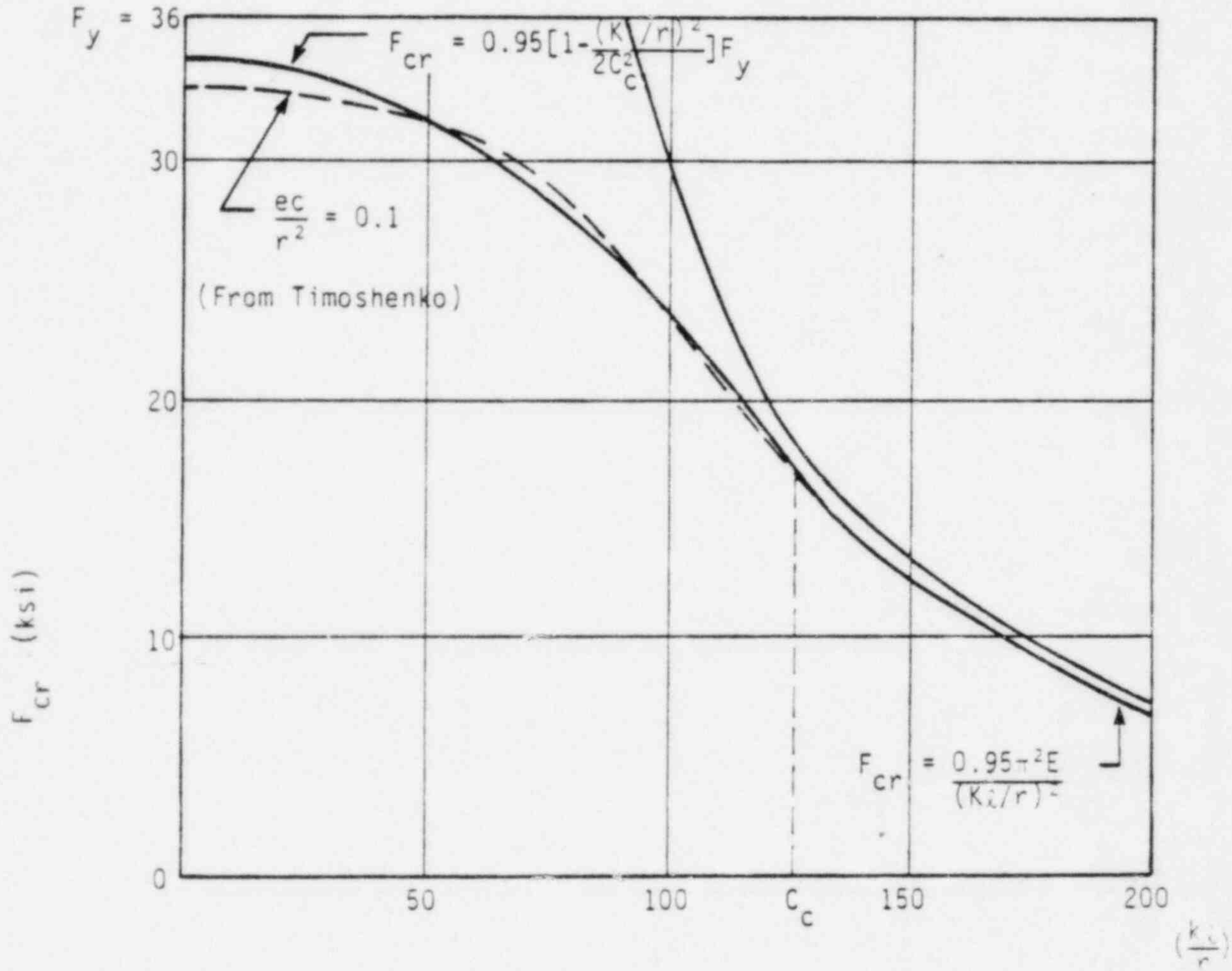
C) The F_y specified is less than the actual.

D) We can count on a ductility factor of at least 5.

E) 16 columns allow for stress redistribution.

FIGURE E-1

ECCENTRIC LOADING



F_y = 36 ksi and E = 29,000 ksi

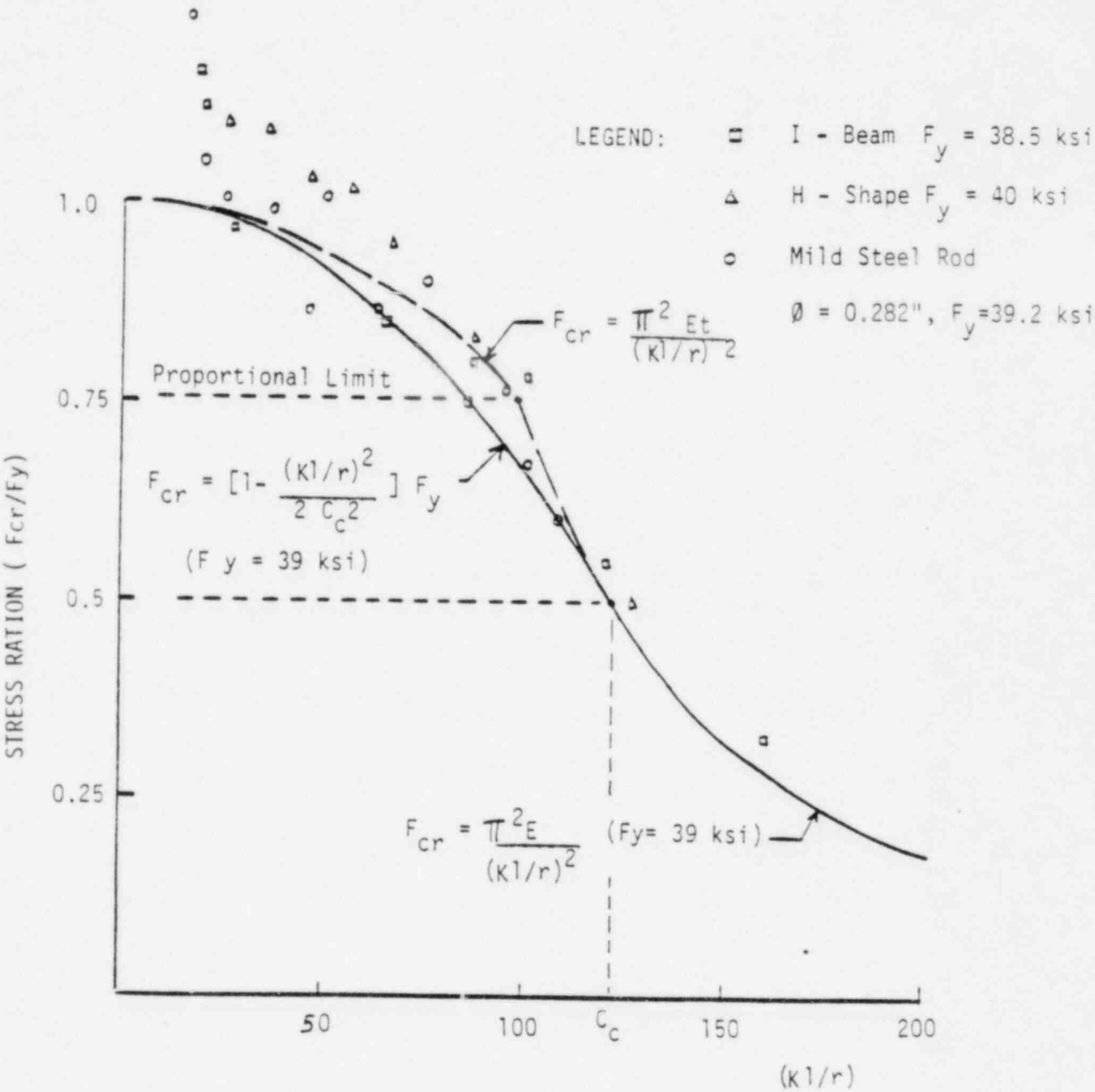
For the columns of SVC, O.D. = 42", t = 7/8", I = 23410 in^4

A = 113.05 in^2, r = 14.543", and c = 21"

$(^{ec}/r^2)$ = 0.1 → e = 1.01"

See Reference 4(z)

Figure E-1-A



COMPARISON OF BUCKLING STRESS

Figure E-1-B

TORSIONAL BUCKLING

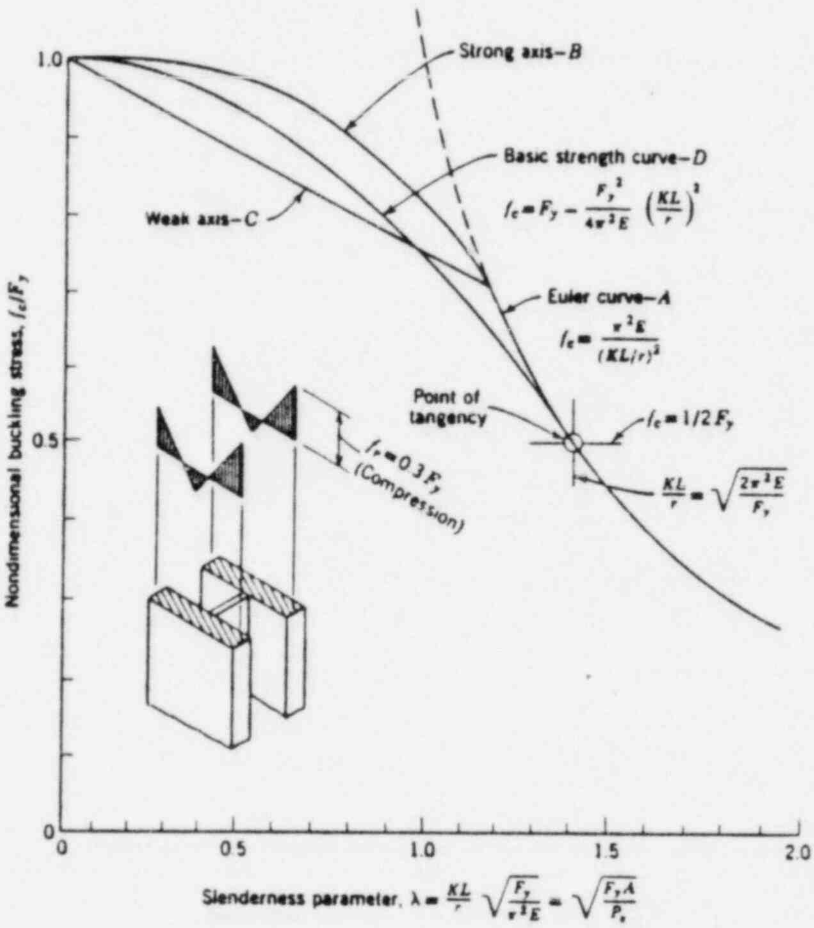


Fig. 9-11 Effect of residual stress on column strength of wide-flange shape. (Ref. 4.)

E.2 Shell Buckling Stress in the Steel Vapor Container

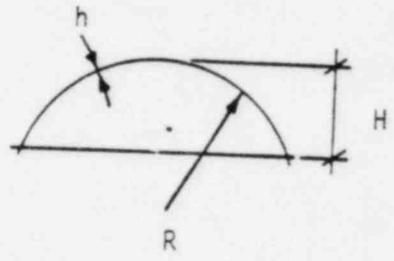
1. Spherical Cap

$\lambda = 2\,[3\,(1 - \nu^2)]^{-1/4}\ \ (H/h)^{1/2}$

$\nu = .3$

$H = R = 750"$

$h = .938"$ (differs)



$\lambda = 2\,[\,3\,(1-.3^2)]^{1/4}\ \ (750/.938)^{1/2} = 72.69$

$P_{cr} = 1.2E\,(h/R)^2\,(T_d)$

P_{cr} = critical pressure

E = 29000 ksi

T_d = test data Fig. 9.16, p. 340 of Reference 4(aa)

T_d = 72.69 out of scale
Test data is for very well manufactured specimens

H	λ	T_d	P_{cr}[ksi]	N_ϕ [ki] (= RP_{cr})
30	14.54	0.67	0.036	27.0
20	11.87	0.67	0.036	27.0
10	8.39	0.67	0.036	27.0
5	5.94	0.67	0.036	27.0
2	3.75	0.5	0.027	20.3
1	2.65	0.6	0.033	24.8
2.28	4	0.4	0.022(min)	16.5
		1.0	0.055 (axisymmetric)	41.3

2. Complete Shell

$$P_{cr} = \frac{2E}{[3(1-\nu^2)]^{1/2}} \left(\frac{h}{R}\right)^2$$

$$= \frac{2 \cdot 29000}{[3(1 - .3^2)]^{1/1}} \left(\frac{.938}{750}\right)^2$$

P_{cr} = 0.055 ksi

N_ϕ = 41.3 ki

This critical pressure is in poor agreement with test data due to two factors:

1. neglect of nonlinearity in the prebuckling analysis.
2. the influence of initial imperfections.

3. Experimental Evidence

$$P_{cr} = cE \left(\frac{h}{R}\right)^2$$

where c from experiments(1) is much smaller than

$2/\ [3(\ 1 - \nu^2)]^{1/2} = 1.21$; c = 0.06 to 0.32

c	P_{cr}	N_ϕ
0.06	0.003	2.0
0.10	0.005	3.4
0.15[2]	0.007	5.1
0.20	0.009	6.8
0.25	0.011	8.5
0.30	0.014	10.2
0.36	0.016	12.2
1.21	0.055	41.2

Notes:

(1) See References 4(bb) and 4(cc).

(2) Adopt c = 0.15

This conservative value considers the influence of neglecting nonlinearities in the prebuckling analysis and those of initial imperfections.

4. Allowable Buckling Stresses

Shell thickness t (inches)	Allowable Stresses, N_ϕ, Ksi
0.875	4.7
0.888	4.8
0.938	5.1
1.0	5.4
1.25	6.8
1.75	9.5
3.0	16.3

APPENDIX F

Memorandum

PROJECT MEMORANDA #002

To: All Engineers - Yankee Rowe (SEP)

Date: December 22, 1981

From: Eric van Stijgeren

Job No: 81060

Subject: Allowable Nozzle Loads, Valve Acceleration, Alternate Criteria and Hanger Uplift Criteria

Copies:

The purpose of this memo is to allow you to follow a consistent guide line in pipe stress analysis on the subject mentioned above.

The details of the subject is attached herewith.

If you have any further questions, please contact your supervisor

Eric van Stijgeren

Eric van Stijgeren
Project Manager

EvS:mec

Attach: 1.) Allowable Nozzle Loads Criteria
2.) Allowable Valve Acceleration Criteria
3.) Allowable Stress Criteria for LLL-Spectra
4.) Hanger Uplift Criteria

CYGNA

APPENDIX G

Memorandum

To: Eric van Stijgeren

Date: December 16, 1981

From: R. Hamati

Job No: 80023

Subject: Yankee Rowe Valve Accelerations Allowables

Copies: Project File

In response to your request, we propose that the following limits on valve accelerations be adhered to for the dynamic analysis of piping systems on the Yankee Rowe project.

1. Resultant horizontal acceleration ≤ 4.25 gs
2. Vertical acceleration ≤ 3.0 gs

This, serving as a design basis, should minimize the number of problems which may arise when the actual valve accelerations from the analyses are submitted to the vendors for their approval.

Naturally, judgement should be used when making changes for cases in which a valve acceleration exceeds these valves by a relatively small amount.

D. Hamati

Raymond Hamati

REH:LJW:mec

APPENDIX H

Memorandum

To: Eric van Stijgeren

Date: December 3, 1981

From: Herman Suryoutomo

Job No: 80023

Subject: Yankee Rowe Alternate Stress Allowable Criteria

Copies: Distribution

In response to your request, we propose that the following criteria be implemented.

Due to the similarity between ANSI B31.1 and ASME B & PV Code, Section III for class 2 components, this Criteria references the latter for allowable stress values.

At the present time, the piping systems at Yankee Rowe have been analyzed for seismic effects using the "YCS" spectra. Loading combinations which include this seismic case have been evaluated using an allowable stress equal to 1.8 S_h. This allowable is equivalent to that which is used for an emergency or Service Level C condition as defined in ASME B & PV Code, Section III, NCA-2142.2

It is now necessary to analyze these piping systems using the "LLL/TERA" spectra. This is a much more severe loading and as such should be considered as a faulted or Service Level D condition. Thus, EQ.(9), NC-3652.2 may be satisfied using a stress limit of 2.4 S_h, (NC-3611.2).

This interpretation is similar to the approach which is being used by others in the industry.

Attached is the proposed revision to the "Seismic Reevaluation Criteria", Document No. DC-1.

HS:LJW:mec

DISTRIBUTION

R. Falciani	M. DeGuzman	B. Kacyra	D. Willis
W. Gang	P. Joadder	N. Chauhan	S. White
J. Vallenas	J. Ameri	J. Famiglietti	L. Manning
L. Privari	P. DiDonato	B. Brown	

APPENDIX I

81060-003

141 Battery Street Suite 400 San Francisco CA 94111

415 433-2100

December 23, 1981

Yankee Atomic Electric Company
1671 Worcester Road
Framingham, MA 01701

Attention: Mr. John Hoffman

Subject: Allowable Nozzle Loads

Dear John:

Attached please find a telecon regarding the subject mentioned above.

If you have any questions in regards to the telecon, please feel free to call me.

Merry Christmas.

Sincerely,

Eric van Stijgeren
Project Manager

EvS:mec

Encl. (1)

cc: A. Roudenko
Document Control Center

CYGNA

Communications Report

Company	Cygna Energy Svc. ☒ Telecon	☐ Conference Report	
Project	Yankee Rowe (LLL/TERA fix)	Job No.	81060
		Date	12/23/81
Subject	Allowable Nozzle Loads	Time	10:30 AM
		Place	

Participants:

P. Joadder of Cygna

A. Roudenko of YAEC

Item	Comments	Req'd Action By
	Mr. A. Roudenko discussed the subject with Dr. Joadder and made the following comments:	
1	In the process of fixing any sensitive lines, such as Reactor coolant loop, Cygna should not look to add any additional restraints as a means to satisfy the conservative allowable nozzle loads developed by Cygna.	
2	Cygna shall transmit to YAEC all such nozzle loads which did not qualify as per Cygna developed allowable nozzle load criteria.	
3	YAEC will take necessary actions on the nozzle loads mentioned in item 2 above.	

Signed: [signature], Pranabesh Joadder Page 1 of 1

Distribution: A. Roudenko, Eric van Stijgeren, John Hoffman

1030 00

EQUIPMENT NOZZLE ALLOWABLES

When the allowable loading on an equipment nozzle, due to the connected piping, is not supplied by the vendor, the following criteria is to be implemented.

The maximum loading, imposed by connected piping, on an equipment nozzle shall be within limits such that the following equation is satisfied:

$$\frac{Fr}{F} + \frac{Mr}{M} \leq 1 \qquad \text{(Eqn. 1)}$$

Where:

$F = aAS/10$ (Eqn. 2)*

$M = aZS$ (Eqn. 3)*

F_r = Actual resultant force on nozzle (lbs.)

M_r = Actual resultant moment on nozzle (in-lbs.)

A = Pipe metal area (in^2)

Z = Pipe section modulus (in^3)

S = Limiting pipe stress (PSI) as follows:

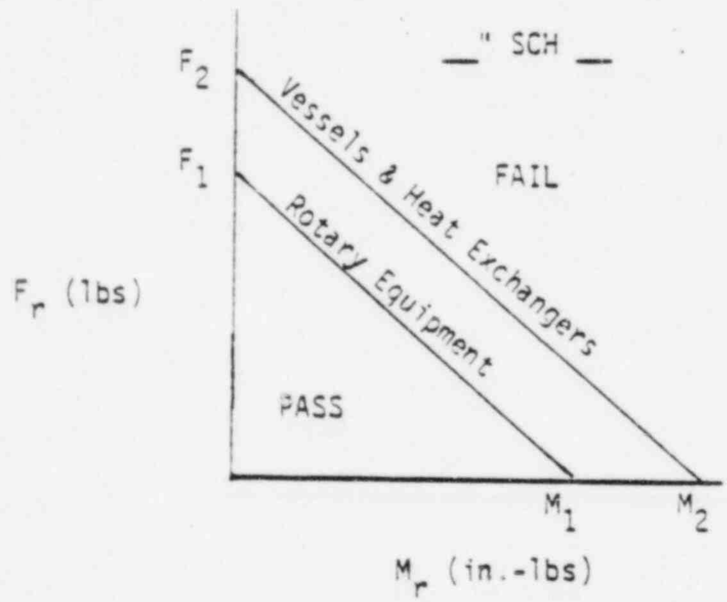
for 4 in. & under: S = 8000 PSI
6 in. to 8 in: S = 6000 PSI
10 in. and over: S = 4000 PSI

a = 1.0 Normal operation condition
    1.2 Upset operating condition
    1.8 Emergency operating condition
    2.4 Faulted operating condition

<u>LIMITATION</u>: Any individual component of F_r (i.e. F_x,F_y,F_z) or M_r (i.e. M_x, M_y,M_z) may not exceed 80% of F or M respectively.

*(NOTE: These values are for nozzles connected to rotary equipment. For nozzles connected to vessels and heat exchangers, these values may be increased by 35%).

Using this approach, a series of graphs can be developed as shown in the figure below.



APPENDIX J

ROD HANGER UPLIFT STUDY

PREPARED FOR

YANKEE ATOMIC ELECTRIC COMPANY
WESTBOROUGH, MASSACHUSETTS

Prepared by: A. F.
Aly Fawzy

Approved by: John Minichiello
John Minichiello

CYGNA ENERGY SERVICES
141 BATTERY STREET, SUITE 400
SAN FRANCISCO, CALIFORNIA 94111

TABLE OF CONTENTS

I. EXECUTIVE SUMMARY

I.1 Purpose

This preliminary report is devoted to the study and development of an alternative method for analyzing piping systems with rod hangers. It is intended for the use of Yankee Atomic Electric Company, Westborough, Massachusetts.

I.2 Scope

The criteria developed here are based on the following:

1 - Study of existing criteria or methodology.

2 - Study of the results of the existing procedure.

3 - Field inspection by Cygna personnel.

4 - Engineering calculations by Cygna personnel.

5 - Professional judgment.

II. LITERATURE REVIEW

II.1 Statement of the Problem

Due to the very high slenderness ratio of the rod hanger, it is actually a one-way restraint element. Because of this nonlinear behavior, the rod hanger is currently assumed rigid over the entire displacement field. According to existing criteria, if after analysis it is discovered that a rod is in compression, this hanger is then removed from the model. This procedure is repeated until no compressive forces exist in any of the rod hangers.

The previous methodology has the following apparent disadvantages:

a - It is lengthy, tedious and expensive.

b - The so called "convergence" is not guaranteed. Even where the model has converged, the solution may not be unique.

c - It does not represent the "actual" system.

d - It ignores the impact loads on both the hanger and the pipe.

e - Response spectrum analysis assumes the same prescribed input support motion is forced at the node where the rod hanger is attached to the pipe; this is a completely unrealistic assumption.

II.2 Mathematical Background

The wire element can be handled by any iterative procedure.

The force displacement models for real support and wire support are shown in Fig. 1. Thus, if the nonlinear effects of the impact are ignored, it is obvious that model (b) operates in the upper half of the (x,y) space domain exactly the same as model (a). This could be explained physically by the fact that both systems or models are conservative systems in an energy sense.

III. ANALYSIS PROCEDURE

III.1 Introduction

In order to find an acceptable linear analysis procedure, the problem and the pipe erection method must be fully understood. Rod hangers (Fig. 2a, 2b, 2c, 2d) are only a fairly stiff wire. Most rod hangers have a turnbuckle setting to allow for construction tolerance. After all pipe segments are welded together, the forces in the rod hangers are set to a known prescribed force $\{F_p\}$ using the turnbuckles. Hence:

"Rod Hangers Have Tensile Known Prescribed Forces Due to Gravity"

III.2 Proposed Solution

III.2.a Solution Steps

STEP 1:

Suppose that the forces in the hangers due to earthquakes only are $\{F_s^H\}$. Obviously, this vector can be either positive or negative.

Let $\{F_n^H\} = \{F_p\} - \{F_s^H\}$
where $\{F_p\}$ is the dead load force in the hanger. $\{F_n^H\}$ are the minimum net forces in the rod hangers due to {D.L. + SEISMIC}. If all the elements in the vector $\{F_n^H\}$ are positive, this means that there are no "uplift" forces in any of the hangers.

Suppose that parts of this vector are negative elements, i.e.

$$\{F_n^H\} = \left\{\frac{+ve}{-ve}\right\} = \left\{\frac{F_{np}^H}{F_{nn}^H}\right\}$$

where $\{F_{np}^H\}$ are the positive elements,

and $\{F_{nn}^H\}$ are the negative elements.

Now examine this condition

if $|F_{nn}^H|_{max} < |F_{np}^H|_{min}$ GO TO STEP 2

if $|F_{nn}^H|_{max} > |F_{np}^H|_{min}$ GO TO STEP 3

STEP 2:

Apply the force vector $\{F_{nn}^H\}$ as a static load after modifying the system by removing the hangers at the corresponding nodes.

The final deadweight and seismic stress condition $\{\sigma\}$ will be:

$$\{\sigma\} = \{\sigma_s\} - \{\sigma_{d.1}\} + \{\overset{MR}{\sigma}\}$$

Where

$\{\sigma_s\}$ is the stress vector due to seismic forces with the $\{F_{nn}^H\}$ supports removed.

$\{\sigma_{d.1}\}$ is the stress vector due to the original dead load analysis.

$\{\overset{M}{\sigma}{}^{R}\}$ is the residual stress due to the force vector $\{F^H_{nn}\}$ applied on the modified system.

STEP 3:

As mentioned before, if

$$|F^H_{nn}|_{max} > |F^H_{np}|_{min}$$

it is likely that releasing the node which has a high uplift force $\{F^H_{nn}\}$ will cause the neighboring hangers to uplift and so on in a chain-like reaction. This is called "progressive failure" or "uncontrolled failure."

Hence in this step <u>all</u> the hangers are to be removed, yet the dead load forces (F_D) have to be preserved.

Example:

Consider the following example:

Suppose after the construction of the beam shown in Fig. (3a) the tension in the hanger H was set to (F_D).

Since this system is a conservative system, thus, this problem is physically equivalent to the problem shown in Fig. (3b) which is a simply supported beam with the same load intensity ω and a force = F_D as an applied force. Now suppose that the problem in Fig. (3a) is subjected to an additional load case, e.g., seismic. This later problem can be solved as follows:

1. Solve problem shown in Fig. (4b) for the second load case, i.e., seismic. (S2)

2. Solve problem shown in Fig. (4a) for the first load case, i.e., gravity. (S1)

3. The solution of the problem shown in Fig. (4a) SF = S2 - S1.

The flow chart of the proposed solution is shown in Fig. (5).

III.2.b Final Hanger Forces

In order to find out the final force in any rod hanger, the final displacement δ_F at the attachment point has to be calculated, and the final force in the hanger is equal to

$$F_F^h = \delta_F \cdot K_{ij} \qquad \text{if } \delta_F \text{ is} > 0$$

$$F_F^h = 0 \qquad \text{if } \delta_F < 0$$

where:

F_F^h = Final force in the rod hanger

δ_F = Final displacement at the attachment point

K_{ij} = Stiffness coefficient of the pipe at point i (attachment point) in the j direction (hanger direction). (See Fig. 6.)

III.2.c Momentum Recovery

The lost momentum has to be recovered to see whether or not the impact forces are high enough to overstress either in the pipe or the rod hanger. For this reason, it is advisable to reduce the allowable forces in the hangers which experience uplift forces by one-half (use $F^{u}_{all} = \frac{1}{2} F_{all}$).

III.2.d Unknown Hanger Forces

In case the prescribed hanger forces $\{F_p\}$ are unknown (or missing for any reason), a gravity analysis should be performed with all attachment points as rigid supports, and then the reaction vector only at the hanger points $\{R\}_H$ should be replaced by the prescribed forces $\{F_p\}$. Then the same steps should be carried out.

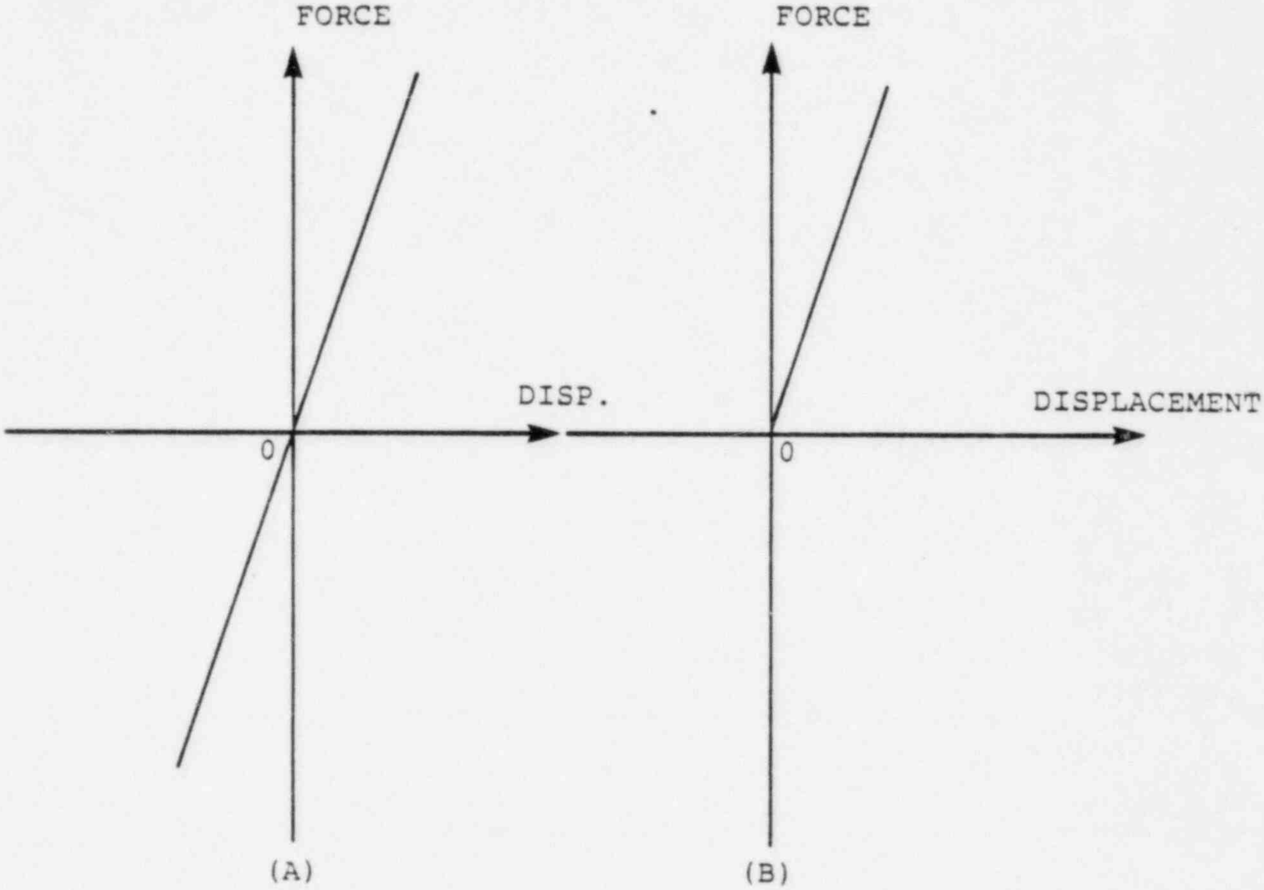
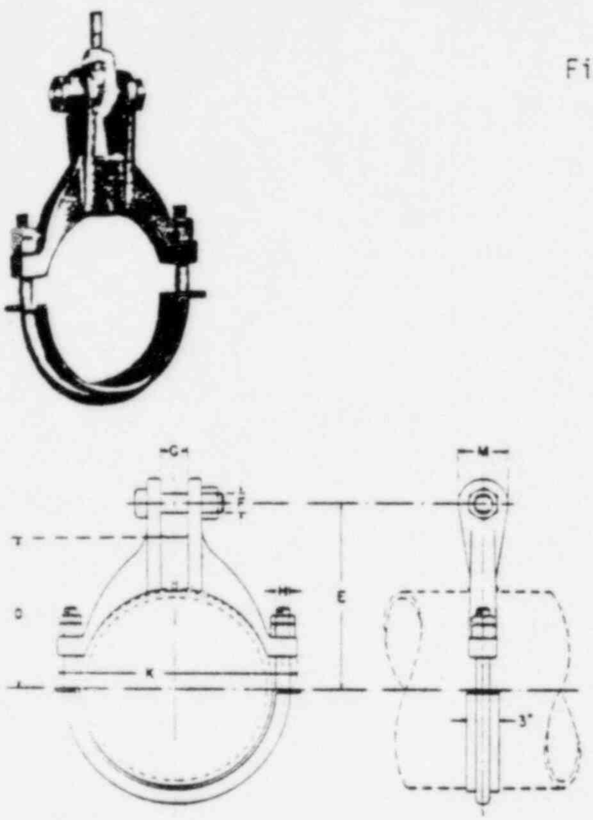


Figure 1

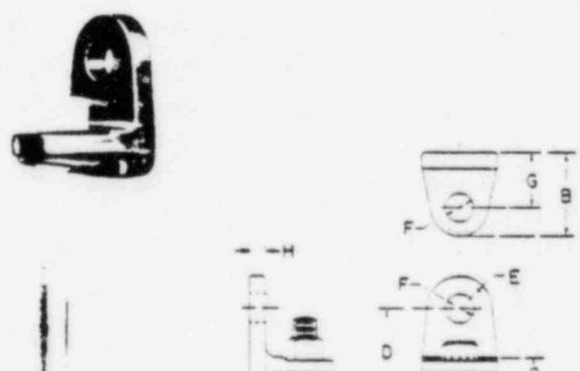
FORCE DISPLACEMENT
MODELS

Fig. 2



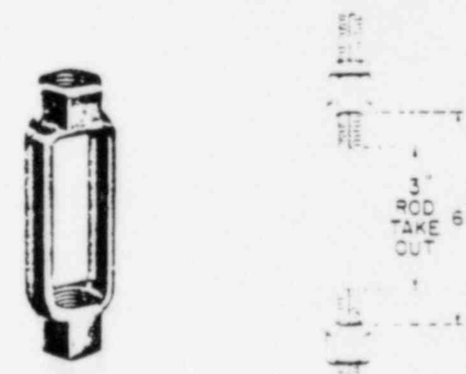
A

B



C

turnbuckle
fig. 230



D

FIGURE 2

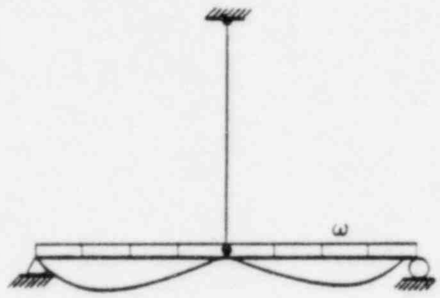


Fig. 3.a

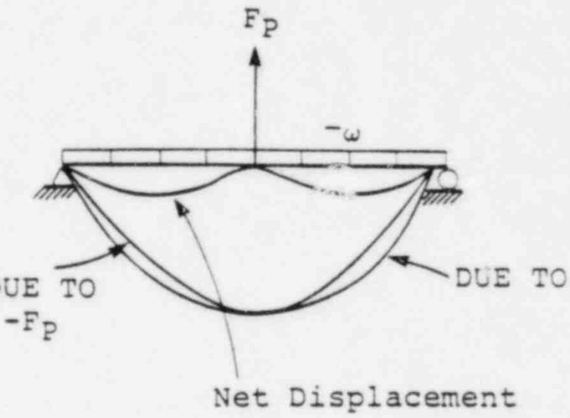


Fig. 3.b

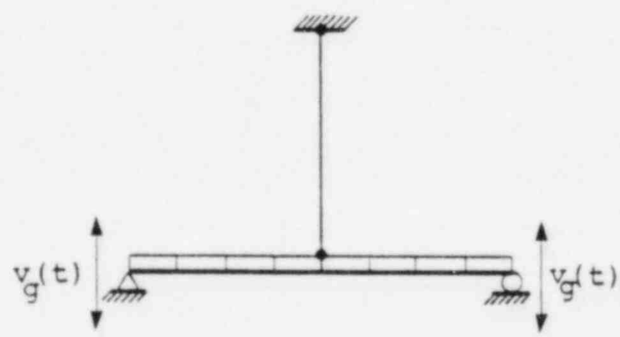


Fig 4.a

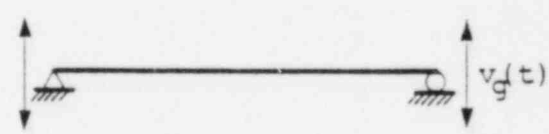
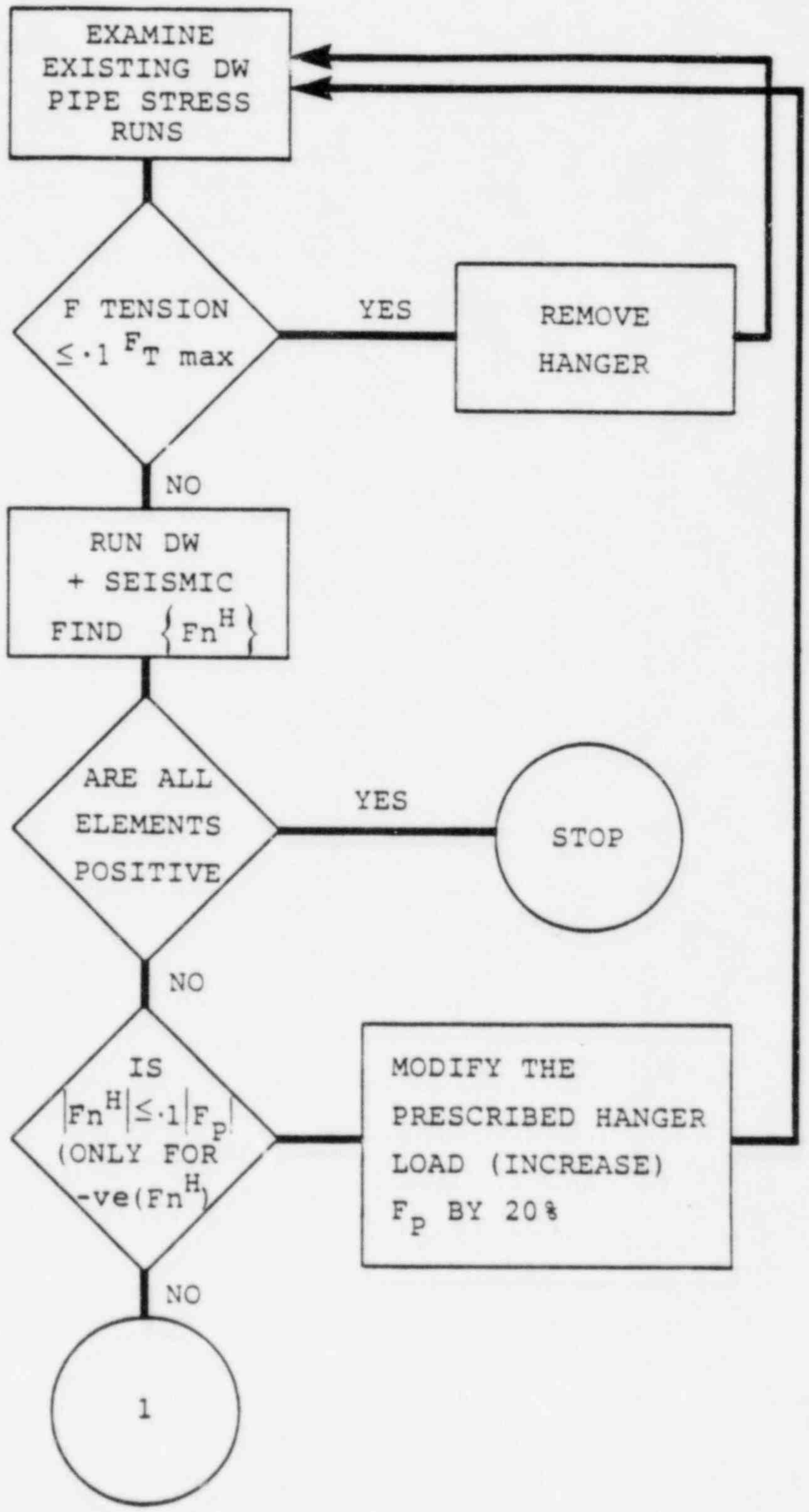
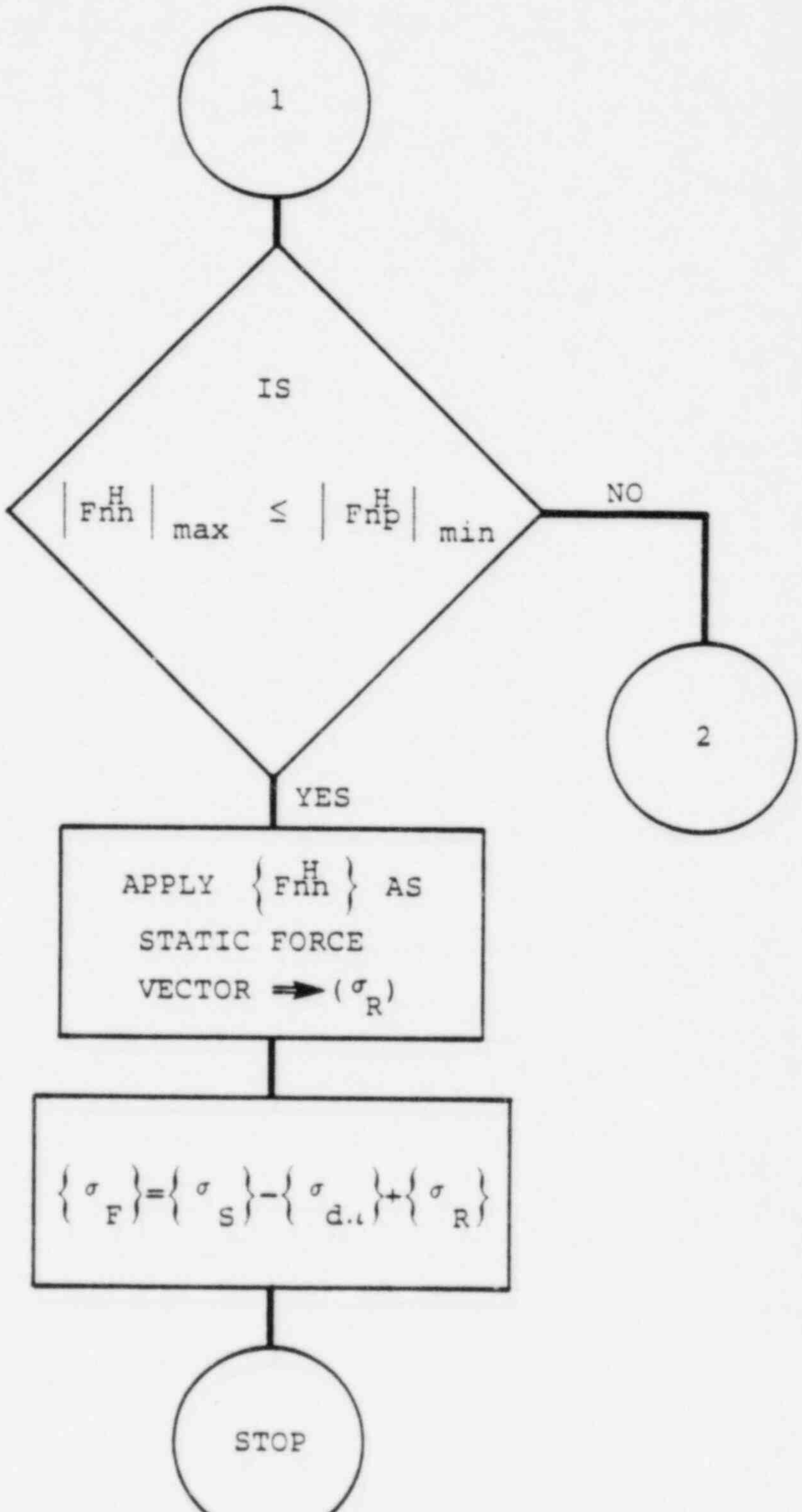


Fig. 4B

FLOW CHART


EXAMINE EXISTING DW PIPE STRESS RUNS
F TENSION
≤·1 $F_{T\,max}$
YES
REMOVE HANGER
NO
RUN DW + SEISMIC FIND $\{Fn^H\}$
ARE ALL ELEMENTS POSITIVE
YES
STOP
NO
IS $|Fn^H| \leq \cdot 1 |F_P|$ (ONLY FOR -ve(Fn^H))
MODIFY THE PRESCRIBED HANGER LOAD (INCREASE) F_P BY 20%
NO
1


1
IS
$\left| F_{nh}^{H} \right|_{max} \leq \left| F_{np}^{H} \right|_{min}$
NO
2
YES
APPLY $\left\{ F_{nh}^{H} \right\}$ AS
STATIC FORCE
VECTOR $\Rightarrow (\sigma_R)$
$\left\{ \sigma_F \right\} = \left\{ \sigma_S \right\} - \left\{ \sigma_{d.l} \right\} + \left\{ \sigma_R \right\}$
STOP

2

EXTRACT EIGEN VALUES & EIGEN VECTORS FOR THE SYSTEM WITHOUT ANY ROD HANGERS ($\sigma 2$)

SOLVE FOR THE FINAL STRESS STATE $\{\sigma_F\} = \{\sigma_2\} - \{\sigma_{d.l}\}$

STOP

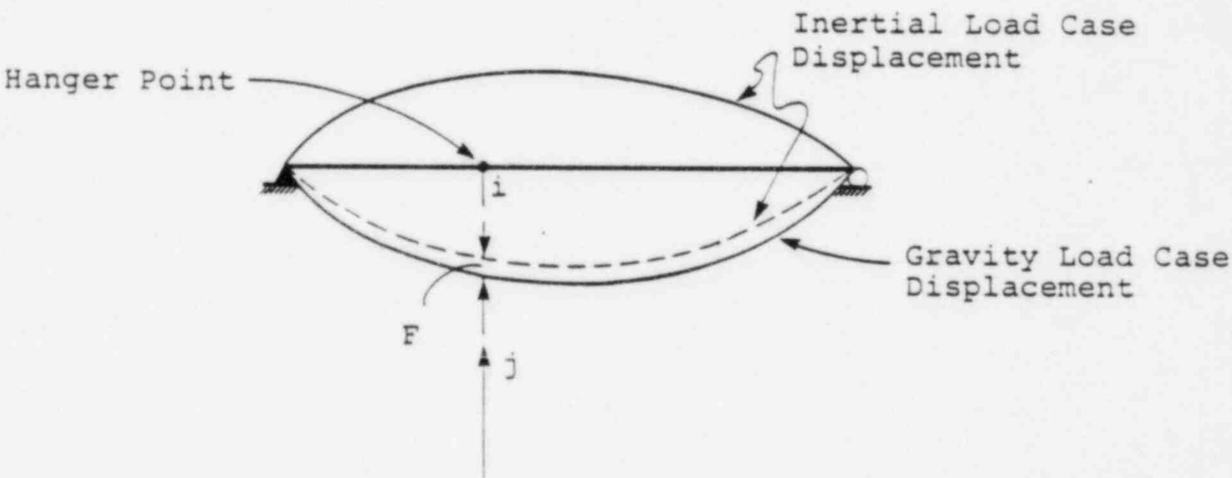

Inertial Load Case
Displacement
Hanger Point
i
Gravity Load Case
Displacement
F
j

Fig. 6

APPENDIX K

COMPUTER PROGRAMS

A. Earthquake Engineering Systems, Inc., INSPEC, Version 1.2, October, 1980.

B. NISEE/Computer applications ANSR-I, March and December, 1975.

C. NISEE/Computer Applications, DRAIN 2D, Version 8/75, August 1975.

D. Earthquake Engineering Systems, Inc., BATS, Version 6.1, November 1980.

E. Earthquake Engineering Systems, Inc., EESAP, Version 1.0, June 1979.

F. Swanson Analysis Systems, Inc., ANSYS, Version 3, July 1, 1979.

G. Earthquake Engineering Systems, Inc., MOST, Version 1.1, November, 1979.

H. Earthquake Engineering Systems, Inc., NRCQUAKE Version 1.0, December, 1980.

I. Arthur D. Little, Inc. ADLPIPE, Version - ADLPIPE (FAST) February, 1977, Rev. 4C.

J. CDC, PIPESD, Version 6.0, August, 1979.

K. Mahin, S.A. and V.V. Bertero, RCCOLA, A Computer Program for Reinforced Concrete Column Analysis, Department of Civil Engineering, University of California, Berkeley, August, 1977.

APPENDIX K

VARIOUS NONLINEAR PROGRAM CAPABILITIES

Program	Availability	Dimension	Input Ground Motion	Steel Brace Element	Steel Beam-Column Element	Reinforced Concrete Beam-Column Element	Reinforced Concrete Shear Panel Element
DRAIN-2D	Public	2-D	Yes	Yes	Yes	Yes	Yes
DRAIN-TABS	Public	3-D	Yes	Yes	Yes	Yes	Yes
NONSAP	Public	3-D	No	No	No	No	Yes
ADINA	Private	3-D	No	No	Yes	No	Yes
ANSR	Public	3-D	Yes	Yes	No	No	No
SAKE	Public	2-D	Yes	No	No	Yes	Yes
MARC-CDC	Private	3-D	Yes	Yes	Yes	No	No
ANSYS	Private	3-D	Yes	Yes	Yes	No	No

See Reference 4(w)

APPENDIX L

ANALYSIS OF BURIED PIPES

PREPARED FOR

YANKEE ATOMIC ELECTRIC COMPANY

WESTBOROUGH, MASSACHUSETTS

Prepared by: ____________________
Aly Fawzy

Approved by: ____________________
John Minichiello

Cygna Energy Services
141 Battery Street, Suite 400
San Francisco, CA 94111
February 15, 1982

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PART I VERTICAL PIPE PENETRATION

PART II NON VERTICAL PIPES

PART I

VERTICAL PIPE PENETRATION

1. Introduction

Two types of behavior should be considered in predicting soil resistance of pipes in sand. Near the surface the pipe may push a soil wedge ahead of it, while at some depth below the surface overburden pressure will prevent wedge action and soil may flow around the pipe.

In estimating relative density of the sand around the pipe to predict E_s (modulus of elasticity) and ultimate resistance values, the method of pipe installation should be considered. If the pipe is driven into place, it is logical that the sand will densify by the driving operation, at least within about three pipe diameters of the pipe centerline. Under these conditions the use of sand density before driving may be overconservative. However, if the pipes are jetted into place, the escaping jet water may tend to increase the soil void ratio in the material adjacent to the pipe, lowering the actual density of the sand below in situ measured values obtained from soil borings, penetration tests, etc. In such cases, use of in situ density may be unconservative.

2. Ultimate Pipe Capacity

2.1 Shallow Pipe Penetrations

The ultimate pipe resistance can be studied using the well-known failure wedge method shown in Fig. (1a). When the pipe (assumed rigid) has moved a sufficient distance, a passive-type wedge failure occurs. Straight failure surfaces are assumed and the curvature of the pipe is neglected.

Active earth pressure force on the cylinder is

$$F_a = 1/2\, K_a \gamma\, bx^2 \tag{1}$$

where b and X are shown in Fig. (1a), K_a is the coefficient of active earth pressure $\{K_a = \tan^2 (45° - \frac{\theta}{2})\}$ and γ is the effective unit weight of the sand. This active force acts on the rear of the pipe in the direction of movement.

Forces which resist sliding are also computed using the wedge of Fig. (1a). Resistance on each plane is computed separately. On plane AEFB, the ultimate resistance on a strip of area dA is

$$F_s = \int_0^x \sigma \cos \beta \, dA + \int_0^x \tau \sin \beta \, dA \tag{2}$$

Where σ and τ are the normal and shear stresses on plane AEFB.

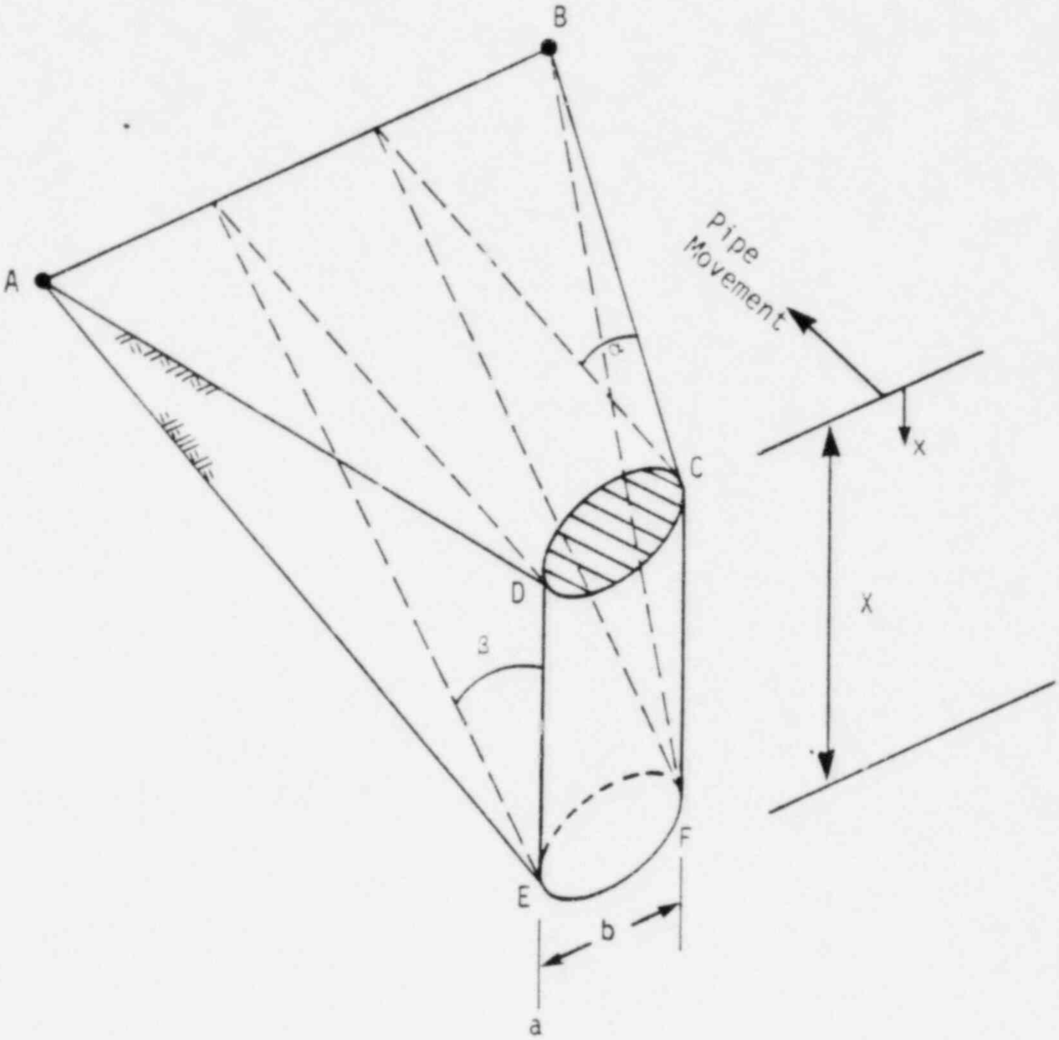


FIG. 1a

ASSUMED PASSIVE WEDGE-TYPE FAILURE

From the Mohr-Coulomb diagram of Fig. (1b), the stresses on the plane are

$$\sigma = 1/2\,(\gamma\, x + K_p\gamma A) + 1/2\,(\gamma x - K_p\gamma x)\ \sin\theta \qquad (3)$$

and

$$\tau = 1/2\ (K_p\gamma x - \gamma x)\ \cos\theta \qquad (4)$$

where

A is the total surface area

K_p is the coefficient of passive earth pressure

$$=\tan^2\left(45° + \frac{\theta}{2}\right)$$

The differential area is expressed as

$$dA = \{b+2\ \tan\alpha\tan\beta\ (X-x)\}\ \sec\,\beta\ dx \qquad (5)$$

After substitution and integration

$$F_s = K_p\gamma X^2\left\{\ \frac{b}{2}\ + \frac{X\ \tan\alpha\tan\beta}{3}\ \right\} \qquad (6)$$

Ultimate lateral resistance on planes ADE and BCF is given by

$$F_u = 2\ \int_0^X \tau\ \cos\alpha\ dA - 2\ \int_0^X \sigma\ \sin\alpha\ dA \qquad (7)$$

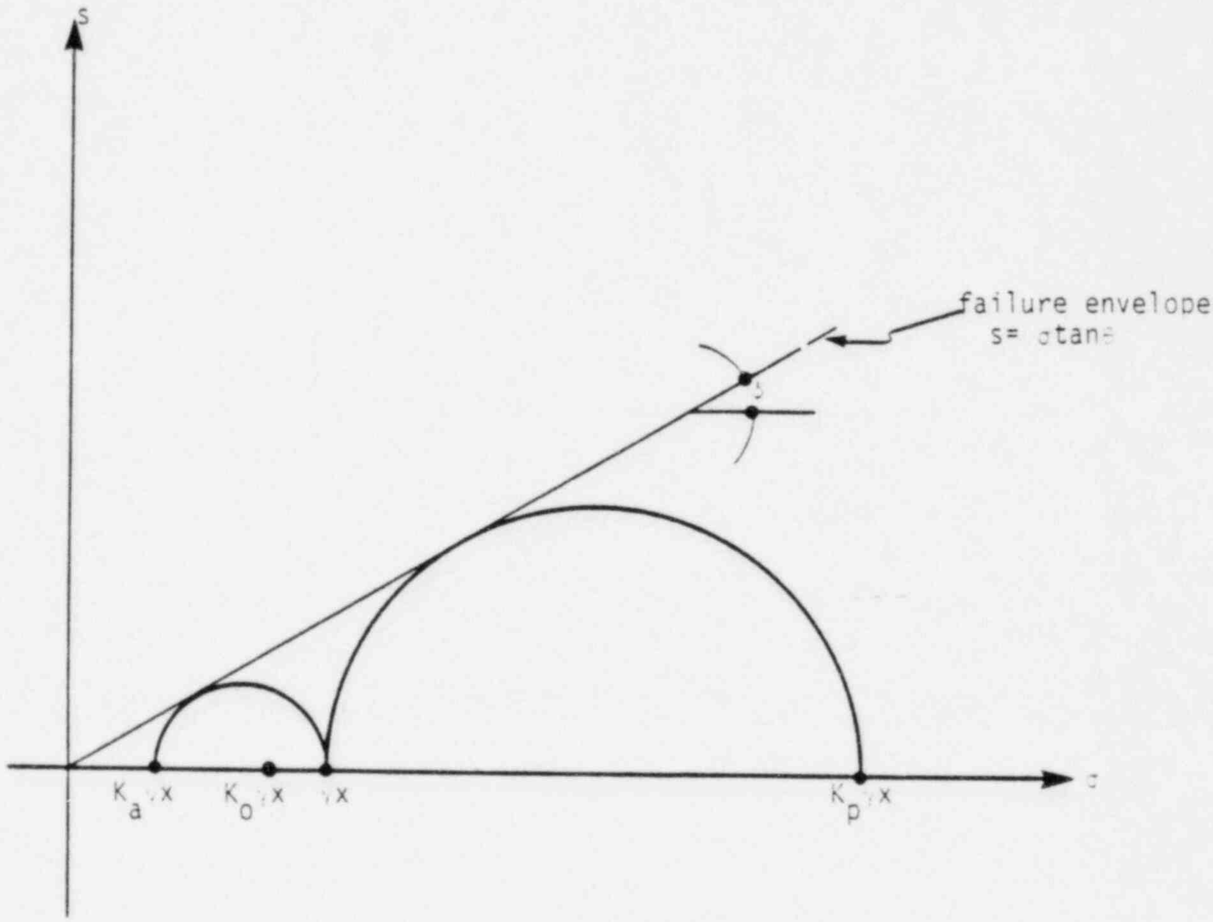


FIG. 1b

MOHR-COLUMB DIAGRAMS

Passive earth pressure conditions do not appear to exist on planes ADE and BCF, so the stresses σ and τ will be defined in terms of some earth pressure coefficient K, such that

$$\sigma = K\gamma x \quad (8)$$

and

$$\tau = K\gamma x \tan\theta \quad (9)$$

On these planes (ADE, BCF) dA is defined as

$$dA = (X-x) \sec\alpha \tan\beta \, dx \quad (10)$$

Substituting and integrating gives

$$F_u = \frac{X^3}{3} \gamma K \tan\beta (\tan\theta - \tan\alpha) \quad (11)$$

Total ultimate resistance of the pipe

$$F_t = F_x + F_u - F_a \quad (12)$$

$$F_t = X^2\gamma \left[\frac{b}{2}(K_p - K_a) + \frac{X}{3} K_p (\tan\alpha \tan\beta) + \frac{X}{3} K \tan\beta (\tan\theta - \tan\alpha)\right] \quad (13)$$

To obtain ultimate soil resistance per unit length of pipe Eq. (13) must be differentiated with respect to depth X, thus

$$q_{u\ell} = \frac{dF_t}{dx} = \gamma x \left[b(K_p - K_a) + K_p x (\tan\alpha \tan\beta) + K x \tan\beta (\tan\theta - \tan\alpha)\right] \quad (14)$$

Where β is given by the Mohr-Couloumb theory as $(45° + \frac{\theta}{2})$.

Values for α and K have not yet been defined. At its maximum, the angle α could equal θ, giving zero resistance on planes ADE and BCF. When α is equal to zero, Reese's solution corresponds to that of Terzaghi for a short wall in sand and K is approximately equal to the coefficient of at-rest earth pressure K_0. As α increases to θ the coefficient K should decrease until it equals K_a when $\alpha = \theta$. This reasoning appears logical because the sides of the failure wedge will tend to expand if $\alpha = \theta$ when the wedge is pushed up and out. Reese estimates α to be a function of sand relative density, with values of $\frac{\theta}{3}$ to $\frac{\theta}{2}$ for loose sand and θ for dense sand.

Eqs. 13 and 14 were derived assuming no shearing on plane DEFC. This assumption (ideally smooth pipe) is in error, however, it is difficult to estimate the earth pressure along the curved surface and thus predict the magnitude of these stresses. In any case, neglecting soil resistance from pipe friction gives an error on the safe side.

2.2 Deep Pipe Penetrations

At some depth below the ground surface the sand in front of the deformed pipe will not be pushed up and out by wedge action but will tend to flow around the pipe, from front to back.

Fig. (2) shows Reese's assumed condition for soil flow. The effect of pipe shape is neglected; in fact, a square pipe section is actually considered.

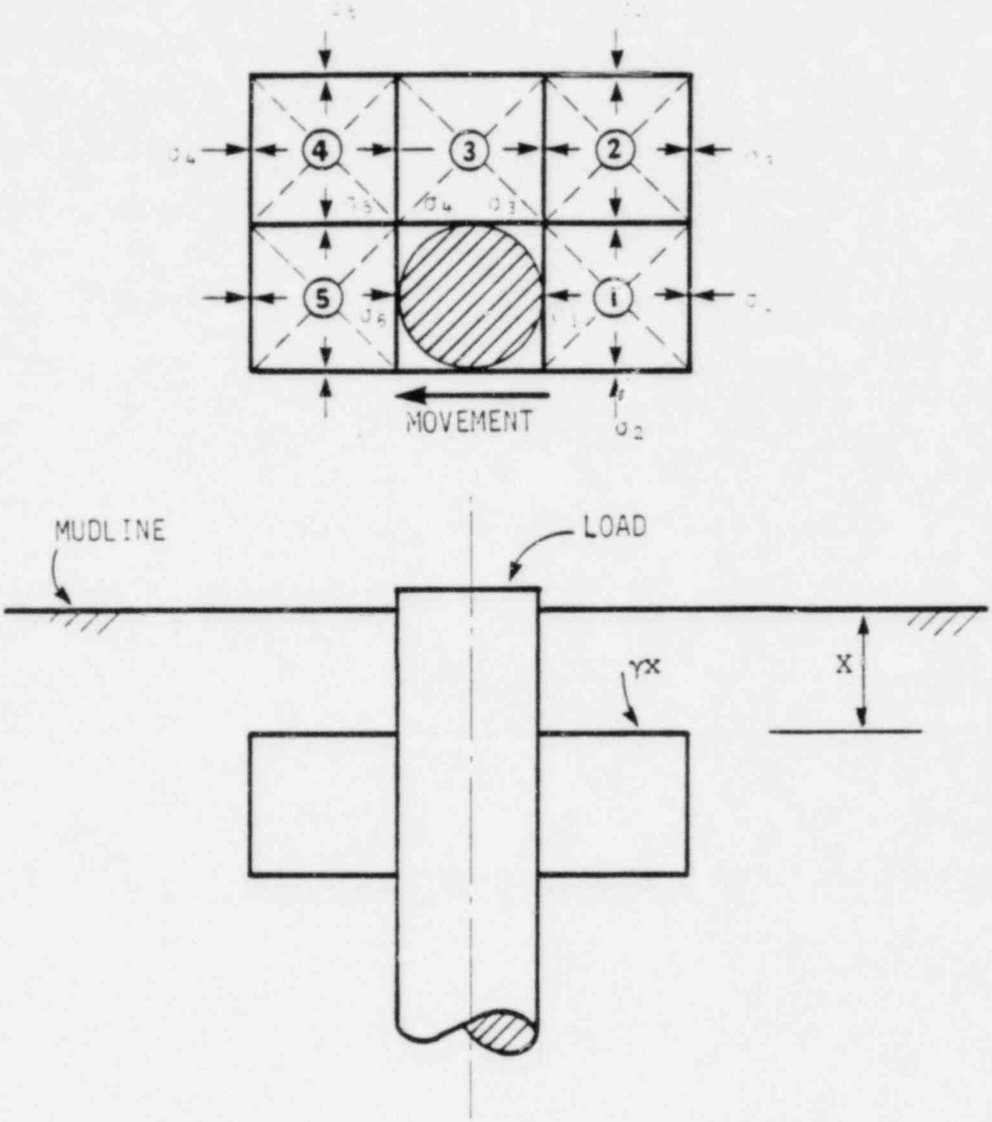


FIG. 2

ASSUMED MODE OF SOIL FAILURE BY LATERAL
FLOW AROUND THE PIPE

The sketch indicates soil flow from the front of the pipe around the side to its back. Only one side of the flow is shown. As indicated in Fig. (2), block (5) develops sufficient stresses to cause failure. The possible failure surfaces are indicated by dotted lines. Stresses are transmitted to blocks 4, 3, 2 and 1, allowing flow around the pipe. Thus pipe movement involves simultaneous failure of blocks 1, 2, 4 and 5 and the sliding of block 3.

Failure stresses are shown on Mohr-Couloumb diagram of Fig. (3).

$$\sigma_1 = K_a \gamma x \tag{15}$$

$$\sigma_2 = \sigma_1 \tan^2\beta = K_a \gamma x \tan^2\beta \tag{16}$$

$$\sigma_3 = \sigma_2 \tan^2\beta = K_a \gamma x \tan^4\beta \tag{17}$$

$$\sigma_4 = \sigma_3 + K_o \gamma x \tan\theta = K_a \gamma x \tan^4\beta + K_o \gamma x \tan\theta \tag{18}$$

$$\sigma_5 = \sigma_4 \tan^2\beta = K_a \gamma x \tan^6\beta + K_o \gamma x \tan^2\beta \tan\theta \tag{19}$$

$$\sigma_6 = \sigma_5 \tan^2\beta = K_a \gamma x \tan^8\beta + K_o \gamma x \tan^4\beta \tan\theta \tag{20}$$

The ultimate resistance per unit length of pipe can be defined, subtracting the active pressure σ_1 acting on the back of the pipe, as

$$q_{u\ell} = K_a\ b\gamma x\ (\tan^8\beta - 1) + K_o b\gamma\ 2\tan^4\beta \tan\theta \tag{21}$$

Since we have two equations for estimating ultimate soil resistance, the "transition depth" x_t has to be calculated.

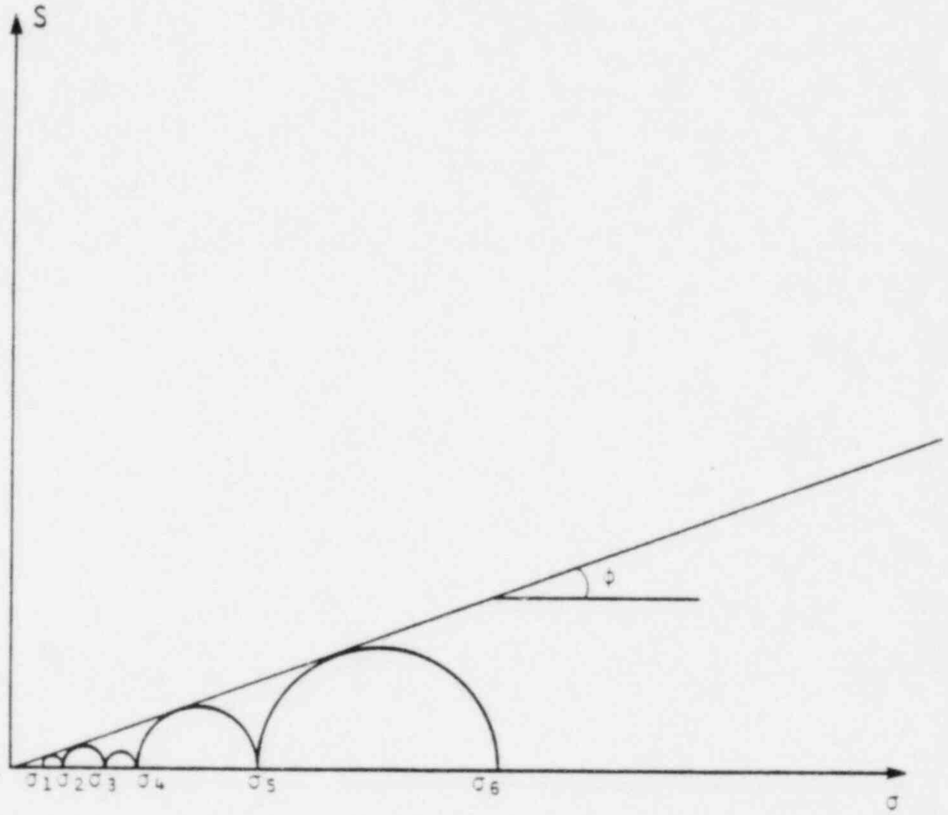


FIG. 3

MOHR-COULOUMB DIAGRAM

FAILURE STRESSES

Equating Eqs. (21) and (14) and solving for x, we get

$$x_t = \frac{b\ (\tan^5\beta + K_o \tan^3\beta \tan\theta - \tan\beta)}{\tan^2\beta \tan x + K\ (\tan\beta - \tan x)} \qquad (22)$$

Thus for $x > x_t$ use Eq. (21), and for $x < x_t$ use Eq. (14). If variations of internal friction angle (θ) and unit weight (γ) exist with depth, a unique analytical solution is not possible. Instead the resistance from both equations should be computed and plotted with depth. The two curves should intersect and the first intersection below the surface should be chosen as x_t. Also, utlimate soil resistance Eqs. (14) and (21) were derived assuming uniform soil conditions. If non-uniform soil conditions are encountered the (γx) terms in these equations should be replaced with values of actual overburden pressure.

3. Estimation of Elastic Modulus

By studying Fig. (4) which shows pressure bulbs of pipes of diameter B and nB at some depth below the surface, one can conclude that

$$E_s = \alpha \frac{1}{B} \qquad (23)$$

For homogenous deposits of cohesionless sand, the soil modulus is assumed to increase linearly with depth

$$E_s = K\ (\frac{X}{D}) \qquad (24)$$

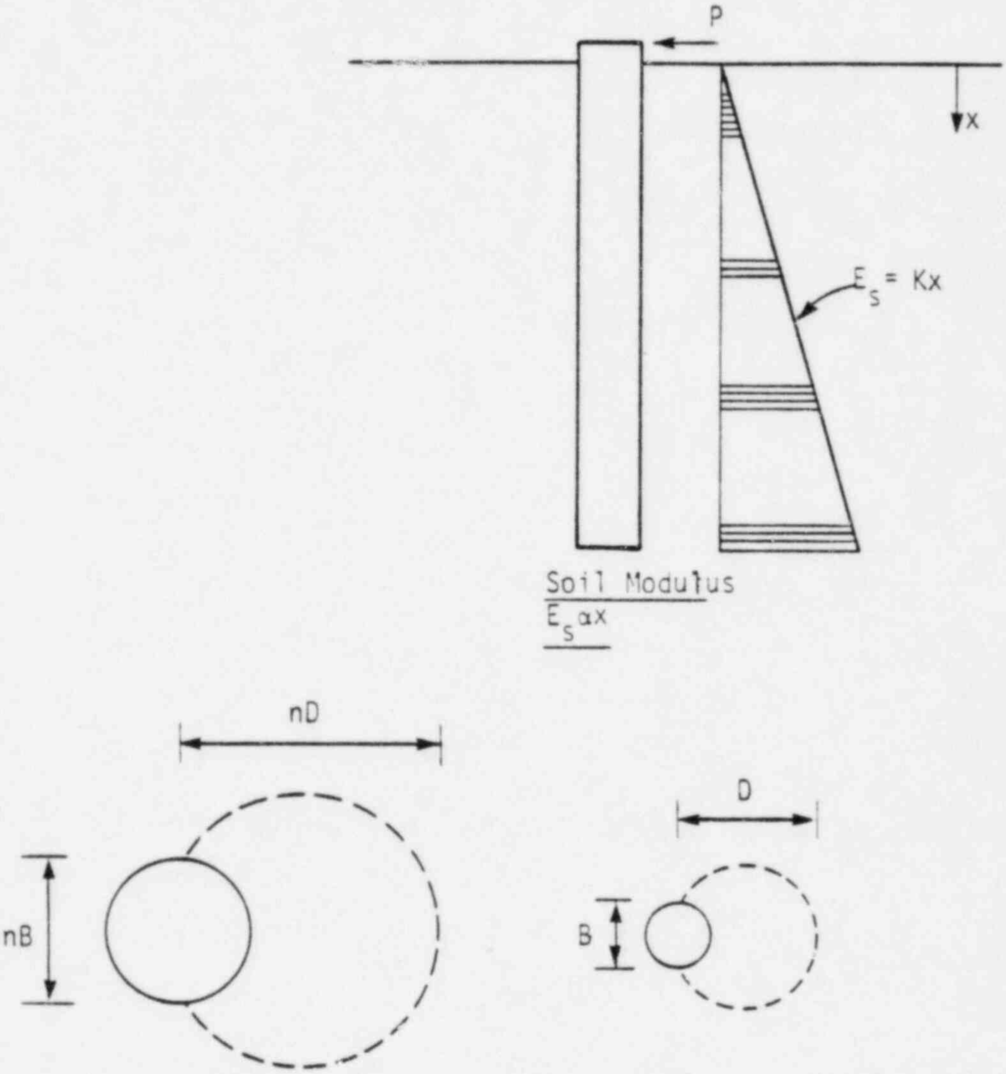


FIG. 4

PRESSURE BULB

D α B

Terzaghi suggests the following values for subgrade constant K.

TABLE 1

Relative Density	Loose	Medium	Dense
Dry or moist sand, T/ft^3	7	21	56
Dry or moist sand, lb/in^3	8	24	65
Submerged sand, T/ft^3	4	14	34
Submerged sand, lb/in^3	5	16	40

Note that E_s values will have units of force/length^3. For example, lb/ {(inch-width) ·(inch-height) ·(inch-deflection) }. Thus to get equivalent spring constant K_s, E_s should be multiplied by pipe diameter times incremental length h along the pipe axis so

$$K_s = E_s \cdot b \cdot h = K \cdot x \cdot h \qquad (25)$$

4. Closure

1. Define the geometry of the site strata and the GWT elevation if any (i.e., the depth of each stratum).

2. Determine the relative density of each stratum.

3. Determine the unit weight of each stratum.

4. Use Table (1) to obtain the corresponding subgrade coefficient K.

5. Decide on appropriate incremental length (h).

6. Use Eq. (25) to determine the spring constant (Ks).

7. Use Eqs. (14) and (21) to determine (X_t), either by plotting or by equating.

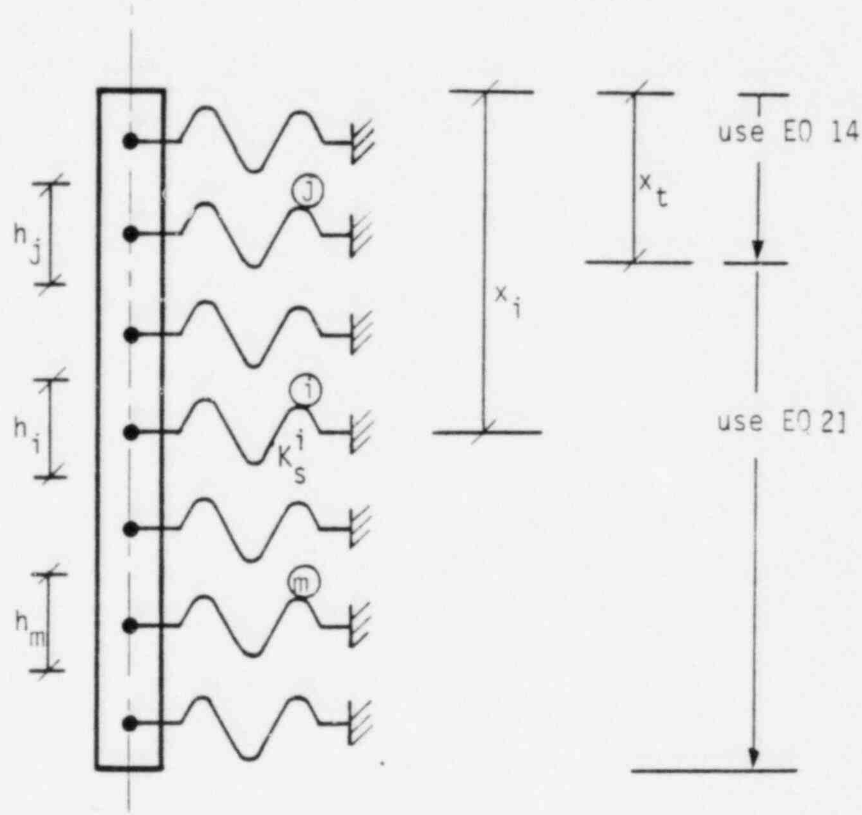
8. Use Eq. (14) to determine ($Q_{u\ell}$) for $0<x<(x_t)$.

9. Use Eq. (21) to determine ($Q_{u\ell}$) for $x>(X_t)$.

10. Place these springs in the finite element model at the station point in the two major horizontal directions (X, Z).

11. Restrain the pipe in the vertical (Y) direction at the lower tip of the pipe. (Refer to Figs. 5 and 6.)

$K_s^i = Kx_i h_i$



Eq. (1A)

$$Q_{u\ell}^{j} = q_{u\ell}^{j} \cdot h_j = h_j \{\gamma x_j [b(K_p - K_a) + K_p x_j (\tan\alpha \tan\beta) + K x_j \tan\beta (\tan\theta - \tan\alpha)]\}$$

Eq. (21)

$$Q_{u\ell}^{m} = q_{u\ell}^{m} \cdot h_m = h_m \{K_a b \gamma x_m (\tan^8\beta - 1) = K_o b \gamma x_m \tan^4\beta \tan\theta\}$$

FIG. 5

FINITE ELEMENT MODEL FOR A PORTION OF A VERTICAL PIPE

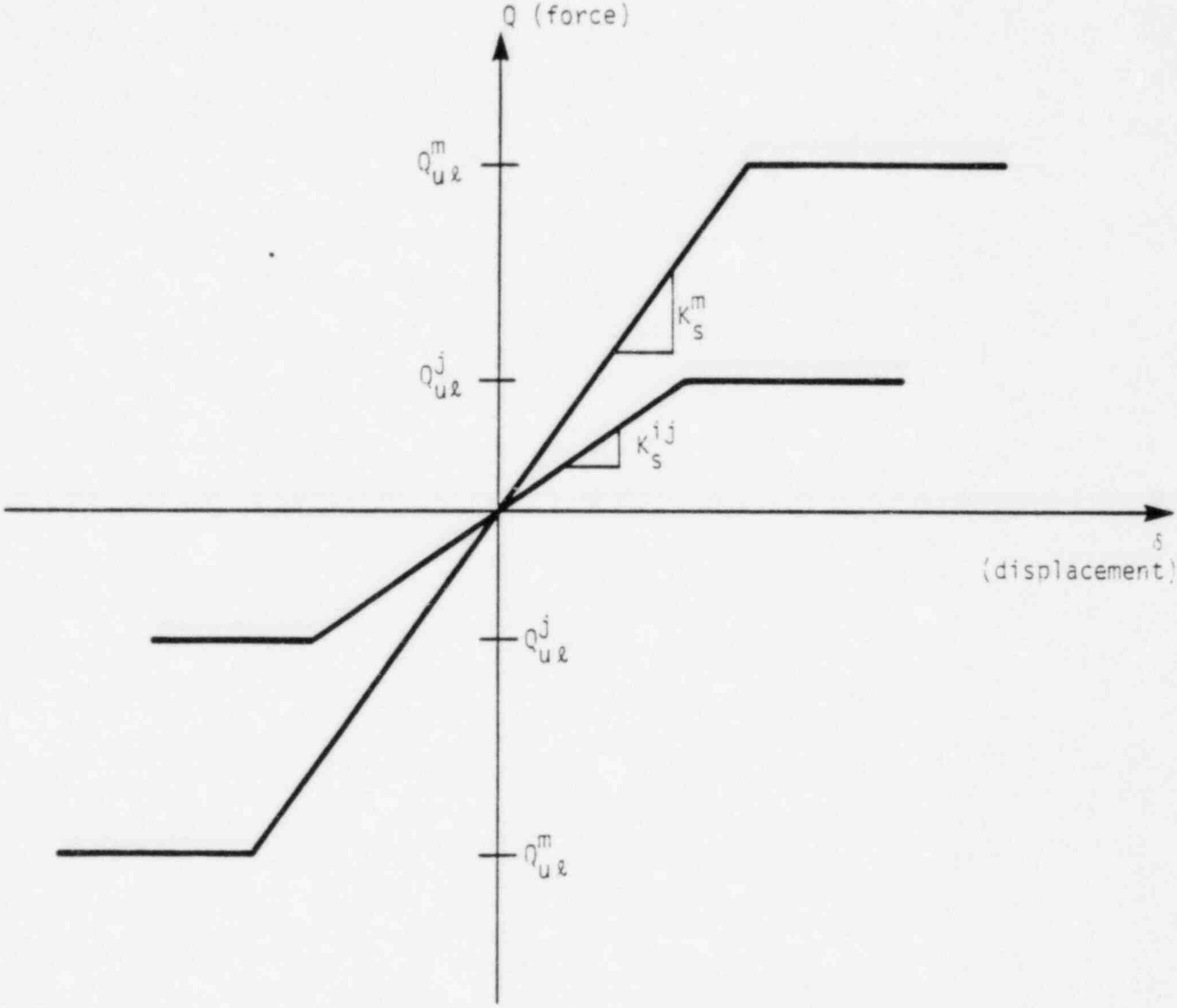


FIG. 6

NONLINEAR FORCE-DISPLACEMENT MODEL

PART II

NON VERTICAL PIPES

1. Horizontal Pipes

<u>Vertical Springs</u>

- Springs constant shall be estimated according to Eq. (25), i.e.,

$$K_S = K \cdot x \cdot h$$

where:

K_s is the required spring constant.
K is the subgrade coefficient (see Table 1).
X is the depth from ground level.
h is the distance between stations.

- Ultimate soil capacity

The ultimate force which any spring can sustain shall be calculated from the equation

$$d_{u\ell} = (h.B) * (.5\gamma B N_\gamma + \gamma x N_q) \qquad (26)$$

where:

$P_{u\ell}$ is the ultimate load (Force).
h is the distance between stations.
B is the pipe diameter.
X is the depth from ground level.
γ is the effective unit weight.
N_γ, N_q are Terzaghi Bearing Capacity Coefficients.

Horizontal Springs

The procedure described in PART I (Vertical Pipe Penetrations) shall be followed to estimate both K_s (spring constant) and $Q_{u\ell}$ (Ultimate Capacity). Refer to Eqs. 14, 21 and 25.

Notice that these horizontal springs are perpendicular to the pipe centerline.

2. Inclined Pipes

Inclined pipes shall be treated as follows:

- The vertical projection shall be analyzed as a segment of vertical pipe [Eqs. (14), (21) and (25)].

- The horizontal projection shall be analyzed as a segment of horizontal pipe [Eqs. (25) and (26)].

3. Closure

1. Determine all the vertical and inclined portions of the pipe.

2. Calculate K_s and $Q_{u\ell}$ as per closure in PART I, page 14.

3. Calculate the vertical stiffness and the ultimate vertical load capacity for the inclined portions of the pipe as per Eq. (25) and (26).

4. Calculate spring stiffness constant and ultimate capacity in both the vertical and horizontal directions for the horizontal portions of the pipe as per PART II.1.

END OF CRITERIA