APPENDIX 3E

CONTAINMENT LINER INSULATION PREOPERATIONAL TESTS

by JOHNS-MANVILLE SALES CORPORATION

BM Containment Insulation SP-5290 Ginna Plant

JOHNS-MANVILLE
SALES CORPORATION
INDUSTRIAL INSULATIONS DIVISION
22 EAST 40th STREET • NEW YORK, N. Y. 10016 • TELEPHONE: 532-7600 • AREA CODE 212

JM

December 22, 1967

Gilbert Associates, Inc.
525 Lancaster Avenue
Reading, Pa. 19603

Attention: Mr. K. T. Momose

Re: BM Containment Insulation
SP-5290 Ginna Plant

Dear Mr. Momose:

On November 29, at your request Mr. E. D. Cox sent to your attention the following reports:

Report E 455-T-258 Vinylcel - Resistance to Flame Exposure

Report E 455-T-266 Vinylcel (4pcf) Effect of Heat and Pressure

Subsequent to this you requested engineering data on the 4 pcf Vinylcel similar to that previously furnished for 6 pcf Vinylcel. This is as follows:

2:07.2 Based on pressure cycling tests of nominal 6 pcf Vinylcel (Report E 455-T-238) and the relative elastic moduli of 6 pcf and 4 pcf Vinylcel, we estimate the maximum deflection of 4 pcf Vinylcel to be 2.8% and the residual deformation to be 0.8%.

3:01.2

a. Thermal conductivity (BTU/hr sq ft/F/in) per ASTM C-518 Heat Flow Meter calibrated per ASTM C-177 Guarded Hot Plate.

Mean Temperature, F.	75	100	125	150
BTU-in	0.22	0.23	0.25	0.27

b. Compressive yield strength per ASTM D1621 - 140 psi at the 0.2 percent point on stress-strain curve.

c. Maximum operating temperature for continuous service - 175F, but may vary with specific application requirements.

d. Maximum allowable temperature for specified time interval. See attached Report No. E455-T-266, Appendix I, Compression Under Combined Heat and Pressure Test.

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e. Moisture vapor permeability per ASTM C-355. See attached Report No. E455-T-268, Appendix I, Table 3.

f. Shear strength per ASTM C-273 - 68 psi ultimate.

g. Shear modulus per ASTM C-273 - 3510 psi.

h. Compressive modulus per ASTM D-1621 - 2300 psi.

i. Density per ASTM D-1622 - 4.0 lbs/cu ft. nominal, 3.7 lbs/cu ft. minimum.

j. Average coefficient of linear expansion - 9.4×10^{-6} in/in/F.

k. Curves for the Case III showing temperature before and after accident plotted against time. See Report No. E 455-T-266, Analogue Study of Vinylcel used as Containment Insulation.

l. Test results of permeability tests per ASTM C-355. See attached Report E 455-T-268,

Predicted curve for 6 month test as required under 2:07.9. See attached Report No. E455-T-268. Dimensional rather than weight change is given as explained under Humid Aging (Results) of the report.

m. Radiation exposure of 8×10^6 roentgens within 6 hours will not change the physical properties of Vinylcel significantly but 10^8 roentgens within 10 hours will cause some progressive deterioration.

The 4 pcf Vinylcel will be supplied 44" x 84" x 1-1/4" thick. Length and width tolerance will be ± 1/32".

Very truly yours,

C. E. ERNST
Chief Engineer

CEE/ca

P.S. As I advised your secretary on Wednesday, Research is sending 6 copies of report E455-T-238 directly to you.

Report No. E455-T-268, VINYLCEL (4 pcf) - Water Vapor Permeability and Humid Aging Tests

JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-268
Date December 20, 1967

Title: VINYLCEL (4 pcf) - Water Vapor Permeability and Humid Aging Tests

SUMMARY

VINYLCEL of 4 pcf nominal density has been tested for water vapor permeability and for dimensional changes under high humidity conditions.

The water vapor permeability of 1-in. thick specimens was 0.06 perm-in. at 3.2 pcf and 0.04 at 4.9 pcf density; both values are very low.

In 6 months at 120F, 100 per cent RH, the volume change was only 1.2 per cent, and length and width changes only 0.3 per cent.

Contents: Summary, Discussion, Results, and Appendixes.

Reported by E. J. Davis
E. J. Davis
Materials Evaluation Section

Sheet 2 of Report No. E455-T-268

DISCUSSION

Test Methods:

Density - ASTM C303

Water Vapor Permeability - ASTM C355 (Desiccant Method)

Humid Aging - Specimens 4 x 4 x 0.65 in. were measured to ±0.001 in. in all dimensions and placed above the water level in a glass vessel containing water. The vessel was closed and placed in an oven (circulating type) which was controlled at 120 ±3F. The specimens were removed periodically and measured.

Results:

Water Vapor Permeability (Table 1) - When tested at 1-in. thickness according to the ASTM C355 desiccant method, VINYLCEL at 3.2 pcf density had a permeability of 0.06, and at 4.9 pcf density of 0.04 perm-in.

Humid Aging (Figure 1) - These tests of VINYLCEL have been conducted for 6 months at 120F, 100 per cent RH. Initially there was a slight expansion (about +0.2 per cent), both linearly and volumetrically. With increasing time, the specimens began to shrink; the shrinkage levelled off at about -0.3 per cent (average, length and width) and -1.2 per cent (volume).

Sheet 3 of Report No. E455-T-268

APPENDIX I

TABLE 1. VINYLCEL, 4 PCF NOMINAL DENSITY, 1-IN. THICK

	Actual Test Density	
	3.2 pcf	4.9 pcf
Water Vapor Permeability, (perm-in.) by: ASTM C355, Desiccant Method	0.06	0.04

Temperature: 90F

Relative Humidity: 52%

Vapor Pressure Difference: 0.73 in. Hg

Test Area: 100 sq in.

Duration of Test: 8 days

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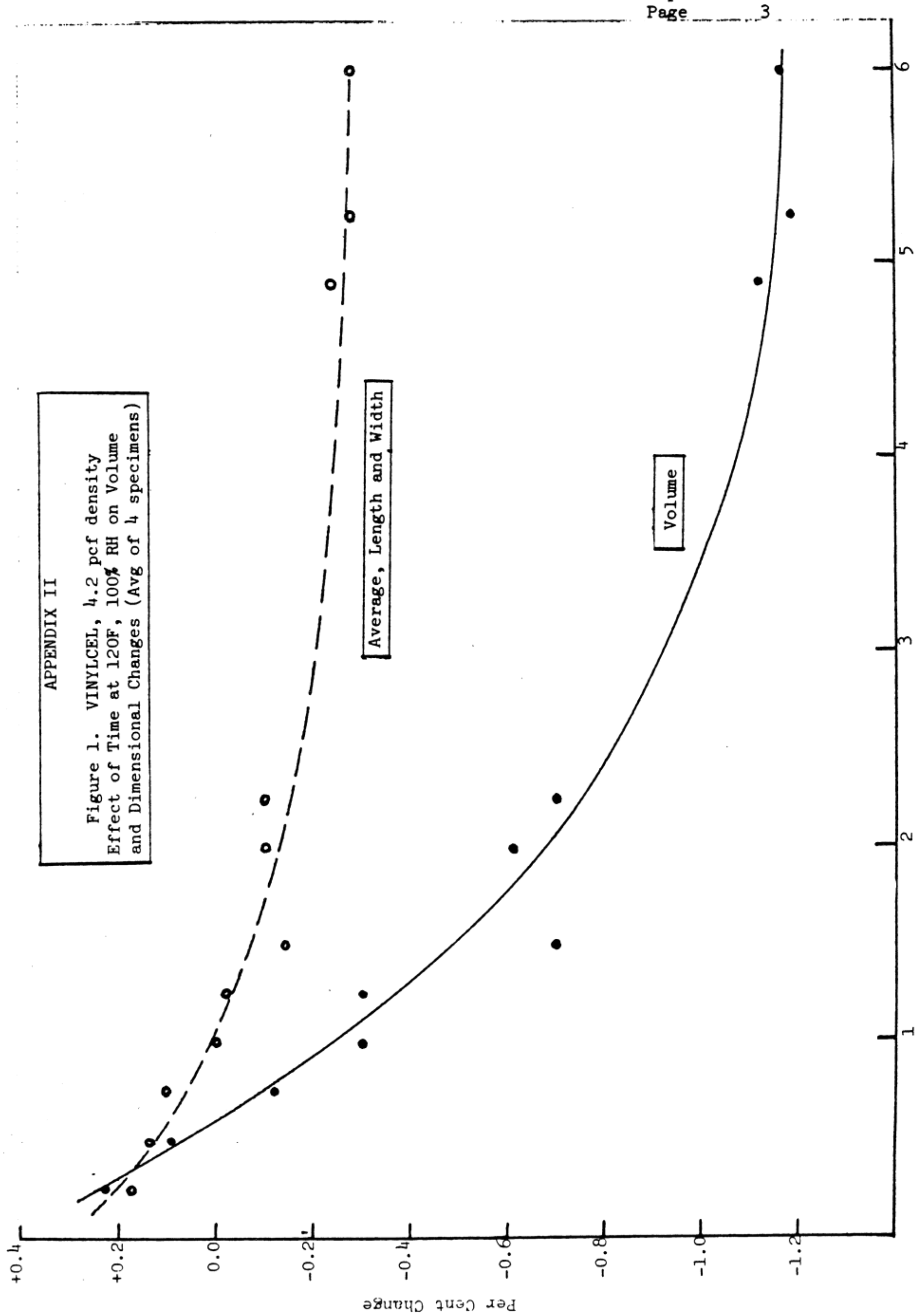

APPENDIX II

Figure 1. VINYLCEL, 4.2 pcf density
Effect of Time at 120F, 100% RH on Volume
and Dimensional Changes (Avg of 4 specimens)

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Report No. E455-T-266, VINYLCEL (4 pcf) - Effect of Heat and Pressure

GINNA/UFSAR

JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-266

Date November 3, 1967

Title: VINYLCEL (4 pcf) - Effect of Heat and Pressure

Requested by: C. E. Ernst

SUMMARY

VINYLCEL of 4 pcf nominal density, 1-1/4-in. thick, has been subjected to a combined heat and compression test to simulate an "incident" in a nuclear reactor containment vessel.

Two tests, according to Gilbert Associates' SP-5920 (Case III), resulted in thickness decreases of 37 per cent and 38 per cent at the critical time of 5-1/2 minutes; the corresponding permanent thickness losses were 29 per cent and 22 per cent.

A network analog simulation of the insulation system under Case III conditions showed that after 5-1/2 minutes the temperature rise of the steel liner was 1F.

Contents: Summary, Discussion, and Appendixes.

Reported by E. J. Davis
E. J. Davis
Materials Evaluation Section

W. F. Gulick
W. F. Gulick
Basic Physics Research Section

rs

Sheet 2 of Report No. E455-T-266

DISCUSSION

Test Method:

A 4 x 4-in. electrically heated hot plate was used with a compression testing machine. The 4 x 4-in. specimen of VINYLCEL, with a thermocouple in a groove on its hot side, was placed on the hot plate; its temperature was read with a direct reading potentiometer. The temperature of the hot plate was raised by means of a variable resistance in series with it. As the specimen was simultaneously heated and loaded by the testing machine, its deflection was measured with a dial gage, accurate to 0.001 in., mounted in a compression rig.

The pressure and temperature conditions of Gilbert Associates Specification SP-5290 (November 30, 1966), Case III, were followed as closely as possible.

Results:

Two tests were run (see Table). In the first, the pressure curve was followed very closely. The temperature lagged by as much as 98F at 30 seconds, but had caught up at 3 minutes; it was then over the desired temperature, by as much as 29F, for the next 7 minutes. At the critical time of maximum pressure (5-1/2 minutes) the specimen had deflected approximately 37 per cent. The maximum deflection was 58.5 per cent at 10 minutes, after which the sample began to regain its thickness and the test was ended. After the test, the permanent thickness loss was about 29 per cent.

A second test was run, because of the high temperatures encountered in the first test. This time the test temperature started a little higher than desired, lagged by as much as 88F at 30 seconds, and reached the desired temperature at 4 minutes; thereafter it remained close to the desired curve.

At 5-1/2 minutes, the specimen had compressed about 38 per cent; the maximum was 49 per cent at 20 minutes, and the permanent loss of thickness about 22 per cent.

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APPENDIX I

VINYLCEL, 1-1/4 in. THICK, 4 pcf NOMINAL DENSITY
Compression Under Combined Heat and Pressure Test
(Gilbert Associates, Case III Conditions)

Time	Desired Conditions Case III		Test A (1.251-in. thick)				Test B (1.248-in. thick)			
	Pressure psi	Temp. °F	Pressure psi	Temp. °F	Sample Deformation in.	Sample Deformation Per Cent	Pressure psi	Temp. °F	Sample Deformation in.	Sample Deformation Per Cent
1 sec.	<20	155	0	156	0	0	1.2	166	0	0
5 sec.	77	205	77	160	0.046	3.7	77	--	0.053	4.2
30 sec.	77	268	77	170	0.057	4.6	77	180	0.065	5.2
1 min.	79	273	79	194	0.103	8.2	79	194	0.118	9.4
2	82	282	82	251	0.191	15.3	82	248	0.193	15.4
3	84	290	84.5	288	0.257	20.6	84	276	0.245	19.6
4	87	295	87	315	0.352	28.2	87	292	0.338	27.0
5	89	302	89	330	0.390	31.2	89	300	0.428	34.2
5-1/2	90 (max)	305	90	334	0.460	36.8	90	304	0.479	38.3
6	89	306	88	332	0.492	39.4	89	306	0.502	40.2
7	87	308	87	330	0.555	44.4	87	312	0.542	43.4
10	82	310 (max)	82	313	0.732	58.5	82	312	0.611	48.9
20	68	305	68	298	0.713	57.0	69	306	0.612	49.0
30	55	298	(ended)				56	297	0.588	47.0
			(after test thickness, 0.890 in.; loss, 0.361 in., or 28.9 per cent)				(ended) (after test thickness, 0.928 in.; loss, 0.280 in., or 22.4 per cent)			

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APPENDIX II

ANALOG STUDY OF VINYLCEL USED AS CONTAINMENT INSULATION

The transient rise in temperature of the insulated cross-section of a nuclear reactor has been measured, based on a temperature profile at the hot surface (hypothetical incident) provided by Gilbert Associates. The measurements were made on an electrical network analog set up to simulate the insulation system.

The cross-section consisted of a VINYLCEL (PVC foam) layer, a steel liner, and a surrounding concrete barrier. The following properties of the layers were assumed:

	VINYLCEL	Steel	Concrete
Thermal conductivity, $\frac{\text{Btu - in.}}{\text{hr - sq ft - F}}$	0.25	312.0	10.0
Specific heat, $\frac{\text{Btu}}{\text{lb - F}}$	0.3	0.106	0.21
Density, lb/ft^3	6.67	480	140
Thickness, in.	0.75*	0.375	42

*Compressed thickness. Uncompressed thickness was 1.25 in.

From these parameters the thermal resistances and capacitances of the system were computed:

	VINYLCEL	Steel	Concrete
Resistance, sec - ft^2 - F/Btu	10,800	4.3	15,100
Capacitance, Btu/ft^2 - F	0.125	1.6	103

Air to surface resistances at the hot and cold surfaces were assumed to be 180 and 900 sec - ft^2 - F/Btu.

Although the hypothetical incident lasts several hours, the temperature rise of the steel liner after 5-1/2 minutes was the information sought. The hot surface temperature profile (Case III) near the beginning of the incident was therefore programmed in detail. The 2-minute running time of the electrical analog was set equal to 16 minutes of thermal time. The VINYLCEL insulation layer was represented by six

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network sections, the steel layer by one section, and the concrete barrier by twenty sections resistively and by the first five sections capacitatively.* The analog was allowed to come into equilibrium with 120F on the hot face and -10F on the cold face.

Curves of temperature vs time for the hot surface of the VINYLCEL insulation, the mid-point of the insulation, and the steel liner are shown in Figure 1.

To obtain a more conservative measure of the temperature rise at the steel liner, an analog run was made in which the hot surface temperature at the start of the incident was raised immediately to 310F (the peak temperature) and held there for 16 (thermal) minutes, the duration of the run. The curves for this experiment are shown in Figure 2.

After 5-1/2 minutes, the temperature rise of the steel liner was 1F (Figure 1) and 1-1/2F (Figure 2).

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*Only enough capacitance was available to fill the first five out of twenty network sections. Tests showed, however, that in 16 minutes the temperature wave had barely penetrated into the concrete.

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FIG. 1

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1.25" VINYLCEL COMPRESSED TO 0.75"

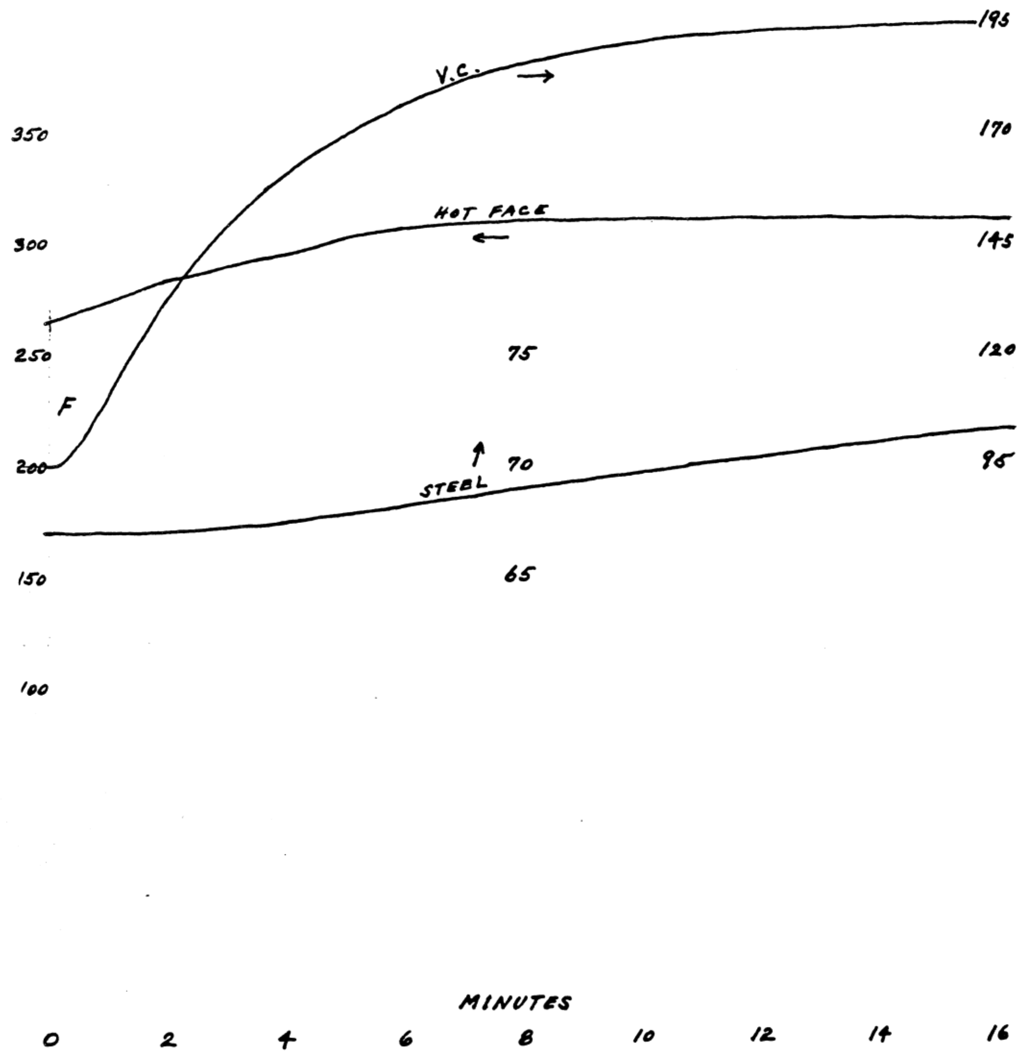

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FIG. 2

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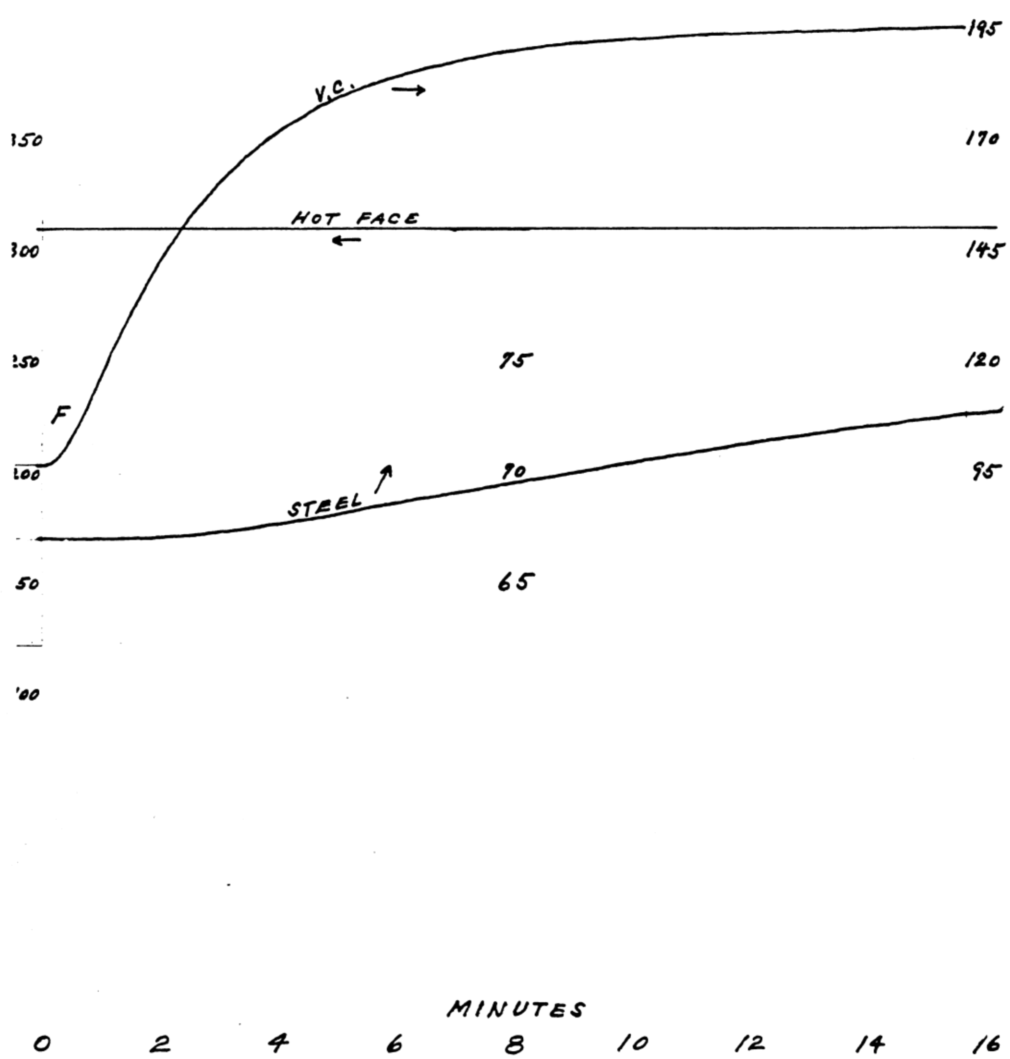
1.25" VINYLCEL COMPRESSED TO 0.75"



Report No. E455-T-258, VINYLCEL - Resistance to Flame Exposure

GINNA/UFSAR

JOHNS-MANVILLE RESEARCH AND ENGINEERING CENTER

Report No. E455-T-258

Date September 21, 1967

Title: VINYLCEL - Resistance to Flame Exposure

SUMMARY

Johns-Manville VINYLCEL foam has been subjected to fire exposure to simulate conditions which might occur during the construction period or during operation when used as insulation for containment shells in nuclear generating plants. The tests were designed and conducted to answer questions raised by the Nuclear Energy Property Insurance Association.

The tests included flame exposures of VINYLCEL, plain, and faced with 24-gage steel, as installed against steel plate; surface burning characteristics of plain and faced sheets as measured by the ASTM E 84-61 Tunnel Test; and the fire resistance characteristics of VINYLCEL faced with 1/2-inch MARINITE 36 when exposed to 30 minutes of the standard time-temperature curve.

Test results obtained on the Bureau of Mines Flame Penetration Test and results of a Thermogravimetric Analysis are also presented.

The test results indicate:

1. VINYLCEL is a product with good flame resistance, low combustibility, and very low surface flame spread.
2. The release of combustible gases is negligible. Weight loss occurs at temperatures in the ranges of 460°F to 572°F and from 572°F to 1112°F. Weight loss of 8 per cent is recorded at 460°F and increases to 38 per cent at 572°F while 1112°F is required to reach a weight loss of 95 per cent.
3. Facing VINYLCEL with 24-gage steel provides improvement in flame resistance. That protection, or facing with 1/2-inch MARINITE 36, greatly reduced heat flow through the construction.

These tests demonstrate that VINYLCEL will offer significant protection against fire exposure. It will not propagate fire nor suffer damage beyond the area of exposure. Flammable gases will not be emitted nor are the gases emitted an explosion hazard.

Contents: Summary, Discussion, Appendix I (Tables 1 & 2), Appendix II (Figures 1-3), and Appendix III (Photographs).

Reported by E. J. Davis
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Materials Evaluation Section

K N Smith
K. N. Smith
Materials Evaluation Section

R H Neisel
R. H. Neisel, P.E.
Materials Evaluation Section
N. J. License No. 9316

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DISCUSSION

Material

VINYLCEL of nominal 4-pcf density, 1-1/4 and 3/4-in. thick, was supplied by the VINYLCEL Production Department on September 6, 1967, and September 11, 1967. Other materials were taken from laboratory stocks.

Witnesses

The Building Fire tests, Tunnel tests, and the Vertical Panel Fire test were witnessed by the following:

R. M. L. Russell	-	Factory Insurance Association
P. H. Dobson	-	Factory Mutual Engineering Corporation
R. R. Koprowski	-	Rochester Gas & Electric Company
R. S. Brown	-	Ebasco Services, Incorporated
L. F. Picone	-	Westinghouse Electric
P. Mitchell	-	Westinghouse Electric
K. T. Momose	-	Gilbert Associates, Incorporated

Test Methods

Building Fire Tests: A concrete block building, 16 x 8 x 8 ft high, was used. (See Photograph A). To its back wall (8 x 8 ft) were bolted two 4 x 8-ft x 3/8-in. steel plates, long dimensions vertical. The joint was sealed with JM No. 450 Insulating Cement. On the exposed surface of these plates were welded 1/8-in. x 2-in. long securement pins, 24-in. on center. Sheets of 4-pcf VINYLCEL, 1-1/4-in. thick, were impaled on the pins; a full sheet (84 x 42-in.) at the top center with long dimension vertical, and cut sheets at the sides and bottom. For Test "A" the VINYLCEL was covered with 24-gage, 4 x 8-ft galvanized steel sheets, also impaled on the pins, with the long dimension horizontal and a 1-in. overlap between the two sheets. For Test "B" the VINYLCEL was left uncovered. In both cases, speed clips were placed over the pins to retain the sheets.

The ceiling was covered with 1-1/4-in. thick VINYLCEL, screwed to the 1/2-in. MARINITE overhead. Poultry wire was secured under the VINYLCEL. The same ceiling insulation was used for both tests. A draft shield of FLEXBOARD, 2-ft deep extended down form the ceiling across the 8-ft width of the room, 8 ft away from the test wall.

Nine chromel-alumel thermocouples were used in each test. Four were between the VINYLCEL and the 3/8-in. steel wall plate, at levels 1, 3, 5, and 7 ft from the floor, and alternated about 9 in. from the vertical center line of the wall. Four were on the exposed surface, at the center line, and at the same levels. One was 3 in. below the ceiling, directly above the center of the fire source. All couples were connected to a switch and a direct reading potentiometer.

The fire source was a steel bucket, 11 in. in diameter and 13 in. deep (area 0.66 sq ft) placed in a hole 6 in. deep in the floor and 2 in. from the center of the test wall. A 6-in. depth of water was in the bottom of [illegible], and a sufficient quantity of heptane was floated on the water to ensure that the fire would last at

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least 30 minutes. The surface of the heptane was about 6 in. above floor level. The peak burning rate of the heptane is estimated as greater than 10,000 Btu per minute.

The heptane was ignited, and readings of the nine thermocouples were taken as rapidly as possible. Notes were made as to flaming, smoking, cracking, or bowing of the test specimens. In addition, during Test "B" of the plain VINYLCEL, a propane torch was used in attempts to ignite the gases collected behind the draft shield.

After the tests, the condition of the VINYLCEL was noted, and the weight losses of the center sheets of VINYLCEL were calculated.

Vertical Panel Fire Test: The composite panel (4 x 8 ft) was assembled as follows: 1/2-in. thick MARINITE 36, 3/4-in. (0.71 in.), 4-pcf VINYLCEL and 3/8-in. steel plate. (The MARINITE was exposed to the fire). The panel was held in place in the furnace buck with MARINITE strips on the furnace side and with bolts and steel fixtures supporting the steel plate on the cold face.

Six thermocouples were placed as follows: two between the MARINITE and VINYLCEL, two between the VINYLCEL and steel plate, and two under asbestos felt pads on the room side of the steel plate. They were placed on the vertical center line of the composite panel, 3 ft and 5 ft from the top. One 7 x 3-1/2-ft panel of VINYLCEL and 4 x 1-ft and 7 x 1/2-ft filler pieces were used.

The test was continued for 30 minutes with the furnace temperature controlled to coincide with the standard time-temperature curve as given in ASTM E 119.

Tunnel Fire Test(s): Two tunnel tests were conducted as prescribed by ASTM E 84. In the first test, the 1-1/4-in. thick, 4-pound per cubic foot VINYLCEL in 7-ft lengths was placed in the tunnel directly exposed to the flame. For the second test, the VINYLCEL was laid on 24-gage galvanized sheet metal (the flame impinged on the sheet metal).

Bureau of Mines Flame Penetration Test: This test uses 6-in. square x 1-in. thick specimens of foam and a propane torch with a pencil-flame burner head with its brass fuel orifice replaced by a steel fuel orifice from a blowtorch head.

The specimen is backed by an 8-in. square x 1/4-in. thick piece of TRANSITE with a 1-1/2-in. diameter hole in its center. A piece of filter paper is placed between the specimen and the TRANSITE.

The burner is first adjusted to produce a temperature of 1910°F to 1960°F, at 2 in. beyond the burner head, and a 3-1/4-in. visible flame. The burner is then placed 1 in. from the vertical assembly of specimen, filter paper, and TRANSITE. The foam specimen is adjudged adequate in resistance to flame penetration if during a 10-minute test flame does not "penetrate the foam or ignite the filter paper." Charring or discoloration of the paper is disregarded.

This test was performed with and without 24-gage galvanized steel over the hot side of the VINYLCEL.

Thermogravimetric Analysis (TGA): The material was analyzed by standard TGA procedure. The data were obtained from ignition in a thermobalance, using a heating rate of 8°C (14.4°F) per minute. The air flow in the combustion chamber was adjusted to 0.5 liters per minute. (From Report No. 455-T-142).

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Results

I. Building Fire Tests:

Test "A", with 24-gage galvanized steel over the 4-pcf VINYLCEL, is covered in Table 1, Figure 1, and Photographs A through F. This test produced little external evidence of damage; the galvanized sheets were buckled and blackened where the flames hit them, but stayed intact and tight of joint. There was some smoking of the insulation from under the galvanized sheets (for about 25 minutes) but most of the smoke was from the heptane fuel. Relatively low temperatures (960°F maximum) were developed on the fire side, and the maximum on the 3/8-in. steel was 378°F.

When the galvanized sheets were removed, a charred or scorched area on the VINYLCEL was found, measuring 54 in. high and 27 in. wide. Within this area were several large shrinkage cracks, to a maximum width of 6 to 7 in., which extended through to the steel plate. Damage was confined to the area of flame impingement, and there was little or no spread either vertically or horizontally; the side sheets were virtually untouched. Weight loss of the central sheet was calculated as 6 per cent.

Test "B", with no covering over the VINYLCEL, (Table 2) produced higher temperatures; 1280°F maximum on the fire side, 660°F maximum on the 3/8-in. steel plate. The VINYLCEL flamed (at times) over an area of 2-1/2 x 7 ft, but mostly at the joints between sheets or at the shrinkage cracks. There was smoking from the VINYLCEL, but again most of the smoke was from the heptane fuel. The gases at ceiling level were tested with a torch at intervals and did not ignite.

After the test, the charred or scorched area was 64 in. high x 32 in. wide. Shrinkage cracks within this area had a maximum width of 8 to 10 in. and extended to the steel plate. There was little or no damage outside the area mentioned, and the side sheets were in good condition. Weight loss of the central sheet was calculated at 16 per cent.

II. Vertical Panel Fire Test:

The temperatures recorded at the various locations are graphically given in Appendix II, Figure 2. The large temperature drop through the VINYLCEL should be noted. After 30 minutes' exposure, this drop was 500°F indicating the high degree of retention of insulating value of the VINYLCEL under this severe exposure.

The MARINITE sheet had cracked horizontally near midheight and vertically from the bottom to the horizontal crack at the center line. This occurred after 23 minutes exposure to the fire. Some barely combustible gas (only flickers of flame when exposed to a torch) was emitted during the middle 10 minutes of the test.

Examination of the panel after the test showed that the VINYLCEL had shrunk and was broken into several pieces but remained in place.

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III. Tunnel Fire Test(s):

The following results were obtained in the tunnel tests:

	Flame Spread	Fuel Contributed	Smoke Developed
Plain VINYLCEL	15.4	5	253
VINYLCEL - Sheet Metal	5.1	0	34

The flame spread results were for a 3-ft flame advance on the plain VINYLCEL and 1 ft for the sheet metal-faced VINYLCEL. Both results indicated that the insulation may be classified as non-combustible as they are less than 25.

Appendix III presents photographs M, N, and O of the VINYLCEL after the tunnel tests. The improvement due to the sheet metal facing is readily apparent. The extent of the physical degradation of the VINYLCEL can be seen (each piece is 7 ft long and 21 in. wide). The VINYLCEL was severely distressed only where the flame actually impinged on it.

IV. Bureau of Mines Flame Penetration Test:

(A). 4-pcf VINYLCEL, with no protection: Two tests were run. The flames penetrated the 1-in. thick specimens and ignited the paper in 40 and 45 seconds, respectively. The penetration appeared to be more because of heat shrinkage than by burning.

(B). Two tests were also run with 24-gage galvanized steel over the VINYLCEL; both were successful, with no burn-through or ignition of the paper in 10 minutes. The paper showed slight discoloration in one test, none in the other. Behind the steel, there was a saucer-shaped depression in the VINYLCEL about 6 in. in diameter by 1/2 in. deep.

V. Thermogravimetric Analysis (TGA):

The VINYLCEL began to lose weight at 140°C (284°F). When 300°C (572°F) was reached, 38 per cent of the weight had been lost. In the second stage of decomposition, between 300 and 600°C (572 and 1112°F), the specimen lost a total of 94.5 per cent of its weight. A curve of weight loss versus temperature is attached (Figure 3).

A comparison of that curve with the TGA curves for other cellular, low density polyvinyl chloride materials presented in Figure 6 of the report "Thermal Decomposition Products and Burning Characteristics of Some Synthetic Low Density Cellular Materials" by Watson, Stark, et al - Bureau of Mines Investigation No. 4777, shows significant weight loss to occur at a temperature 72°F lower than that for VINYLCEL.

This difference is believed due to the cross-linked structure of VINYLCEL. The Bureau of Mines data showed that hydrogen chloride gas was released at 374°F and the similarity of the curves indicates that the same product is released from VINYLCEL but at a significantly higher temperature (446°F).

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APPENDIX I

Table 1

Building Fire Test of VINYLCEL (Test A)
(8-in. Concrete Block Wall, 3/8-in. Steel Plate, 1-1/4-in. VINYLCEL 4-pcf, 24-ga. Galvanized Steel Sheet)

	Temperatures, °F									
	On 3/8" Steel, Bottom to Top				Galvanized, Bottom to Top				Ceiling	
Time	1	2	3	4	5	6	7	8	9	Notes
0	57	58	58	60	61	63	64	63	63	Fuel Ignited
30 Sec	--	--	--	--	220	144	125	140	207	Flames, 3-5 ft; smoke 5-8 ft
1 Min	--	--	--	--	240	300	210	210	264	
1-1/2	--	--	--	--	320	470	270	230	280	
2	--	--	--	--	344	500	290	240	280	Flames 4 ft, jumping to 6 ft
2-1/2	--	--	--	--	374	600	292	247	308	
3	57	59	59	60	424	660	380	290	300	Galvanized buckled toward VINYLCEL, about 3 ft in diameter
4	57	59	60	60	440	700	372	300	300	
5	57	59	59	60	490	940	460	330	320	Smoke, from under galvanized
6	57	58	59	59	485	840	460	320	310	
7	62	59	59	59	540	900	460	340	330	
8	62	59	60	59	480	800	460	320	330	Buckling deeper, but pins still holding
9	62	61	62	59	490	820	440	320	310	
10	62	67	62	59	520	960	490	360	330	
11	60	68	64	59	560	880	530	360	320	
12	67	--	64	60	540	900	480	350	310	
13	64	--	66	61	525	930	520	362	326	
14	74	--	67	61	520	640	480	350	330	Joint in galvanized still tight
15	87	--	68	62	530	700	410	330	290	
16	79	--	68	61	524	820	430	320	300	
17	78	--	69	63	560	650	400	330	320	
18	83	110	69	62	530	720	420	320	320	
19	88	200	70	63	520	630	430	320	320	Smoking (less) from under galvanized;flames 5-6 ft
20	98	270	70	64	510	720	400	320	300	
21	106	310	70	64	490	620	375	300	295	
22	110	330	70	64	520	800	400	320	310	
23	114	360	72	65	500	710	390	310	285	
24	117	370	70	65	520	630	370	310	300	
25	120	378	70	65	510	580	380	310	290	
26	120	370	72	65	520	760	400	320	300	No smoke from under galvanized;(can be touched 2 ft from fire)
27	122	364	70	65	580	810	410	310	300	
28	120	360	70	65	520	740	390	300	290	
29	122	360	72	66	520	610	300	300	295	
30	125	370	72	66	480	630	360	290	280	
31	127	370	72	66	580	620	350	290	280	
32	128	370	72	67	470	600	345	282	270	
33	128	370	74	68	530	680	375	310	280	Fire extinguished

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APPENDIX I

Table 2

Building Fire Test of VINYLCEL (Test B)

(8 in. Concrete Block Wall, 3/8-in. Steel Plate, 1-1/4-in. VINYLCEL 4-pcf)

Time	Temperatures, °F: On Steel, Bottom to Top 1	2	3	4	VINYLCEL, Bottom to Top 5	6	7	8	Ceiling 9	Notes
0	67	72	68	68	75	76	76	76	82	Fuel ignited, flames 5 ft
30 Sec	--	--	--	--	340	390	290	200	240	
1 Min	--	--	--	--	370	570	310	240	300	VINYLCEL flaming over area 2-1/2 x 7 ft
1-1/2	--	--	--	--	580	1200	690	460	430	Smoke Level 4 ft
2	--	--	--	--	740	1210	565	420	410	
3	--	--	--	--	700	1060	560	405	425	Flaming 8 ft high;less smoke
4	--	--	--	--	700	750	550	420	420	Flaming at cracks;gases not ignitible by torch
5	--	--	--	--	840	835	640	455	420	
6	95	660	78	106	800	1280	710	440	420	Flames 6 ft high
7	124	384	82	116	740	1060	640	420	400	
8	--	--	--	--	800	1270	780	432	395	
9	166	285	88	132	900	1145	540	445	415	Gases not ignitible
10	195	325	90	126	840	900	515	425	430	Wind caused ignition of fresh area (at cracks) to 5 ft
11	190	440	92	125	820	620	440	400	370	4-5 in. opening between sheets
12	190	430	95	125	580	520	370	355	370	VINYLCEL flames out
13	190	560	100	150	480	780	380	390	380	VINYLCEL burning again, new location
14	200	450	100	150	440	590	360	350	360	Flames 6 ft high
15	202	420	100	170	470	820	370	380	350	
16	210	400	100	160	440	780	380	360	360	
17	210	390	100	150	560	660	390	350	350	
19	--	--	--	--	--	--	--	--	--	TC in flames indicated from 1200°F to 1700°F max.
20	242	310	100	165	480	920	390	360	360	
21	250	380	110	160	420	700	360	350	310	No flames from VINYLCEL
22	252	420	110	170	510	970	360	350	320	Heptane flames 4-5 ft smoke diminished.
23	265	310	110	170	430	930	370	340	320	
24	272	360	115	170	430	780	350	340	320	
25	268	360	120	170	450	620	330	320	310	
26	275	320	120	170	390	740	310	310	290	
27	290	315	120	170	390	590	290	290	290	
28	285	360	120	170	380	620	280	290	290	
29	285	340	120	160	370	400	280	280	280	
30	285	330	120	170	400	480	250	270	270	
31	292	320	120	170	400	500	300	280	280	
32	290	370	125	165	365	410	280	270	280	
33	310	295	120	170	380	500	270	275	280	Fire extinguished

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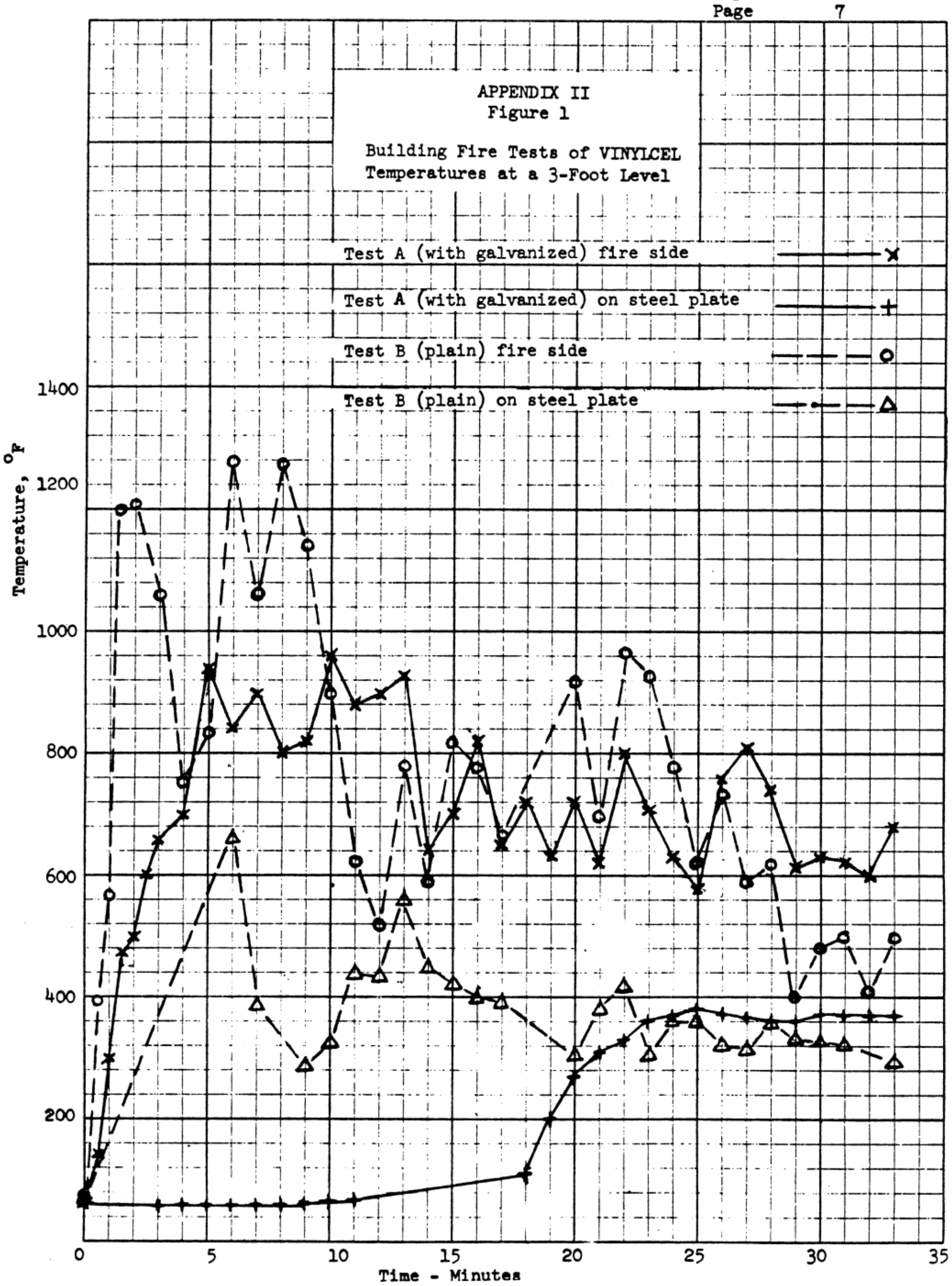

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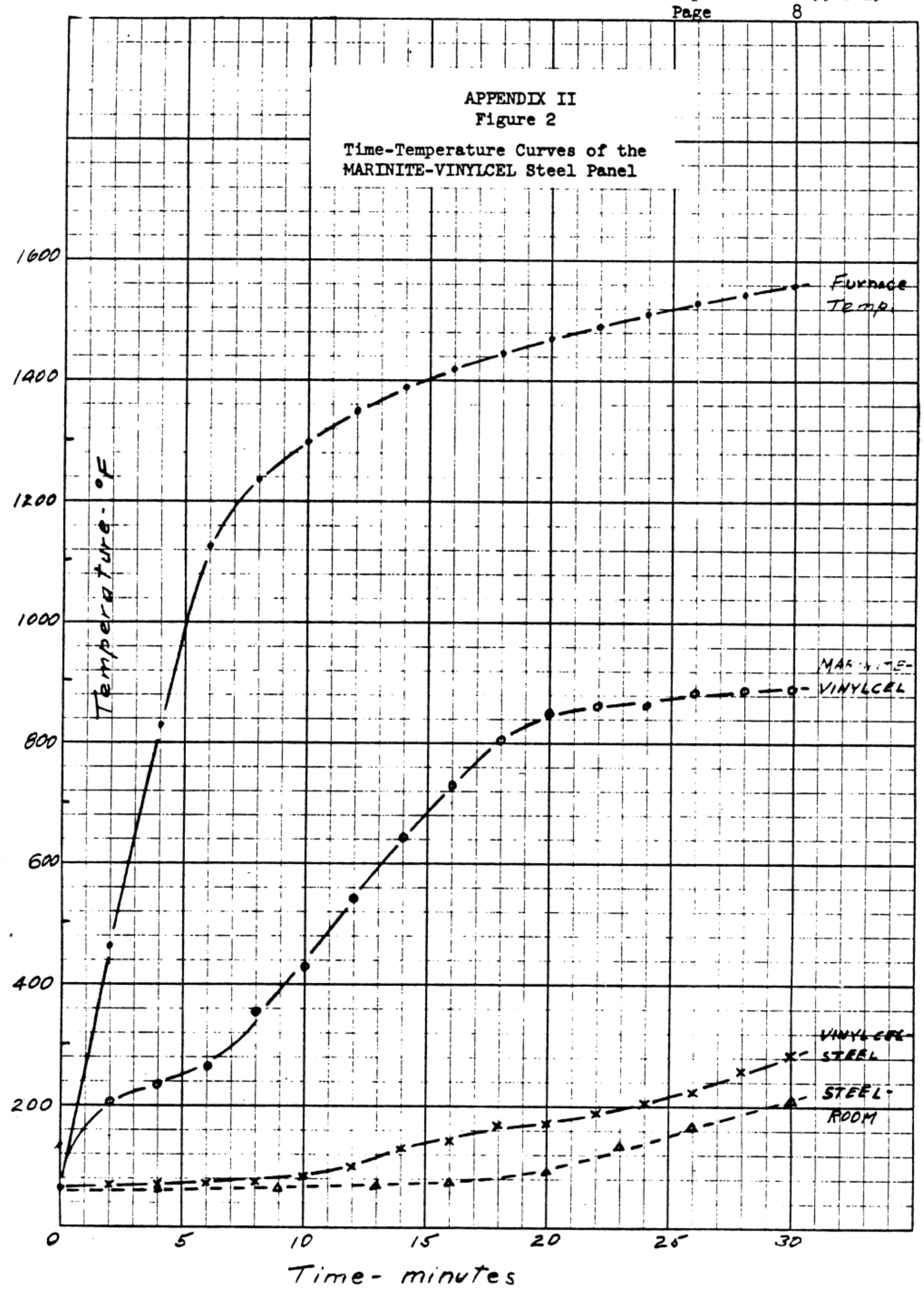

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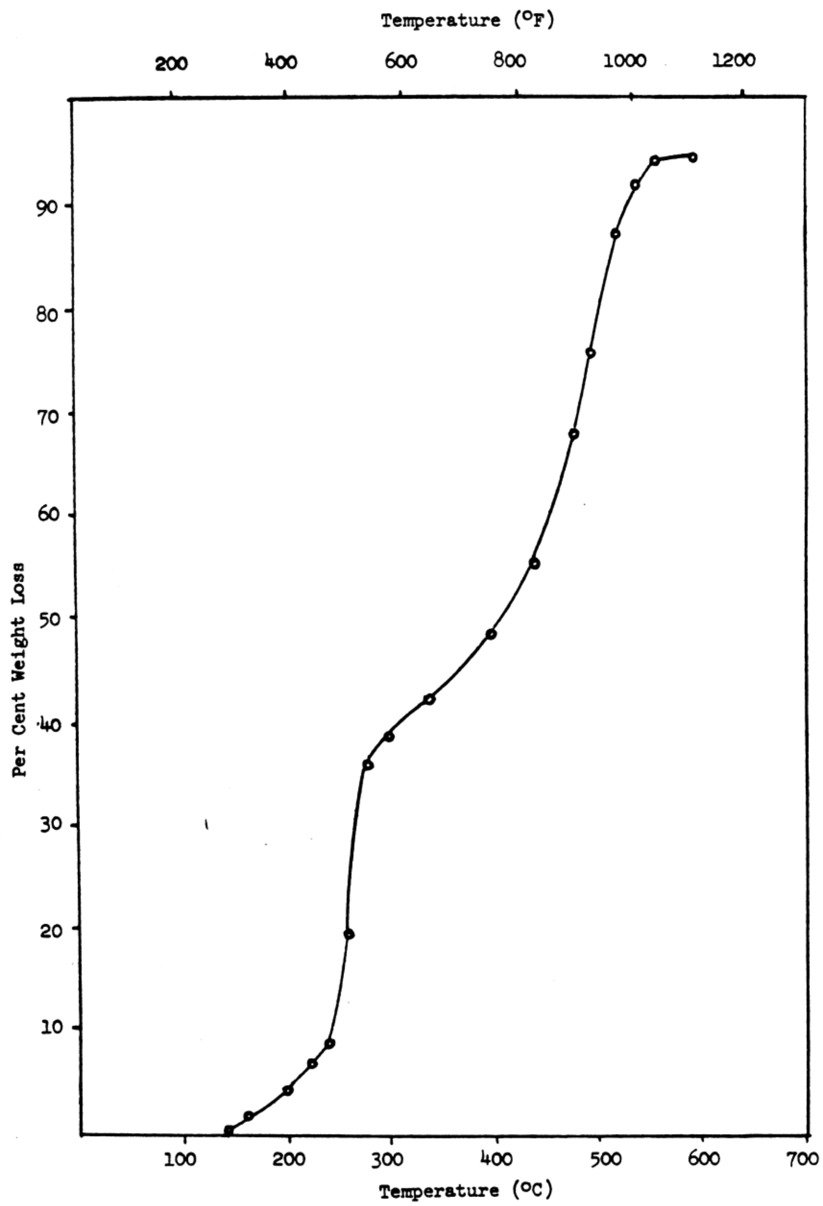
APPENDIX II
Figure 3
TGA Curve of VINYLCEL



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BEST IMAGE AVAILABLE

APPENDIX III

Photograph A
Fire Test Building

The test area is 16 x 8 x 8 ft with a 5 x 5-ft attached vestibule. The vestibule door was left open to admit air. A suspended ceiling of MARINITE sealed the space between the two top lines of vents. The first vent from the left on the second line from the top was left open for smoke venting. The first and second vents from the left on the bottom line were left open for air. Similar vents are on the other side of the building.

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APPENDIX III

Photograph B

View Through Vestibule During Test

This was taken during the test with steel facing. It is not believed the total height of flame appears in the photograph.

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BEST IMAGE AVAILABLE

APPENDIX III

Photograph C
View of Test A (Steel Facing)
Prior to Light Off

Note speed-clip fasteners at 1, 3, 5, and 7 ft elevation, spaced 2 ft on centers across width. Thermocouple wires are shown attached to the surface and the ceiling. Note also the VINYLCEL supported against the ceiling with chicken mesh and clips. The edge of the asbestos-cement draft shield 8 ft from the test face may also be seen at the top.

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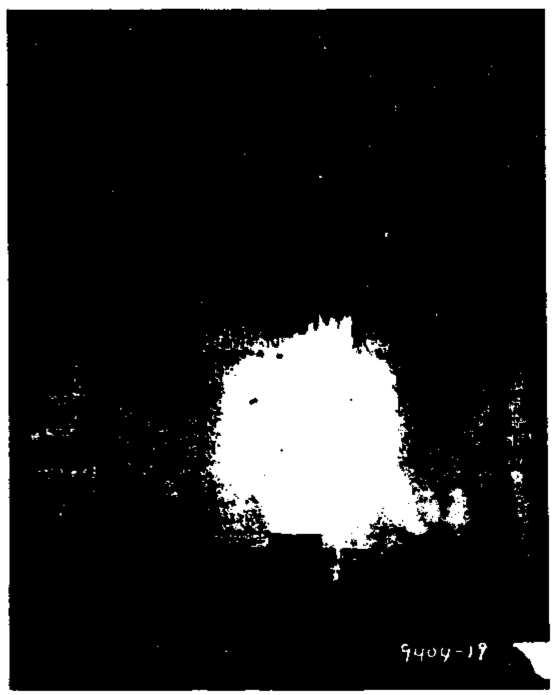
BEST IMAGE AVAILABLE

APPENDIX III

Photograph D

Exposure Flame 50 Seconds After Light Off on Test A



The visible flame height is shown at 5 ft from the lower edge of the material at this time. Table 1 notes flame heights of 3-5 ft.

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BEST IMAGE AVAILABLE

APPENDIX III

Photograph E

View of Test A After Fire Was Extinguished

Only smoke deposit and burn-off of zinc noted.

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BEST IMAGE AVAILABLE

APPENDIX III

Photograph F

Condition of VINYLCEL After Removal of Steel

The center sheet measured 7 x 3-1/2 ft and the horizontal line at 4 ft shows the position of the joint in the steel facing.

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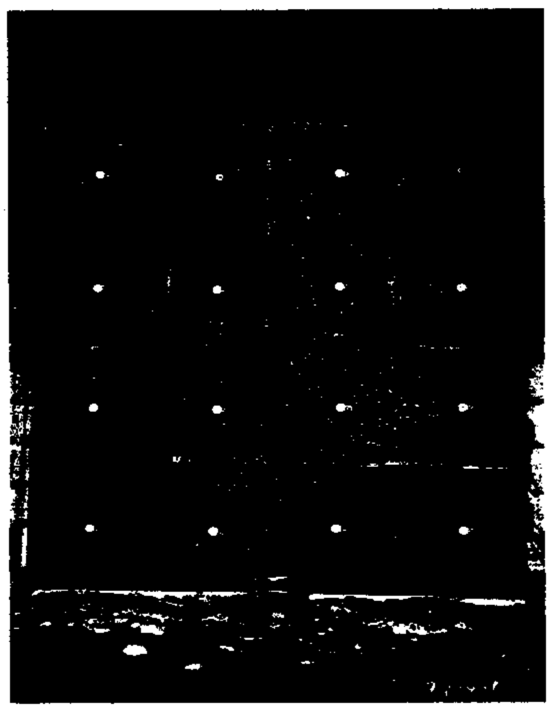
APPENDIX III

Photograph G

View of Uncovered Sample for Test B



Erection was similar to that of Test A.

Note speed-clip fasteners at 1, 3, 5, and 7 ft elevation, spaced 2 ft on centers across width. Thermocouple wires are shown attached to the surface and the ceiling. Note also the VINYLCEL supported against the ceiling with chicken mesh and clips. The edge of the asbestos-cement draft shield 8 ft from the test face may also be seen at the top.

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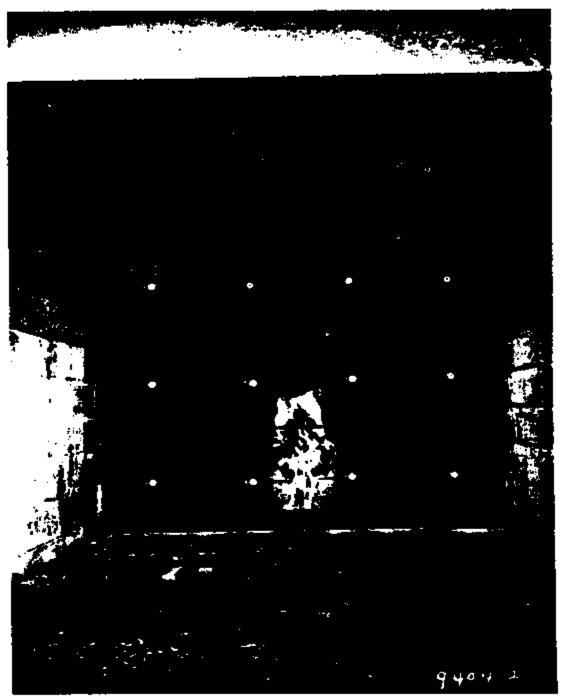
APPENDIX III

Photograph H

View of Test B 30 Seconds After Light Off



Flames 5 ft high were noted, but were not visible on the print.

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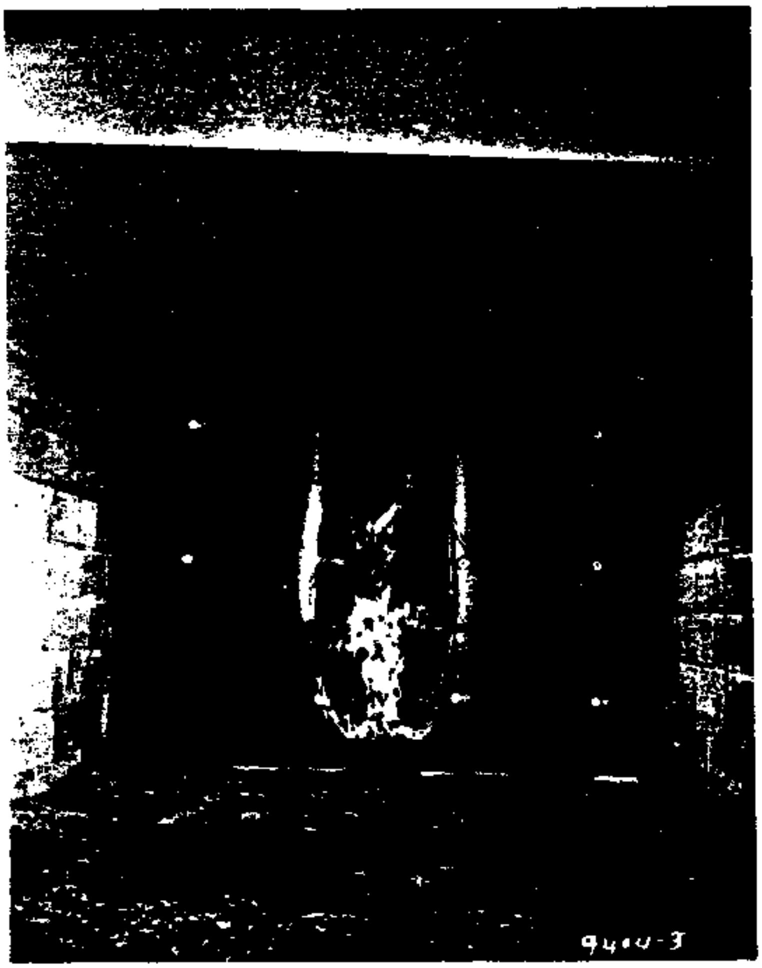
Photograph J

View of Test B During Exposure



Note flames to 5 ft and cracks in material.

BEST IMAGE AVAILABLE

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Photograph K

Bare VINYLCEL of Test B After Exposure

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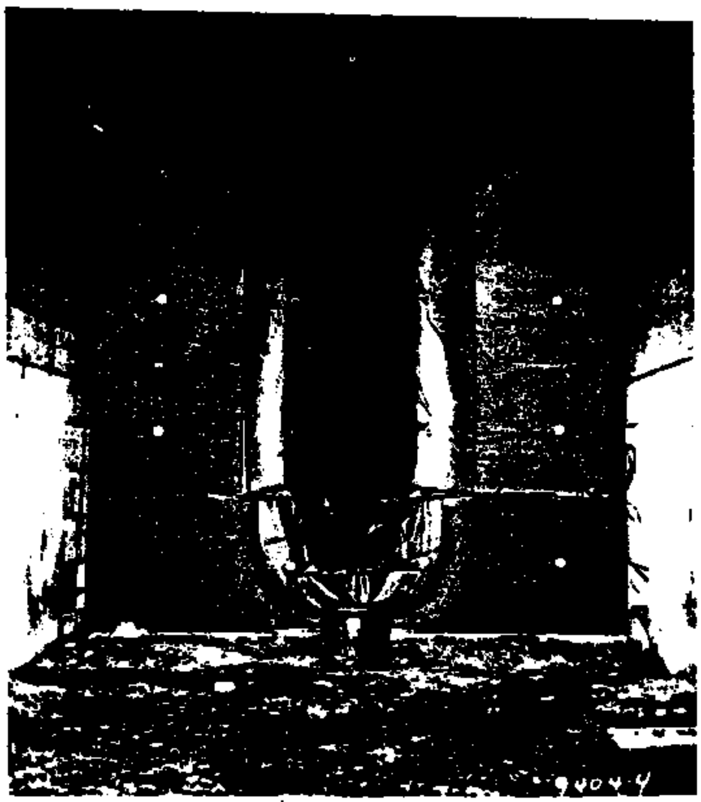

The charred area was 64 x 32 in. Width of through cracks was 8 to 10 in. Little lateral spread of flame noted.

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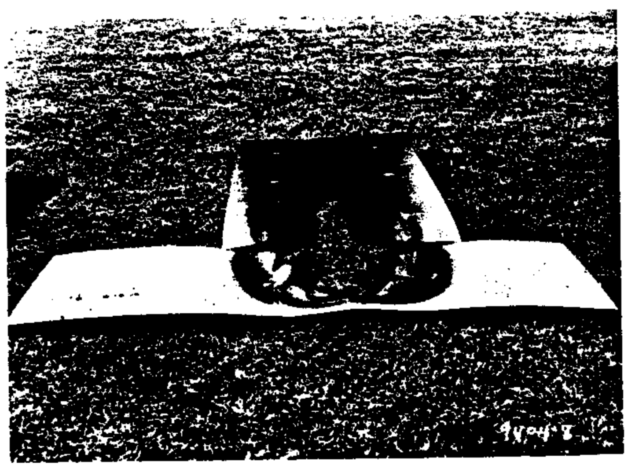
APPENDIX III

Photograph L

Sample of Test B After Removal



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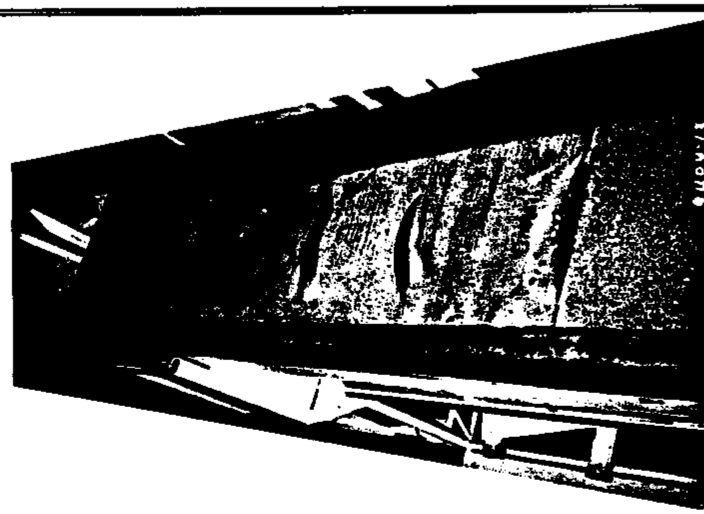
APPENDIX III

Photographs M, N, O

Samples After Tunnel Exposure

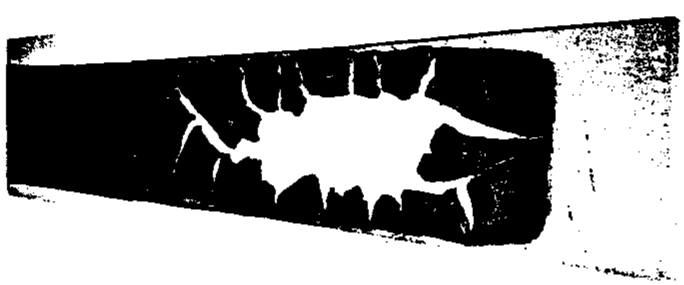


Photograph O.

Reverse Side of VINYLCEL After Tunnel Test - With sheet metal.

Photograph N.

VINYLCEL After Tunnel Test - Sheet metal used. (Exposed VINYLCEL shown).



Photograph M.

VINYLCEL After Tunnel Test - No sheet metal facing. (Exposed side down).

1.

APPENDIX 3F

SUMMARY OF STRUCTURAL DESIGN CODE COMPARISON

BY THE FRANKLIN RESEARCH CENTER

TABLE OF CONTENTS

3F.1 INTRODUCTION

The Franklin Research Center, under contract to the NRC, compared the structural design codes and loading criteria used in the design of the R. E. Ginna Nuclear Power Plant against the corresponding codes and criteria currently used for licensing of new plants at the time of the Systematic Evaluation Program (SEP). The current and older codes were compared paragraph by paragraph to determine what effects the code changes could have on the load carrying capacity of individual structural members.

The scope of the review was confined to the comparison of former structural codes and criteria with counterpart current requirements. Correspondingly, the assessment of the impact of changes in codes and criteria was confined to what can be deduced solely from the provisions of the codes and criteria.

In order to carry out the code review objective of identifying criteria changes that could potentially impair perceived margins of safety, the following scheme of classifying code change impacts was used.

Where code changes involved technical content (as opposed to those which are editorial, organizational, administrative, etc.), the changes were classified according to the following scheme.

Each such code change was classified according to its potential to alter perceived margins of safety [a] in structural elements to which it applied. Four categories were established:

- Scale A Change - The new criteria have the potential to substantially impair margins of safety as perceived under the former criteria.
- Scale A_X Change - The impact of the code change on margins of safety is not immediately apparent. Scale A_X code changes require analytical studies of model structures to assess the potential magnitude of their effect upon margins of safety.
- Scale B Change - The new criteria operate to impair margins of safety but not enough to cause engineering concern about the adequacy of any structural element.
- Scale C Change - The new criteria will give rise to larger margins of safety than were exhibited under the former criteria.

This appendix is the summary of the code comparison findings. It has been reproduced directly from Appendix B to the Franklin Research Center Report, TER-C5257-322, Design Codes, Design Criteria and Loading Combinations (SEP Topic III-7.B), R. E. Ginna Nuclear Power Plant, dated May 27, 1982, which was transmitted by letter to RG&E from the NRC, dated January 4, 1983.

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a. That is, if (all other considerations remaining the same) safety margins as computed by the older code rules were to be recomputed for an as-built structure in accordance with current code provisions, would there be a difference due only to the code change under consideration.

Table 3F.2-1
AISC 1963 VERSUS AISC 1980 SUMMARY OF CODE COMPARISON

Scale A

Referenced Subsection			Comments	
AISC 1980	**AISC 1963**	**Structural Elements Potentially Affected**	**Limitations**	**Scale**
1.5.1.1	1.5.1.1	Structural members under tension, except for pin connected members		
			$F_y \leq 0.833\ F_u$	C
			$0.8333\ F_u < F_y < 0.875\ F_u$	B
			$F_y \geq 0.875\ F_u$	A
1.5.1.2.2	—	Beam and connection where the top flange is coped and subject to shear, failure by shear along a plane through fasteners, or shear and tension along and perpendicular to a plane through fasteners	See case study 1 for details.	
1.5.1.4.1 Subpara.6	1.5.1.4.1	Box-shaped members (subject to bending) of rectangular cross section whose depth is not more than 6 times their width and whose flange thickness is not more than 2 times the web thickness	New requirement in the 1980 Code	
1.5.1.4.1 Subpara.7	1.5.1.4.1	Hollow circular sections subject to bending	New requirement in the 1980 Code	
1.5.1.4.4	—	Lateral support requirements for box sections whose depth is larger than 6 times their width	New requirement in the 1980 Code	
1.5.2.2	1.7	Rivets, bolts, and threaded parts subject to 20,000 cycles or more	Change in the requirements	
1.7 & Appendix B	1.7	Members and connections subject to 20,000 cycles or more	Change in the requirements	

1.9.1.2 & Appendix C	1.7	Slender compression unstiffened elements subject to axial compression or compression due to bending when actual width-to-thickness ratio exceeds the values specified in subsection 1.9.1.2	New provisions added in the 1980 Code, Appendix C. See case study 10 for details.	
1.9.2.3 & Appendix C	—	Circular tubular elements subject to axial compression	New requirement in the 1980 Code	
1.10.6	1.10.6	Hybrid girder - reduction in flange stress	New requirements added in the 1980 Code. Hybrid girders were not covered in the 1963 Code. See case study 9 for details.	
1.11.4	1.11.4	Shear connectors in composite beams	New requirements added in the 1980 Code regarding the distribution of shear connectors (eqn. 1.11-7). The diameter and spacing of the shear connectors are also introduced.	
1.11.5	—	Composite beams or girders with formed steel deck	New requirement in the 1980 Code	
1.15.5.2 1.15.5.3 1.15.5.4	—	Restrained members when flange or moment connection plates for and connections of beams and girders are welded to the flange of I or H shaped columns	New requirement in the 1980 Code	
1.13.3	—	Roof surface not provided with sufficient slope towards points of free drainage or adequate individual drains to prevent the accumulation of rain water (ponding)		
1.14.2.2	—	Axially loaded tension members where the load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the members	New requirement in the 1980 Code	
2.4 1st Para.	2.3 1st Para.	Slenderness ratio for columns. Must satisfy:	See case study 4 for details.	<u>Scale</u>
		$$\frac{l}{r} \leq \sqrt{\frac{2\pi^2 E}{F_y}}$$	$F_y \leq 40$ ksi $40 < F_y < 44$ ksi $F_y \geq 44$ ksi	C B A

2.7	2.6	Flanges of rolled W, M, or S shapes and similar built-up single-web shapes subject to compression	See case study 6 for details.	<u>Scale</u>
			$F_y \leq 36$ ksi	C
			$36 < F_y < 38$ ksi	B
			$F_y \geq 38$ ksi	
2.9	2.8	Lateral bracing of members to resist lateral and torsional displacement	See case study 7 for details.	
Appendix D	—	Web tapered members	New requirement in the 1980 Code	
<u>Scale B</u>				
1.9.2.2	1.9.2	Flanges of square and rectangular box sections of uniform thickness, of stiffened elements, when subject to axial compression or to uniform compression due to bending	The 1980 Code limit on width-to-thickness ratio of flanges is slightly more stringent than that of the 1963 Code.	
1.10.1	—	Hybrid girders	Hybrid girders were not covered in the 1963 Code. Application of the new requirement could not be much different from other rational method.	
1.11.4	1.11.4	Flat soffit concrete slabs, using rotary kiln produced aggregates conforming to ASTM C330	Lightweight concrete is not permitted in nuclear plants as structural members (Ref. ACI-349).	
1.13.2	—	Beams and girders supporting large floor areas free of partitions or other source of damping, where transient vibration due to pedestrian traffic might not be acceptable	Lightweight construction not applicable to nuclear structures which are designed for greater loads	
1.14.6.1.3	—	Flare type groove welds when flush to the surface of the solid section of the bar		
1.16.4.2	1.16.4	Fasteners, minimum spacing, requirements between fasteners		
1.16.5	1.16.5	Structural joints, edge distances of holes for bolts and rivets		

1.15.5.5	—	Connections having high shear in the column web	New insert ion the 1980 Code
2.3.1 2.3.2	—	Braced and unbraced multi-story frame - instability effect	Instability effect on short buildings will have negligible effect.
2.4	2.3	Members subject to combined axial and bending moments	Procedure used in the 1963 Code for the interaction analysis is replaced by a different procedure. See case study 8 for details.
Scale C			
1.3.3	1.3.3	Support girders and their connections - pendant operated traveling cranes	
		The 1963 Code requires 25% increase in live loads to allow for impact as applied to traveling cranes, while the 1980 Code requires 10% increase.	The 1963 Code requirement is more stringent, and, therefore, conservative.
1.5.1.5.3	1.5.2.2	Bolts and rivets - projected area - in shear connections	
		$F_p = 1.5\ F_u$ (1980 Code) $F_p = 1.35\ F_y$ (1963 Code)	Results using 1963 Code are conservative.
1.10.5.3	1.10.5.3	Stiffeners in girders - spacing between stiffeners at end panels, at panels containing large holes, and at panels adjacent to panels containing large holes	New design concept added in 1980 Code giving less stringent requirements. See case study 5 for details.
1.11.4	1.11.4	Continuous composite beams; where longitudinal reinforcing steel is considered to act compositely with the steel beam in the negative moment regions	New requirement added in the 1980 Code

Table 3F.3-1
ACI 318-63 VERSUS ACI 349-76 SUMMARY OF CODE COMPARISON

Scale A

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
7.10.3	805	Columns designed for stress reversals with variation of stress from f_y in compression to 1/2 f_y in tension	Splices of the main reinforcement in such columns must be reasonably limited to provide for adequate ductility under all loading conditions.
Chapter 9 9.1, 9.2, & 9.3 most specifically	Chapter 15	All primary load-carrying members or elements of the structural system are potentially affected	Definition of new loads not normally used in design of traditional buildings and redefinition of load factors and capacity reduction factors has altered the traditional analysis requirements.*
10.1 & 10.10	—	All primary load-carrying members	Design loads here refer to Chapter 9 load combinations.*
11.1	—	All primary load-carrying members	Design loads here refer to Chapter 9 load combinations.*
11.13	—	Short brackets and corbels which are primary load-carrying members	As this provision is new, any existing corbels or brackets may not meet these criteria and failure of such elements could be non-ductile type failure. Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
11.15	—	Applies to any elements loaded in shear where it is inappropriate to consider shear as a measure of diagonal tension and the loading could induce direct shear-type cracks	Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
11.16	—	All structural walls - those which are primary load-carrying, e.g., shear walls and those which serve to provide protection from impacts of missile-type objects	Guidelines for these kinds of wall loads were not provided by older codes; therefore, structural integrity may be seriously endangered if the design fails to fulfill these requirements.
18.1.4 & 18.4.2	—	Prestressed concrete elements	New load combinations here refer to Chapter 9 load combinations.*

Chapter 19	—	Shell structures with thickness equal to or greater than 12 inches	This chapter is completely new; therefore, shell structures designed by the general criteria of older codes may not satisfy all aspects of this chapter. Additionally, this chapter refers to Chapter 9 provisions.
Appendix A	—	All elements subject to time-dependent and position-dependent temperature variations and which are restrained such that thermal strains will result in thermal stresses	New appendix; older Code did not give specific guidelines on temperature limits for concrete. The possible effects of strength loss in concrete at high temperatures should be assessed.
Appendix B	—	All steel embedments used to transmit loads from attachments into the reinforced concrete structures	New appendix; therefore, considerable review of older designs is warranted.**
Appendix C	—	All elements whose failure under impulsive and impactive loads must be precluded	New appendix; therefore, considerations and review of older designs is considered important.**

Scale B

1.3.2	103(b)	Ambient temperature control for concrete inspection - upper limit reduced 5° (from 100°F to 95°F) applies to all structural concrete	Tighter control to ensure adequate control of curing environment for cast-in-place concrete.
1.5	—	Requirement of a "Quality Assurance Program" is new. Applies to all structural concrete	Previous codes required inspection but not the establishment of a quality assurance program.
Chapter 3	Chapter 4	Any elements containing steel with f_y > 60,000 psi or lightweight concrete	Use of lightweight concrete in a nuclear plant not likely. Elements containing steel with f_y > 60,000 psi may have inadequate ductility or excessive deflections at service loads.
3.2	402	Cement	This serves to clarify intent of previous code.
3.3	403	Aggregate	Eliminated reference to lightweight aggregate.
3.3.1	403	Any structural concrete covered by ACI 349-76 and expected to provide for radiation shielding in addition to structural capacity	Controls of ASTM C637, "Standard Specifications for Aggregates for Radiation Shielding Concrete," closely parallel those for ASTM C33, "Standard Specification for Concrete Aggregates."

3.3.3	403	Aggregate	To ensure adequate control.
3.4.2	404	Water for concrete	Improve quality control measures.
3.5	405	Metal reinforcement	Removed all reference to steel with f_y > 60,000 psi.
3.6	406, 407, & 408	Concrete mixtures	Added requirements to improve quality control.
4.1 & 4.2	501 & 502	Concrete proportioning	Proportioning logic improved to account for statistical variation and statistical quality control.
4.3	504	Evaluation and acceptance of concrete	Added provision to allow for design specified strength at age > 28 days to be used. Not considered to be a problem, since large cross sections will allow concrete in place to continue to hydrate.
5.7	607	Curing of very large concrete elements and control of hydration temperature	Attention to this is required because of the thicker elements encountered in nuclear-related structures.
6.3.3	—	All structural elements with embedded piping containing high temperature materials in excess of 150°F, or 200°F in localized areas not insulated from the concrete	Previous codes did not address the problem of long periods of exposure to high temperature and did not provide for reduction in design allowables to account for strength reduction at high (> 150°F) temperatures.
7.5, 7.6, & 7.8	805	Members with spliced reinforcing steel	Sections on splicing and tie requirements amplified to better control strength at splice locations and provide ductility.
7.9	805	Members containing deformed wire fabric	New sections to define requirements for this new material.
7.10 & 7.11	—	Connection of primary load-carrying members and at splices in column steel	To ensure adequate ductility.
7.12.3 7.12.4	—	Lateral ties in columns	To provide for adequate ductility.
7.13.1 through 7.13.3	—	Reinforcement in exposed concrete	New requirements to conform with the expected large thicknesses in nuclear related structures.
8.6	—	Continuous nonprestressed flexural members.	Allowance for redistribution of negative moments has been redefined as a function of the steel percentage.

9.5.1.1	—	Reinforced concrete members subject to bending - deflection limits	Allows for more stringent controls on deflection in special cases.
9.4	1505	Reinforcing steel - design strength limitation	See comments in Chapter 3 summary.
9.5.1.2 through 9.5.1.4	—	Slab and beams - minimum thickness requirements	Minimum thickness generally would not control this type of structure.
9.5.2.4	909	Beams and one-way slabs	Affects serviceability, not strength.
9.5.3	—	Non-prestressed two-way construction	Immediate and long time deflections generally not critical in structures designed for very large live loadings; however, design by ultimate requires more attention to deflection controls.
9.5.4 & 9.5.5	—	Prestressed concrete members	Control of camber, both initial and long time in addition to service load deflection, requires more attention for designs by ultimate strength.
10.2.7	—	Flexural members - new limit on B factor	Lower limit on B of 0.65 would correspond to an f'_c of 8,000 psi. No concrete of this strength likely to be found in a nuclear structure.
10.3.6	—	Compression members, with spiral reinforcement or tied reinforcement, non-prestressed and prestressed.	Limits on axial design load for these members given in terms of design equations. See case study 2.
10.6.1 10.6.2 10.6.3 10.6.4	1508	Beams and one-way slabs	Changes in distribution of reinforcement for crack control.
10.6.5	—	Beams	New insert
10.8.1 10.8.2 10.8.3	912	Compression members, limiting dimensions	Moment magnification concept introduced for compression members. Results using column reduction factors in ACI 318-63 are reasonably the same as using magnification.

10.11.1 10.11.2 10.11.3 10.11.4 10.11.5 10.11.5.1 10.11.5.2 10.11.6 10.11.7 10.12	915 916	Compression members, slenderness effects	For slender columns, moment magnification concept replaces the so-called strength reduction concept but for the limits stated in ACI 318-63 both methods yield equal accuracy and both are acceptable methods.
10.15.1 10.15.2 10.15.3 10.15.4 10.15.5 10.15.6	1404 - 1406	Composite compression members	New items - no way to compare; ACI 318-63 contained only working stress method of design for these members.
10.17	—	Massive concrete members, more than 48 in. thick	New item - no comparison.
11.2.1 11.2.2	—	Concrete flexural members	For non-prestressed members, concept of minimum area of shear reinforcement is new. For prestressed members, Eqn. 11-2 is the same as in ACI 318-63. Requirement of minimum shear reinforcement provides for ductility and restrains inclined crack growth in the event of unexpected loading.
11.7 through 11.8.6	—	Non-prestressed members	Detailed provisions for this load combination were not part of ACI 318-63. These new sections provide a conservative logic which requires that the steel needed for torsion be added to that required for transverse shear, which is consistent with the logic of ACI 318-63. This is not considered to be critical, as ACI 318-63 required the designer to consider torsional stresses; assuming that some rational method was used to account for torsion, no problem is expected to arise.

11.9 through 11.9.6	—	Deep beams	Special provisions for shear stresses in deep beams is new. The minimum steel requirements are similar to the ACI 318-63 requirements of using the wall steel limits. Deep beams designed under previous ACI 318-63 criterion were reinforced as walls at the minimum and therefore no unreinforced section would have resulted.
11.10 through 11.10.7	—	Slabs and footings	New provision for shear reinforcement in slabs or footings for the two-way action condition and new controls where shear head reinforcement is used. Logic consistent with ACI 318-63 for these conditions and change is not considered major.
11.11.1	1707	Slabs and footings	The change which deletes the old requirement that steel be considered as only 50% effective and allows concrete to carry 1/2 the allowable for two-way action is new. Also deleted was the requirement that shear reinforcement not be considered effective in slabs less than 10 in. thick. Change is based on recent research which indicates that such reinforcement works even in thin slabs.
11.11.2 through 11.11.2.5	—	Slabs	Details for the design of shearhead is new. ACI 318-63 had no provisions for shearhead design. This section for slabs and footings is not likely to be found in older plant designs. If such devices were used, it is assumed a rational design method was used.
11.12	—	Openings in slabs and footings	Modification for inclusion of shearhead design. See above conclusion.
11.13.1 11.13.2	—	Columns	No problem anticipated since previous code required design consideration by some analysis.

Chapter 12	—	Reinforcement	Development length concept replaces bond stress concept in ACI 318-63. The various 1_d lengths in this chapter are based entirely on ACI 318-63 permissible bond stresses. There is essentially no difference in the final design results in a design under the new code compared to ACI 318-63.
12.1.6 through 12.1.63	918(C)	Reinforcement	Modified with minimum added to ACI 318-63, 918(C).
12.2.2 12.2.3	—	Reinforcement	New insert in ACI 349-76.
12.4	—	Reinforcement of special members	New insert. Gives emphasis to special member consideration.
12.8.1 12.8.2	—	Standard hooks	Based on ACI 318-63 bond stress allowables in general; therefore, no major change.
12.10.1 12.10.2(b)	—	Wire fabric	New insert. Use of such reinforcement not likely in Category I structures for nuclear plants.
12.11.2	—	Wire fabric	New insert. Mainly applies to precast prestressed members.
12.13.1.4	—	Wire fabric	New insert. Use of this material for stirrups not likely in heavy members of a nuclear plant.
13.5	—	Slab reinforcement	New details on slab reinforcement intended to produce better crack control and maintain ductility. Past practice was not inconsistent with this in general.
14.2	—	Walls with loads in the Kern area of the thickness	Change of the order of the empirical equation (14-1) makes the solution compatible with Chapter 10 for walls with loads in the Kern area of the thickness.

15.5	—	Footings - shear and development of reinforcement	Changes here are intended to be compatible with change in concept of checking bar development instead of nominal bond stress consistent with Chapter 12.
15.9	—	Minimum thickness of plain footing on piles	Reference to minimum thickness of plain footing on piles which was in ACI 318-63 was removed entirely.
16.2	—	Design considerations for a structure behaving monolithically or not, as well as for joints and bearings.	New but consistent with the intent of previous code.
17.5.3	2505	Horizontal shear stress in any segment	Use of Nominal Average Shear Stress equation (17-1) replaces the theoretical elastic equation (25-1) of ACI 318-63. It provides for easier computation for the designer.
18.4.1	—	Concrete immediately after prestress transfer	Change allows more tension, thus is less conservative but not considered a problem.
18.5	2606	Tendons (steel)	Augmented to include yield and ultimate in the jacking force requirement.
18.7.1	—	Bonded and unbonded members	Eqn. 18-4 is based on more recent test data.
18.9.1 18.9.2 18.9.3	—	Two-way flat plates (solid slabs) having minimum bonded reinforcement	Intended primarily for control of cracking.
18.11.3 18.11.4	—	Bonded reinforcement at supports	New to allow for consideration of the redistribution of negative moments in the design.
18.13 18.14 18.15 18.16.1	—	Prestressed compression members under combined axial load and bending. Unbonded tendons. Post tensioning ducts. Grout for bonded tendons.	New to emphasize details particular to prestressed members not previously addressed in the codes in detail.
18.16.2	—	Proportions of grouting materials	Expanded definition of how grout properties may be determined.
18.16.4	—	Grouting temperature	Expanded definition of temperature controls when grouting.

Scale C

7.13.4	—	Reinforcement in flexural slabs	

10.14	2306	Bearing - sections controlled by design bearing stresses	ACI 318-63 is more conservative, allowing a stress of 1.9 (0.25 f'_c) = 0.475 $f'_c < 0.6 f'_c$
11.2.3	1706	Reinforcement concrete members without prestressing	Allowance of spirals as shear reinforcement is new. Requirement, where shear stress exceeds $6\phi\sqrt{f'_c}$ of 2 lines of web reinforcement was removed.
13.0 to end	—	Two-way slabs with multiple square or rectangular panels	Slabs designed by the previous criteria of ACI 318-63 are generally the same or more conservative.
13.4.1.5	—	Equivalent column flexibility stiffness and attached torsional members	Previous code did not consider the effect of stiffness of members normal to the plane of the equivalent frame.
17.5.4 17.5.5	—	Permissible horizontal shear stress for any surface, ties provided or not provided	Nominal increase in allowable shear stress under new code.

* Special treatment of load and loading combinations is addressed in other sections of the report.

** Since stress analysis associated with these conditions is highly dependent on definition of failure planes and allowable stress for these special conditions, past practice varied with designers' opinions. Stresses may vary significantly from those thought to exist under previous design procedures.

Table 3F.4-1
ACI 301-63 VERSUS ACI 301-72 (REVISED 1975) SUMMARY OF CODE COMPARISON

Scale B

Referenced Section ACI 301-72	Referenced Section ACI 301-63	Structural Elements Potentially Affected	Comments
3.8.2.1 3.8.2.3	309b	Lower strength concrete can be proportioned when "working stress concrete" is used	ACI 301-72 (Rev. 1975) bases proportioning of concrete mixes on the specified strength plus a value determined from the standard deviation of test cylinder strength results. ACI 301-63 bases proportioning for "working stress concrete" on the specified strength plus 15 percent with no mention of standard deviation. High standard deviations in cylinder test results could require more than 15 percent under ACI 301-72 (Rev. 1975)
3.8.2.2 3.8.2.3	309d	Mix proportions could give lower strength concrete	ACI 301-72 (Rev. 1975) requires more strength tests than ACI 301-63 for evaluation of strength and bases the strength to be achieved on the standard deviation of strength test results.
17.3.2.3	1704d	Lower strength concrete could have been used	ACI 301-72 (Rev. 1975) requires core samples to have an average strength at least 85 percent of the specified strength with no single result less than 75 percent of the specified strength. ACI 301-63 simply requires "strength adequate for the intended purpose." If "adequate for the intended purpose" is less than 85 percent of the specified strength, lower strength concrete could be used.

17.2	1702a 1703a	Lower strength concrete could have been used	ACI 301-72 (Rev. 1975) specifies that no individual strength test result shall fall below the specified strength by more than 500 psi. ACI 301-63 specifies that either 20 percent (1702a) or 10 percent (1703a) of the strength tests can be below the specified strength. Just how far below is not noted.
15.2.6.1	1502b1	Weaker tendon bond possible	ACI 301-72 (Rev. 1975) requires fine aggregate in grout when sheath is more than four times the tendon area. ACI 301-63 requires fine sand addition at five times the tendon area.
15.2.2.1 15.2.2.2 15.2.2.3	1502e1	Prestressing may not be as good	ACI 301-72 (Rev. 1975) gives considerably more detail for bonded and unbonded tendon anchorages and couplings. ACI 301-63 does not seem to address unbonded tendons.
8.4.3	804b	Cure of concrete may not be as good	ACI 301-72 (Rev. 1975) provides for better control of placing temperature. This will give better initial cure.
8.2.2.4	802b4	Concrete may be more nonuniform when placed	ACI 301-72 (Rev. 1975) provides for a maximum slump loss. This gives better control of the characteristics of the placed concrete.
8.3.2	803b	Weaker columns and walls possible	ACI 301-72 (Rev. 1975) provides for a longer setting time for concrete in columns and walls before placing concrete in supported elements.
5.5.2	—	Poor bonding of reinforcement to concrete possible	ACI 301-72 (Rev. 1975) provides for cleaning of reinforcement. ACI 301-63 has no corresponding section.
5.2.5.3	—	Reinforcement may not be as good	ACI 301-72 (Rev. 1975) provides for use of welded deformed steel wire fabric for reinforcement. ACI 301-63 has no corresponding section.
5.2.5.1 5.2.5.2	503a	Reinforcement may not be as good when welded steel wire fabric is used	ACI 301-72 (Rev. 1975) provides a maximum spacing of 12 in. for welded intersection in the direction of principal reinforcement.

5.2.1	—	Reinforcement may not have reserve strength and ductility	ACI 301-72 (Rev. 1975) has more stringent yield requirements.
4.6.3	406c	Floors may crack	ACI 301-72 (Rev. 1975) provides for placement of reshores directly under shores above, while ACI 301-63 states that reshores shall be placed "in approximately the same pattern."
4.6.2	—	Concrete may sag or be lower in strength	ACI 301-72 (Rev. 1975) provides for reshoring no later than the end of the working day when stripping occurs.
4.6.4	—	Concrete may sag or be lower in strength	ACI 301-72 (Rev. 1975) provides for load distribution by reshoring in multistory buildings.
4.2.13	—	Low strength possible if reinforcing steel is distorted	ACI 301-72 (Rev. 1975) requires that equipment runways not rest on reinforcing steel.
3.8.5	—	Possible to have lower strength floors	ACI 301-72 (Rev. 1975) places tighter control on the concrete for floors.
3.7.2 3.4.4	—	Embedments may corrode and lower concrete strength	ACI 301-72 (Rev. 1975) requires that it be demonstrated that mix water does not contain a deleterious amount of chloride ion.
3.4.2 3.4.3	—	Possible lower strength	ACI 301-72 (Rev. 1975) places tighter control on water-cement ratios for watertight structures and structures exposed to chemically aggressive solutions.
1.2	—	Possible damage to green or underage concrete resulting in lower strength	ACI 301-72 (Rev. 1975) provides for limits on loading of emplaced concrete.

Scale C

3.5	305	Better strength resulting from better placement and consolidation	ACI 301-63 gives a minimum slump requirement. ACI 301-72 (Rev. 1975) omits minimum slump which could lead to difficulty in placement and/or consolidation of very low slump concrete. A tolerance of 1 in above maximum slump is allowed provided the average slump does not exceed maximum. Generally the placed concrete could be less uniform and of lower strength.

3.6	306b	Better strength resulting from better placement and consolidation	ACI 301-63 provides for use of single mix design with maximum nominal aggregate size suited to the most critical condition of concreting. ACI 301-72 (Rev. 1975) allows waiver of size requirement if the architect-engineer believes the concrete can be placed and consolidated.
3.8.2.1	309b	Higher strength from better proportioning	ACI 301-63 bases proportioning for "ultimate strength" concrete on the specified strength plus 25%. ACI 301-72 (Rev. 1975) bases proportioning on the specified strength plus a value determined from the standard deviation of test cylinder strengths. The requirement to exceed the specified strength by 25% gives higher strengths than the standard deviation method.
4.4.2.2	404c	Better bond to reinforcement gives better strength	ACI 301-63 provides that form coating be applied prior to placing reinforcing steel. ACI 301-72 (Rev. 1975) omits this requirement. If form coating contacts the reinforcement, no bond will develop.
4.5.5	405b	Better strength and less chance of cracking or sagging	ACI 301-63 provides for keeping forms in place until the 28-day strength is attained. ACI 301-72 (Rev. 1975) provides for removal of forms when specified removal strength is reached.
4.6.2	406b	Better strength and less chance of cracking or sagging	Same as above but applied to reshoring.
4.7.1	407a	Better strength by curing longer in forms	ACI 301-63 provides for cylinder field cure under most unfavorable conditions prevailing for any part of structure. ACI 301-72 (Rev. 1975) provides only that the cylinders be cured along with the concrete they represent. Cure of cylinders could give higher strength than the in-place concrete and forms could be removed too soon.

5.2.2.1 5.2.2.2	—	Better strength, less chance of cracked reinforcing bars	ACI 301-72 (Rev. 1975) has less stringent bending requirement for reinforcing bars than does ACI 318-63.
5.5.4 5.5.5	505b	Better strength from reinforcement	ACI 301-63 provides for more overlap in welded wire fabric.
12.2.3	1201d	Better strength from better cure of concrete	ACI 301-63 provides for final curing for 7 days with air temperature above 50°F. ACI 301-72 (Rev. 1975) provides for curing for 7 days and compressive strength of test cylinders to be 70 percent of specified strength. This could allow termination of cure too soon.
14.4.1	1404	Better strength resulting from better uniformity	ACI 301-63 provides for a maximum slump of 2 in. ACI 301-72 (Rev. 1975) gives a tolerance on the maximum slump which could lead to nonuniformity in the concrete in place.
15.2.1.1	1502-c1b	Higher strength from higher yield prestressing bars	ACI 301-63 requires higher yield stress than does ACI 301-72 (Rev. 1975).
15.2.1.2	1502-c2	Higher strength from better prestressing steel	ACI 301-63 requires that stress curves from the production lot of steel be furnished. ACI 301-72 (Rev. 1975) requires that a typical stress-strain curve be submitted. The use of the typical curve may miss lower strength material.
16.3.4.3	1602-4c	Better strength resulting from better cylinder tests	ACI 301-63 requires 3 cylinders to be tested at 28 days; if a cylinder is damaged, the strength is based on the average of two. ACI 301-72 (Rev. 1975) requires only two 28-day cylinders; if one is damaged, the strength is based on the one survivor.
16.3.4.4	1602-4d	Better strength, less chance of substandard concrete	ACI 301-63 requires that less than 100 yd^3 of any class of concrete placed in any one day be represented by 5 tests. ACI 301-72 (Rev. 1975) allows strength tests to be waived on less than 50 yd^3.

17.3.2.3	1704d	Better strength could be developed	ACI 301-63 requires core strengths "adequate for the intended purposes." ACI 301-72 (Rev. 1975) requires an average strength at least 85 percent of the specified strength with no single result less than 75 percent of the specified strength. If "adequate for the intended purpose" is higher than 85 percent of the specified strength, the concrete is stronger.

Table 3F.5-1
ACI 318-63 VERSUS ASME B&PV CODE, SECTION III, DIVISION 2, 1980, SUMMARY OF CODE COMPARISON

Scale A

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3230	1506	Containment (load combinations and applicable load factor)*	Definition of new loads not normally used in design of traditional buildings.
Table CC-3230-1	1506	Containment (load combinations and applicable load factor)*	Definition of loads and load combinations along with new load factors has altered the traditional analysis requirements.
CC-3421.5	—	Containment and other elements transmitting in-plane shear	New concept. There is no comparable section in ACI 318-63, i.e., no specific section addressing in-plane shear. The general concept used here (that the concrete, under certain conditions, can resist some shear, and the remainder must be carried by reinforcement) is the same as in ACI 318-63. Concepts of in-plane shear and shear friction were not addressed in the old codes and therefore a check of old designs could show some significant decrease in overall prediction of structural integrity.
CC-3421.6	1707	Peripheral shear in the region of concentrated forces normal to the shell surface	These equations reduce to $V_c = 4\sqrt{f'_c}$ when membrane stresses are zero, which compares to ACI 318-63, Sections 1707 (c) and (d) which address "punching" shear in slabs and footings with the ϕ factor taken care of in the basic shear equation (Section CC-3521.2.1, Eqn. 10). Previous code logic did not address the problem of punching shear as related to diagonal tension, but control was on the average uniform shear stress on a critical section. See case study 12 for details.

CC-3421.7	921	Torsion	New defined limit on shear stress due to pure torsion. The equation relates shear stress from a biaxial stress condition (plane stress) to the resulting principal tensile stress and sets the principal tensile stress equal to $6\sqrt{f'_c}$. Previous code superimposed only torsion and transverse shear stresses. See case study 13 for details.
CC-3421.8	—	Bracket and corbels	New provisions. No comparable section in ACI 318-63; therefore, any existing corbels or brackets may not meet these criteria and failure of such elements could be non-ductile type failure.
CC-3532.1.2	—	Where biaxial tension exists	ACI 318-63 did not consider the problem of development length in biaxial tension fields.
CC-3900 All sections in this chapter	—	Concrete containment*	New design criteria. ACI 318-63 did not contain design criteria for loading such as impulse or missile impact. Therefore, no comparison is possible for this section.

Scale B

CC-3320	—	Shells	Added explicit design guidance for concrete reactor vessels not stated in the previous code. Acceptance of elastic behavior as the basis for analysis is consistent with the logic of the older codes.
CC-3340	—	Penetrations and openings	Added to ensure the consideration of special conditions particular to concrete reactor vessels and containments. These conditions would have been considered in design practice even though not specifically referred to in the old code.

Table CC-3421-1	1503(c)	Containment-allowable stress for factored compression loads	ACI 318-63 allowable concrete compressive stress was 0.85 f'_c if an equivalent rectangular stress block was assumed; also ACI 318-63 made no distinction between primary and secondary stress. ACI 318-63 used 0.003 in./in. as the maximum concrete compressive strain at ultimate strength.
CC-3421.4.1	1701	Containment and any section carrying transverse shear	Modified and amplified from ACI 318-63, Section 1701.1. 1. ϕ factors removed from all equations and included in CC-3521.2.1, Eqn. 17. 2. Separation of equations applicable to sections under axial compression and axial tension. New equations added. 3. Equations applicable to cross sections with combined shear and bending modified for case where $\rho < 0.015$. 4. Modification for low values of ρ will not be a large reduction; therefore, change is not deemed to be major.
CC-3421.4.2	2610(b)	Prestressed concrete sections	ACI 318-63, Eqn. 26-13 is a straight line approximation of Eqn. 8 (the "exact" Mohr's circle solution) with the prestress force shear component "$V\rho$" added. (Ref. ACI 426 R-74) ACI 318-63, Eqn. 26-12 modified to include members with axial load on the cross section and modified to reflect steel percentage. Remaining logic similar to ACI 318-63, Section 2610. Both codes intend to control the principal tensile stress.
CC-3422.1	1508(b)	Reinforcing steel	ACI 318-63 allowed higher f_y if full scale tests show adequate crack control.

			The requirement for tests where f_y > 60 ksi was used would provide adequate assurance, in old design, that crack control was maintained.
CC-3422.1	1503(d)	All ordinary reinforcing steel	ACI 318-63 allowed stress for load resisting purposes was f_y. However, a capacity reduction factor ϕ of 0.9 was used in flexure. Therefore, allowable tensile stress due to flexure could be interpreted as limited to some percentage of f_y less than 1.0 f_y and greater than 0.9 f_y. Limiting the allowable tensile stress to 0.9 f_y is in effect the same as applying a capacity reduction factor ϕ of 0.9 to the theoretical equation.
CC-3422.1		All ordinary reinforcing steel	ACI 318-63 had no provision to cover limiting steel strains; therefore, this section is completely new. Traditional concrete design practice has been directed at control of stresses and limiting steel percentages to control ductility. The logic of providing a control of design parameters at the centroid of all the bars in layered bar arrangement is consistent with older codes and design practice.
CC-3422.2	1503(d)	Stress on reinforcing bars	ACI 318-63 allowed the compressive steel stress limit to be f_y; however, the capacity reduction factor for tied compression members was $\phi = 0.70$ and for spiral ties $\phi = 0.75$, applied to the theoretical equation. As this overall reduction for such members is so large, part of the reduction could be considered as reducing the allowable compressive stress to some level less than f_y; therefore, the 0.9 f_y limit here is consistent with and reasonably similar to the older code.
CC-3423	2608	Tendon system stresses	ACI 318-63 Section 2608 is generally less conservative.

CC-3431.3	—	Shear, torsion, and bearing	ACI 318-63 does not have a strictly comparable section; however, the 50% reduction of the ultimate strength requirements on shear and bearing stresses to get the working stress limits is identical to the ACI 318-63 logic and requirements.
Table CC-3431-1	—	Allowable stresses for service compression loads	Allowable concrete compressive stresses are less conservative than or the same as the ACI 318-63 equivalent allowables.
CC-3432.2	1003(b)	Reinforcing bar (compression)	ACI 318-63 is slightly more conservative in using 0.4 f_y up to a limit of 30 ksi. The upper limit is the same, since ACI 359-80 stipulates max f_y = 60 ksi.
CC-3432.2 (b), (c)	1004	Reinforcing bar (compression)	Logic similar to older codes. Allowance of 1/3 overstress for short duration loading.
CC-3433	2606	Tendon system stress	Limits here are essentially the same as in ACI 318-63 or slightly less conservative; ACI 318-63 limits effective prestress to 0.6 of the ultimate strength or 0.8 of the yield strength, whichever is smaller.
CC-3521	—	Reinforced concrete	Membrane forces in both horizontal and vertical directions are taken by the reinforcing steel, since concrete is not expected to take any tension. Tangential shear in the inclined direction is taken, up to V_c' by the concrete, and the rest by the reinforcing steel. In all cases, the ACI concept of ϕ is incorporated in the equation as 0.9. While not specifically indicating how to design for membrane stresses, ACI 318-63 indicated the basic premises that tension forces are taken by reinforcing steel (and not concrete) and that concrete can take some shear, but any excess beyond a certain limit must be taken by reinforcing steel.
CC-3521.2.1	1701	Nominal shear stress	Similar to ACI 318-63, with the exception of ϕ, which equals 0.85, being included in the Eqn. 17.

			Placing ϕ in the stress formula, rather than in the formulae for shear reinforcement, provides the same end result.
CC-3532	—	Where bundled bars are used	Bundled bars were not commonly used prior to 1963; therefore, no criteria were specified in ACI 318-63. In more recent codes, identical requirements are specified for bundled bars.
CC-3532.1.2	918(c)	Where tensile steel is terminated in tension zones	Similar to older code, but maximum shear allowed at cutoff point increased to 2/3, as compared to 1/2 in ACI 318-63, over that normally permitted. Slightly less conservative than ACI 318-63. This is not considered critical since good design practice has always avoided bar cutoff in tension zones.
CC-3532.1.2	1801	Where bars carrying stress are to be terminated	Development lengths derived from the basic concept of ACI 318-63 where: bond strength = tensile strength $\Sigma_0 \mu L = A_b f_y$ $L = \frac{A_b f_y}{(\mu \pi D)}$ If $\mu = 9.5 \sqrt{f'_c}$ then $L = 0.0335 \frac{A_b f_y}{\sqrt{f'_c}}$ With $\phi = 0.85$ $L = 0.0394 \frac{A_b f_y}{\sqrt{f'_c}}$ No change in basic philosophy for #11 and smaller bars.
CC-3532.3	919(h) 801	Hooked bars	Change in format. New values are similar for small bars and more conservative for large bars and higher yield strength bars. Not considered critical since prior to 1963 the use of f_y > 40 ksi steel was not common.

CC-3533	919	Shear reinforcement	Essentially the same concepts. Bend of 135° now permitted (versus 80° formerly) and two-piece stirrups now permitted. These are not considered as sacrificing strength. Other items here are identical.
CC-3534.1	—	Bundled bars - any location	Provisions for bundled bars were not considered in ACI 318-63. Bundled bars were not commonly used before the early 1960s. Later codes provide identical provisions.
CC-3536	—	Curved reinforcement	Early codes did not provide detailed information, but good design practice would consider such conditions.
CC-3543	2614	Tendon and anchor reinforcement	Similar to concepts in ACI 318-63, Section 2614 but new statement is more specific. Basic requirements are not changed.
CC-3550	—	Structures integral with containment	Statement here is specific to concrete reactor vessels. The logic of this guideline is consistent with the design logic used for all indeterminate structures. ACI 318-63 did not specifically state any guideline in this regard.
CC-3560		Foundation requirements	There is no comparable section in ACI 318-63. These items were assumed to be controlled by the appropriate general building code of which ACI 318-63 was to be a referenced inclusion. All items are considered to be part of common building design practice.
Scale C			
CC-3421.9	2306 (f) and (g)	Bearing	ACI 318-63 is more conservative, allowing a stress of 1.9 (0.25 f'_c) = 0.475 $f'_c < 0.6 f'_c$
CC-3431.2	2605	Concrete (allowable stress in concrete)	Identical to ACI 318-63 logic.

Appendix II	—	Concrete reactor vessels	ACI 318-63 did not contain any criteria for compressive strength modification for multiaxial stress conditions. Therefore, no comparison is possible for Section II-1100. Because of this, ACI 318-63 was more conservative by ignoring the strength increase which accompanies triaxial stress conditions. This section probably does not apply to concrete containment structures.
CC-3531	—	All	Rather conservative for service loads. Using ϕ of 0.9 for flexure, $\frac{U}{\phi} = \frac{1.5}{0.9}\ to\ \frac{1.8}{0.9} = 1.67\ to\ 2.0$ for ACI 318-63. By using the value of 2.0, the upper limit of the ratio of factored to service loads is employed.

* Special treatment of load and load combinations is addressed in other sections of the report.