ATTACHMENT 2

JUSTIFICATION FOR THE CRITERIA FOR THE RE-EVALUATION OF CONCRETE MASONRY WALLS INDIAN POINT GENERATING STATION, UNIT 2

February 1981

CONTENTS

JUSTIFICATION FOR THE

CRITERIA FOR THE RE-EVALUATION

OF CONCRETE MASONRY WALLS

1.0 GENERAL

The specification is provided to establish design requirements and criteria for use in re-evaluating the structural adequacy of concrete block walls in nuclear power plants. Direct reference to building code criteria was not used for the following reasons:

1) The definition of the magnitude of seismic loads in building codes is different than that used in nuclear power plants. In building codes damping, ductility, site effects and framing systems are factored into the seismic design base shear force. In nuclear power plants these factors are considered explicitly in the design of components.

2) Building code allowable stresses do not consider two levels of earthquake ground motion and the magnitude of the ground motion included in the building code design spectrum is not explicit.

3) Factors such as damping, analysis procedures, effect of attached equipment, two levels of allowable stresses, operability and frequency variations are not considered in building codes.

Thus the specification was developed to address the problems unique to nuclear power plants.

2.0 GOVERNING CODES

As noted in Sec. 1 the specification covers most of the factors unique to nuclear power plants. Items not explicitly covered by the specification will be governed by the American Concrete Institute "Building Code Requirements for Concrete Masonry Structures". ACI-531(29). This code incorporates most of the recent research data available on concrete masonry.

3.0 LOADS AND LOAD COMBINATIONS

These are in conformance with the plant FSAR and are in accordance with the design of all structural elements.

4.0 MATERIALS

The project specifications indicate that materials used for the performance of the work were originally specified to meet the requirements given in this section.

5.0 DESIGN ALLOWABLES

The design allowable stresses given in Tables 1 and 2 are based on f'_m the prism compressive strength, m_o the mortar compressive strength or f_y the steel yield strength. The mortar compressive strength is based on the mimimun specified compressive strength of ASTM C-270. The concrete block unit compressive strength is based on the applicable ASTM Standard - ASTM C-90 for hollow units, ASTM C-145 for solid units and ASTM C-129 for hollow non-load bearing units. The steel yield strength is based on the specified grade of the steel.

The prism compressive strength f'_m is based on the specified values given in Table 4-3 of ACI 531-79. This Table provides a conservative estimate of f'_m based on the mortar and concrete block unit compressive strengths. The minimum ASTM specified values of these variables was used in determining the conservative estimate of f'_m.

5.1 ALLOWABLE STRESSES

The justification for the allowable stresses of Tables 1 and 2 on pgs 39 and follows.

5.1.1 AXIAL COMPRESSION (Reinforced and Unreinforced)

The following discussion of test results has been extracted from the commentary to the N.CMA Specification for the Design and Construction of Load Bearing Concrete Masonry.

The objective was to develop reasonable and safe engineering design criteria for nonreinforced concrete masonry based on all existing data. A review in 1967 of the compilation of all available test data on compressive strength of concrete masonry walls did not, according to some, provide a suitable relationship between wall strength and slenderness ratio. From a more recent analysis, it was noted in many of the 418 individual pieces of data that either the masonry units or mortar, or in some cases, both units and mortar, did not comply with the minimum strength requirements established for the materials permitted for use in "Engineered Concrete Masonry" construction. Accordingly, it was decided to re-examine the data, discarding all tests which included materials that did not comply with the following minimum requirements:

Material	Compressive Strength
Solid units	1000 psi
Hollow units	600 psi (gross)
Mortar	700 psi

Also eliminated from the new correlation were walls with a slenderness ratio of less than 6; walls with h/t ratio less than 6 were considered to be in the category of "prisms." For evaluation of slenderness reduction criteria, only axially loaded walls were used. The data that was available consisted of tests on 159 axially loaded walls with h/t ratio ranging between 6 and 18. With this as a starting point, the data were analyzed assuming that the parabolic slenderness reduction function, $(1 - (\frac{h}{40t})^3)$, is valid.

Basic equation used to evaluate the test data was:

$$\frac{f_{test}}{S.F.} = C_o f'_m \left(1 - \left(\frac{h}{40t}\right)^3\right) \qquad (1)$$

$$\frac{f_{test}}{f'_m \left(1 - \left(\frac{h}{40t}\right)^3\right)} = C_o \times S.F. \qquad (2)$$

$$C_o \times S.F. = K \qquad (3)$$

where

f'_m = Assumed masonry strength, net area, based on strength of units

f_{test} = Net area compressive strength of panel

S.F. = Safety factor

C_o = Strength reduction coefficient

h = Height of specimen, inches

t = Thickness of specimen, inches

Net area used in the above formulae is net area of the masonry, and does not distinguish between type of mortar bedding. In the evaluation, mortar strength was assumed to be constant and was not considered as a significant influence on wall strength.

It was determined that the objective of reasonable and safe criteria would be met if 90% of the "K" values were greater than the K value selected and gave a minimum safety factor of 3. Accordingly, the K values were listed in ascending order and the value satisfying the above conditions was K = .610 for the 159 tests as seen from Table 3. Therefore, from equation (3):

$$C_o \times S.F. = K$$

$$C_o \times 3 = 0.610$$

$$C_o = \frac{0.610}{3} = 0.205$$

This value, 0.205, agrees very closely with the coefficient 0.20 which had been used for a number of years with reinforced masonry design.

An analysis of the safety factors present with the formula:

$$f_m = 0.205 f'_m \left(1 - \left(\frac{h}{40t}\right)^3\right)$$

indicates the following:

Safety factor greater than 3 is available in 93% of the tests; greater than 4 in 51% of the tests; greater than 5 in 15% of the tests, and greater than 6 in 5% of the tests.

In ACI 531 the factor of 0.20 was increased to 0.225. The recommended value of 0.22 for unfactored loads has factors of safety comparable to those given above. Doubling this value for the factored loads was deemed reasonable and gives a factor of safety of 1.5 for 93% of all tests performed. Although the derivation given is for unreinforced walls the same values are recommended for reinforced walls.

Based on formula (2), "K" factors were calculated for all of the test specimens as listed in the following table:

TABLE 3

Ref.	Concrete Masonry Units: Percent Solid	Concrete Masonry Units: Strength, psi, net area	Concrete Masonry Units: f'_m, psi	Mortar: Str., psi	Mortar: Bedding	Walls: h/t	Walls: Strength, psi, net f_{test}	Walls: f'_m(C)	Walls: K	Walls: S.F.
1	63	1160	980	1180	Full	6.0	750	978	.798	3.83
	63	1160	980	1180	Full	6.0	685	978	.701	3.49
	63	1160	980	1160	FS	6.0	670	978	.686	3.42
	63	1160	980	900	FS	6.0	555	978	.568	2.83
	63	1200	1000	1230	Full	6.0	860	995	.863	4.30
	63	1200	1000	730	Full	6.0	625	995	.627	3.12
	63	1200	1000	960	FS	6.0	580	995	.582	2.89
	63	1200	1000	780	FS	6.0	650	995	.652	3.25
	63	1320	1060	880	Full	6.0	1110	1055	1.050	5.25
	63	1320	1060	810	Full	6.0	970	1055	.918	4.58
	63	1320	1060	810	FS	6.0	780	1055	.738	3.69
	63	1160	980	1080	Full	6.0	800	978	.818	4.08
	63	1160	980	1080	Full	6.0	670	978	.686	3.42
	63	1810	1275	1270	Full	6.0	940	1270	.739	3.67
	63	1810	1275	1270	Full	6.0	940	1270	.739	3.67
	63	1505	1150	1670	Full	6.0	825	1145	.719	3.60
	63	1505	1150	1670	Full	6.0	820	1145	.715	3.57
	63	1240	1020	980	Full	6.0	1010	1015	.993	4.95
	63	1240	1020	980	Full	6.0	870	1015	.856	4.26
	63	1720	1230	880	Full	6.0	1035	1225	.844	4.21
	63	1720	1230	880	Full	6.0	940	1225	.766	3.81
	63	1380	1090	1730	Full	6.0	1000	1085	.920	4.58
	63	1380	1090	1730	Full	6.0	1010	1085	.930	4.63
	63	1780	1262	1870	Full	6.0	1450	1257	1.152	5.75
	63	1780	1262	1870	Full	6.0	1570	1257	1.248	6.22
	43	3300	1790	1230	Full	6.0	1560	1782	.874	4.36
	43	3300	1790	1230	Full	6.0	1730	1782	.969	4.84
	70	1645	1208	1140	Full	6.0	1000	1200	.830	4.15
	70	1645	1208	1140	Full	6.0	1220	1200	1.013	5.06
8	63	509	458	3140	Full	6.0	303	455	.664	3.30
	63	509	458	1610	Full	6.0	295	455	.646	3.21
	63	509	458	1060	Full	6.0	295	455	.646	3.21
	63	840	756	3140	Full	6.0	532	753	.706	3.52
	63	840	756	1610	Full	6.0	540	753	.716	3.58
	63	840	756	1060	Full	6.0	505	753	.670	3.35
	63	875	788	3140	Full	6.0	438	785	.558	2.79

TABLE 3 (Continued)

	Concrete Masonry Units			Mortar		Walls				
Ref.	Percent Solid	Strength, psi, net area	f'_m, psi	Str., psi	Bedding	h/t	Strength, psi, net f_{test}	f'_m C	K	S.F.
8	63	875	788	1610	Full	6.0	430	785	.547	2.74
	63	875	788	1060	Full	6.0	500	785	.637	3.17
	63	1080	940	3140	Full	6.0	605	936	.646	3.22
	63	1080	940	1610	Full	6.0	715	936	.763	3.81
	63	1080	940	1060	Full	6.0	765	936	.817	4.07
	63	1230	1015	3140	Full	6.0	1160	1010	1.146	5.70
	63	1230	1015	1610	Full	6.0	1000	1010	.988	4.92
	63	1230	1015	1060	Full	6.0	1110	1010	1.097	5.46
	63	1410	1105	3140	Full	6.0	1140	1100	1.030	5.16
	63	1410	1105	1610	Full	6.0	985	1100	.893	4.45
	63	1410	1105	1060	Full	6.0	1030	1100	.935	4.66
	63	1520	1157	3140	Full	6.0	660	1152	.572	2.85
	63	1520	1157	1610	Full	6.0	740	1152	.642	3.20
	63	1520	1157	4780	Full	6.0	830	1152	.719	3.58
	63	1860	1295	3140	Full	6.0	1476	1290	1.143	5.70
	63	1860	1295	1610	Full	6.0	1539	1290	1.192	5.94
	63	1860	1295	1060	Full	6.0	1365	1290	1.058	5.27
	63	2510	1554	3140	Full	6.0	1698	1550	1.096	5.47
	63	2510	1554	1610	Full	6.0	1365	1550	.881	4.39
	63	2510	1554	1060	Full	6.0	1325	1550	.856	4.27
	63	3030	1710	3140	Full	6.0	2222	1705	1.304	6.50
	63	3030	1710	1610	Full	6.0	2222	1705	1.304	6.50
	63	3030	1710	1060	Full	6.0	1984	1705	1.164	5.80
	63	3740	1923	3140	Full	6.0	1857	1918	.969	4.82
	63	3740	1923	1610	Full	6.0	2523	1918	1.316	6.56
	63	3740	1923	4780	Full	6.0	2317	1918	1.209	6.03
	63	6640	2400	3140	Full	6.0	3587	2392	1.499	7.48
	63	6640	2400	1610	Full	6.0	3856	2392	1.612	8.04
	63	6640	2400	4780	Full	6.0	5031	2392	2.102	10.49
12**	100	1383	1257	2562	Full	7.0	1140	1254	.910	4.13
	100	1388	1640	3017	Full	7.0	1358	1635	.830	4.57
	100	1892	1853	2317	Full	7.0	1469	1846	.795	4.52
	100	1923	1630	2153	Full	7.0	1394	1625	.858	4.29
	100	2508	2390	2427	Full	7.0	1947	2380	.817	4.56
	100	2529	2630	2347	Full	7.0	2151	2620	.820	4.68
	100	2545	2130	2143	Full	7.0	1930	2120	.909	4.17
	100	2610	2220	3195	Full	7.0	2078	2210	.939	4.71
	100	2678	2030	2322	Full	7.0	1832	2020	.905	3.99
	100	4474	2210	2792	Full	7.0	1810	2200	.821	4.10
	100	4474	2540	2154	Full	7.0	2157	2530	.937	4.09

** f'_m values from this reference were determined from prism tests instead of assumed values. Test results multiplied by factor of 1.2

TABLE 3 (Continued)

Ref.	Concrete Masonry Units: Percent Solid	Strength, psi, net area	f'_m, psi	Mortar: Str., psi	Bedding	Walls: h/t	Strength, psi, net f_{test}	f'_m C	K	S.F.
5	62	2547	1556	1400	FS	9.0	1241	1540	.807	4.05
	62	1886	1305	1400	FS	9.0	1153	1290	.894	4.50
	62	1999	1350	1400	FS	9.0	967	1335	.724	3.63
	62	1499	1150	1400	FS	9.0	685	1135	.603	3.02
	62	1934	1325	1400	Full	9.0	1354	1310	1.033	5.19
	62	2305	1473	1400	FS	9.0	1096	1455	.752	3.78
	62	2136	1405	1400	FS	9.0	1128	1390	.812	4.07
	62	1773	1260	1400	FS	9.0	1088	1245	.873	4.38
	62	1298	1049	1400	FS	9.0	854	1037	.823	4.14
	62	1241	1031	1400	FS	9.0	685	1010	.678	3.41
	62	1612	1196	1400	FS	9.0	991	1180	.838	4.20
	62	1805	1273	1400	FS	9.0	1088	1260	.864	4.33
	62	1491	1146	1400	FS	9.0	854	1133	.754	3.78
	62	1088	944	1400	FS	9.0	629	933	.673	3.38
	62	1918	1318	1400	FS	9.0	1072	1302	.822	4.12
	62	1169	985	1400	FS	9.0	605	975	.621	3.12
	45	2655	1598	1400	FS	9.0	989	1578	.626	3.15
	62	1088	944	1400	FS	9.0	564	933	.604	3.03
	62	1290	1045	1400	FS	9.0	701	1032	.678	3.41
	62	1999	1350	1400	FS	9.0	1104	1335	.826	4.16
	62	1862	1296	1400	Full	9.0	1378+	1280	1.075	5.44+
	62	967	870	1400	Full	9.0	758	860	.881	4.42
	62	1967	1338	1400	Full	9.0	1241	1320	.938	4.72
5	57	2280	1463	1400	FS	9.3	1228	1450	.849	4.27
	67	1917	1318	1400	FS	9.3	836	1302	.642	3.23
	67	1380	1090	1400	FS	9.3	724	1078	.672	3.37
	67	1902	1312	1400	FS	9.3	1223	1300	.943	4.74
	67	1246	1023	1400	FS	9.3	739	1010	.731	3.67
	57	2087	1386	1400	FS	9.3	1193	1370	.871	4.38
	57	2087	1386	830	FS	9.3	1298	1370	.948	4.76
	57	2385	1505	1400	FS	9.3	719	1485	.484	2.44
	57	2385	1505	1400	FS	9.3	789	1485	.530	2.67
	57	2385	1505	1400	FS	9.3	1105	1485	.743	3.74
	57	2385	1505	1400	FS	9.3	1140	1485	.766	3.85
1	39	1590	1187	1130	Full	9.5	885	1170	.756	3.79
	39	1590	1187	1010	Full	9.5	1000	1170	.853	4.28
	39	1718	1238	1070	Full	9.5	949	1220	.777	3.89
	39	1718	1238	840	Full	9.5	910	1220	.745	3.73

TABLE 3 (Continued)

Ref.	Concrete Masonry Units: Percent Solid	Concrete Masonry Units: Strength, psi, net area	Concrete Masonry Units: f'_m, psi	Mortar: Str., psi	Mortar: Bedding	Walls: h/t	Walls: Strength, psi, net f_{test}	Walls: f'_m C	Walls: K	Walls: S.F.
1	63	1159	985	1180	Full	14.3	683	940	.726	3.62
	63	1159	985	1440	Full	14.3	690	940	.734	3.66
	63	1159	985	1440	Full	14.3	738	940	.784	3.91
	63	1159	985	1060	FS	14.3	532	940	.565	2.82
	63	1159	985	900	FS	14.3	563	940	.599	2.98
	63	1159	985	1920	FS	14.3	563	940	.599	2.98
	63	1206	1020	1230	Full	14.3	738	974	.758	3.80
	63	1206	1020	730	Full	14.3	683	974	.702	3.51
	63	1206	1020	1130	Full	14.3	746	974	.765	3.83
	63	1206	1020	960	FS	14.3	571	974	.586	2.94
	63	1206	1020	780	FS	14.3	603	974	.619	3.10
	63	1206	1020	1250	FS	14.3	595	974	.610	3.05
	63	1317	1080	880	Full	14.3	905	1030	.877	4.38
	63	1317	1080	750	Full	14.3	1063	1030	1.030	5.14
	63	1317	1080	810	Full	14.3	929	1030	.901	4.49
	63	1317	1080	1020	FS	14.3	714	1030	.692	3.45
	63	1317	1080	1020	FS	14.3	667	1030	.647	3.23
	63	1159	985	1120	Full	14.3	579	940	.616	3.07
	63	1159	985	1150	Full	14.3	635	940	.675	3.37
	63	1159	985	1080	Full	14.3	635	940	.675	3.37
	63	1810	1274	1270	Full	14.3	873	1218	.717	3.54
	63	1810	1274	940	Full	14.3	881	1218	.725	3.58
	63	1810	1274	1120	Full	14.3	817	1218	.671	3.32
	63	1508	1153	1380	Full	14.3	706	1100	.641	3.17
	63	1508	1153	1380	Full	14.3	746	1100	.677	3.34
	63	1508	1153	1670	Full	14.3	643	1100	.584	2.88
	63	1238	1025	1920	Full	14.3	833	978	.851	4.24
	63	1238	1025	980	Full	14.3	802	978	.819	4.09
	63	1238	1025	1280	Full	14.3	817	978	.835	4.16
	63	1714	1230	800	Full	14.3	1111	1172	.946	4.73
	63	1714	1230	800	Full	14.3	1127	1172	.959	4.79
	63	1714	1230	750	Full	14.3	1079	1172	.918	4.59
	63	1381	1090	1730	Full	14.3	968	1040	.930	4.64
	63	1381	1090	2200	Full	14.3	960	1040	.923	4.61
	63	1774	1245	2100	Full	14.3	1240	1190	1.043	5.21
	63	2253	1450	1230	Full	14.3	936	1385	.675	3.42
	63	2253	1450	1270	Full	14.3	920	1385	.664	3.37
	70	1643	1206	1180	Full	14.3	807	1150	.701	3.55
	70	1643	1206	1300	Full	14.3	986	1150	.857	4.33
	55	1273	1040	1220	Full	14.3	727	993	.732	3.66
	55	1273	1040	1220	Full	14.3	764	993	.770	3.84
7	100	2900	1665	1475	Full	15.0	1250	1565	.801	3.93
5	65	1746	1250	1400	Full	18.0	1108	1135	.975	4.87
	65	1246	1015	1400	Full	18.0	785	925	.850	4.25
	65	1562	1175	1400	Full	18.0	1208	1065	1.131	5.66

5.1.2 FLEXURAL COMPRESSION (Reinforced and Unreinforced)

It is assumed that masonry can develop 85% of its specified compressive strength at any section. The recommended procedure for calculating the flexural strength of a section is the working stress procedure, which assumes a triangular distribution of strain.

For normal loads an allowable stress of 0.33 f'_m has a factor of safety of 2.6 for the peak stress, which only exists at the extreme fibre of the unit and has been used in practice for many years. The recommended value for factored loads also only exists at the extreme fibre and is the value recommended in the ATC-3-06 provisions.

5.1.3 BEARING (Reinforced and Unreinforced)

These values for normal loads are taken directly from the ACI code. The value recommended for factored loads is the value recommended in the ATC-3-06 provision.

5.1.4 SHEAR (Reinforced)

Two major test programs have evaluated the shear strength on concrete block masonry walls. The first was performed by Schneider and his test results were used as the basis for developing the UBC, NCMA and ACI code allowable stresses for reinforced masonry.

A more recent and extensive test program has been performed at the University of California, Berkeley and these results will be used as a comparison with the code allowables. The test results are shown in Figure 2 and lower bound values are indicated for reinforcement taking all the shear and masonry taking all the shear. These are compared to the allowables recommended for unfactored and factored loads in Table 4.

For the unfactored loads the factor of safety varies from 2.22 to 3.0. For the factored loads the factor of safety varies from 1.20 to 1.76. The ductility indicator associated with stress levels for the factored loads is of the order of 3 which provides an added factor of safety.

Table 4: Comparison of Test Restuls and Code Allowables

Description	S	U	Test Results	$\frac{\text{Tests}}{\text{S}}$	$\frac{\text{Tests}}{\text{U}}$
Masonry Takes Shear					
M/Vd = 1	$0.9\sqrt{f'_m}$	$1.5\sqrt{f'_m}$	$2\sqrt{f'_m}$	2.22	1.33
M/Vd = 0	$2.0\sqrt{f'_m}$	$3.4\sqrt{f'_m}$	$5\sqrt{f'_m}$	2.50	1.47
Reinforcement Takes Shear					
M/Vd = 1	$1.5\sqrt{f'_m}$	$2.5\sqrt{f'_m}$	$3\sqrt{f'_m}$	2.0	1.20
M/Vd = 0	$2.0\sqrt{f'_m}$	$3.4\sqrt{f'_m}$	$6\sqrt{f'_m}$	3.0	1.76

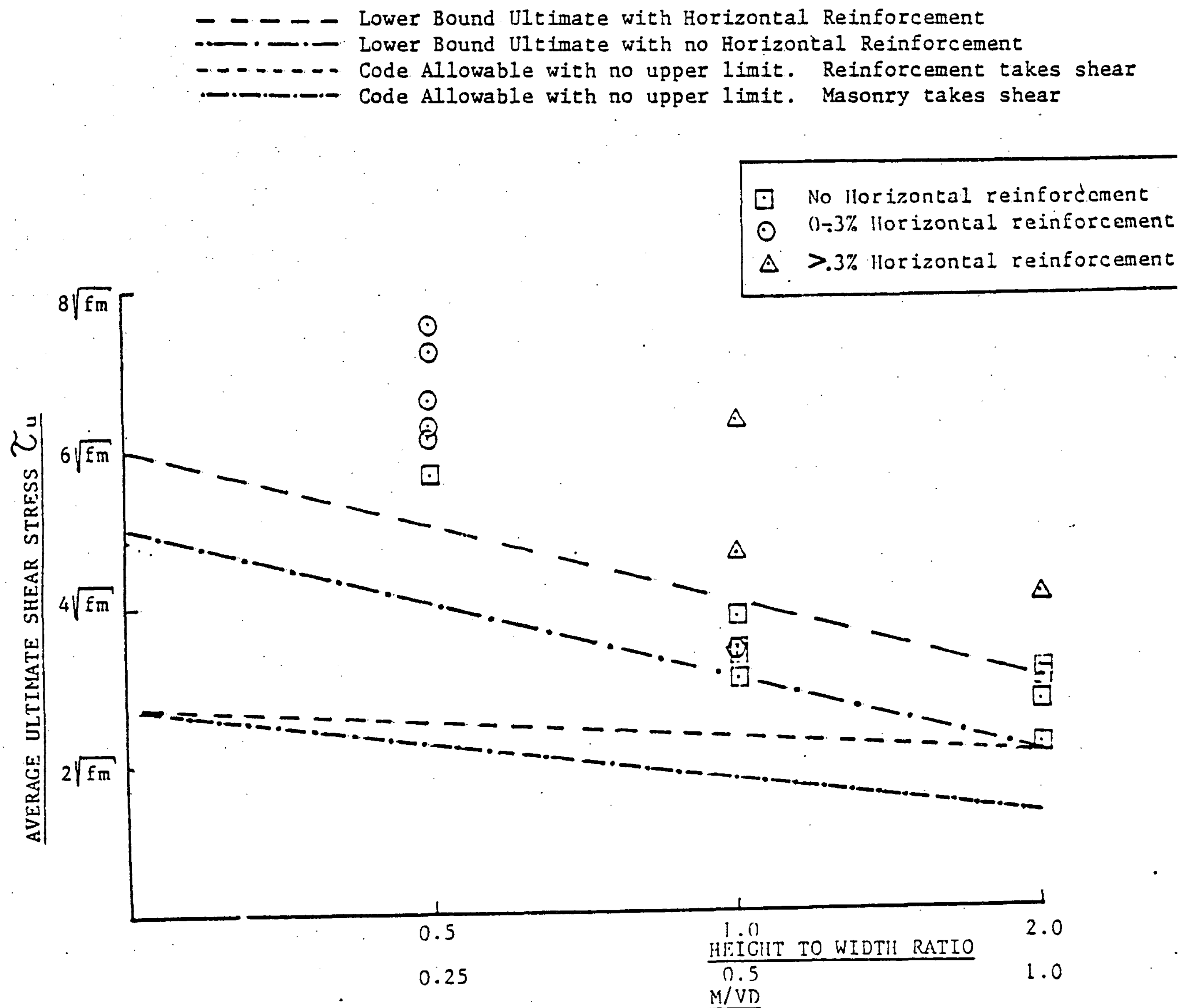
Lower Bound Ultimate with Horizontal Reinforcement
Lower Bound Ultimate with no Horizontal Reinforcement
Code Allowable with no upper limit. Reinforcement takes shear
Code Allowable with no upper limit. Masonry takes shear
No Horizontal reinforcement
0-.3% Horizontal reinforcement
>.3% Horizontal reinforcement
AVERAGE ULTIMATE SHEAR STRESS τ_u
$8\sqrt{fm}$
$6\sqrt{fm}$
$4\sqrt{fm}$
$2\sqrt{fm}$
0.5
1.0
2.0
HEIGHT TO WIDTH RATIO
0.25
0.5
1.0
M/VD

Figure 2

5.1.5 SHEAR (Unreinforced)

INTRODUCTION

The present literature on shear strength capability varies greatly on the approach used to determine acceptable values and to some extent, the controversey over these approaches and interpretation of the results. Debate, on the applicability of model or full size tests and the effects of monotonic versus cyclic loading further seems to complicate this resolution.

Much of the effort to define a permissible in-plane shear stress may be somewhat academic, in that the normal case for unreinforced walls being used in nuclear plant structures, the nature of the shear is one of being forced on the structural panel as a result of being confined by the building frame and not one of depending on the panel to transmit building shear forces. This forced drift or displacement results in shear stresses and strains, but because of the complex interaction between the panel and the confining structural elements strain or displacement is a more meaningful index for qualifying the in-plane performance of the panel. The area of in-plane strains is being addressed in another committee report.

The most extensive review on shear strength literature appears to have been done by Mayes, et al, [1] and published in Earthquake Engineering Research Center Report EERC No. 75-15 which was done for both brick and masonry block.

This report attempts to summarize some of the findings that appear to be pertinent towards defining permissible shear stress values that can be used for reevaluation of the non reinforced concrete masonry.

SUMMARY

The shear value of 0.9 $\sqrt{f'_m}$ provided by the ACI-531-79 [2,3] code for reinforced masonry appear to be reasonable basis on which to proceed with the reevaluation program. This value appears to conservatively bound the actual expected shear strength of concrete block masonry. A summary of several different sources for shear stress-design values is shown by Table 5. An increase in these allowable values for the re-evaluation program of $1.35\sqrt{f'_m}$ for severe loading conditions appears warranted. Any further increase at this time without further substantiation and review is not seen as advisable.

DISCUSSION

A number of tests have been identified as being the primary basis for permissible shear stress values in both National Concrete Masonry Association (NCMA) "Specification for the Design and Construction of Load-Bearing Concrete Masonry" [4,5] and the American Concrete Institute Standard "Building Code Requirements for Concrete Masonry Structures" (ACI-531-79). [2,3] No apparent tests are traceable to the origin of the Uniform Building Code (UBC) chapter 24 on "Masonry."[6]

Those tests performed to substantiate the NCMA values are primarily performed by the National Bureau of Standards (NBS) on full size (4 ft by 8 ft, and 8 ft by 8 ft) test panels. These tests were performed by Whittemore, et al and Fishburn[10] within the period 1939 to 1961. The Whittemore tests were done, as usual in that period, utilizing a hold down detail and thereby introducing a clamping or compressive stress within the assemblage. A number of studies have shown that compressive stresses affect the shear strength significantly. The Fishburn tests, utilize a racking configuration with the testing being performed on the panel in its original laid up position. A load setting up principal tension stress causing failure is an accepted measure of shear stress determination by the American Society of Testing Material for brickwork.[11] The test results from the above references used by NCMA are shown on Table 6.

The principal tests that seem to formulate the ACI 531 basis are the tests performed on concrete masonry piers for Masonry Research of Los Angeles, by Schneider.[12] These tests had a system for removing the compressive load on the specimen being loaded by shear and were set up to vary the a/d (M/Vd) ratio and measure this effect on a parametric basis.

The two predominant failure modes of a masonry panel under shear are diagonal tension (causing a "splitting" failure) and shear bond (causing a "joint separation" failure) or some combination of these two effects. The theory behind these were elaborated on by Yokel et al.[13] The parameter of normal stress and its effects on a shear strength, which was also reviewed by Yokel[13] and Mayes[1,14], has been demonstrated to be consequential on the determination of actual shear stress capability. This parameter is not identified, today, by any of the codes[2,4,6,15,16] shown in Table 5.

It is expected that under zero or small compressive loads the predominate shear failure will be by the shear bond mode of failure. Tests which have been done with regard to the determination of joint separation were performed by Copeland and Saxer,[17] as well as Hamid, et al.[18] These tests are, by their nature, extremely sensitive to normal stress and consequently do relate the effects of normal stress on permissable shear values. This relationship is shown on Table 5. It is of interest that there appears to be good correlation between these tests on the shear strength with zero normal stress.

The Applied Technology Council (ATC) is presently reviewing a formulation for increasing the shear stress as a function of normal stress. This formulation is developed to coincide with their present permissible shear stress of 12 psi and is consistent with the UBC's fundamental direction as a design code, forcing reinforcing for seismicly designed masonry structures.

As a practical matter, walls subject to the conditions of confinement will experience large compressive loads - although these are difficult to determine. Compressive loads for the most part, imparted by boundary conditions and behavior of the building frame are ignored in the evaluation of the masonry panel. If these normal stresses are added the shear resistance would be increased. This implies a conservatism on the allowable shear value when one assumes this value as chosen on the basis of zero normal stress. On this basis, and the tests results discussed, the shear value of $0.9\sqrt{f'_m}$ chosen by the ACI code appears to be justified and should be established as a reasonable basis by which to proceed with the re-evaluation.

Out of plane, or so called flexural shear is defined by the code as equalling $1.1\sqrt{f'_m}$. The derivation of this value is analogous to be permissible shear value of concrete, disregarding any reinforcement, of $1.1\sqrt{f'_c}$. Although this is somewhate different (there is no tension steel by which to determine the appropriate j distance), the actual value is a mute point since tension will be the critical value for determining out-of-plane acceptability of a flexural member.

Because of the nature of the stresses, however, and the various concerns with regard to the correctness of interpretation of the effects on boundary conditions as well as such conditions as: actual mortar properties; absorbtivity of the mortar; confinement or lack of it on the test specimen during test; arrangement and effect of actual load, it does not seem warranted to increase these stresses beyond a value of $1.35\sqrt{f'_m}$ (1.5×0.9 f'_m). This value is consistent with an adequate margin of safety for both the full panel wall test specimens referenced and the shear bond values observed by test. Any additional increase in the shear stress values for nonreinforced masonry under extreme environmental loads is not recommended at this time.

TABLE 5

SUMMARY - UNGROUTED MASONRY

	Source	Date	Shear Stress	Remarks
(1)	ACI-531 [2]	79	$0.9\sqrt{f'm} \leq 34$	$M/VD \geq 1$
(1)	NMCA [4]	79	34 23	Type M or S Motor Type N Mortar Based on NBS tests (circa 1939-1961)
(1)	UBC [6]	79	12/10*	Type M or S/N Mortar *12 psi for solid units
(1)	ATC 3-06 [15]	78	12 $*12 + 0.20\sigma_c \leq 30$	Lightweight units limited to 85 percent shear value *being proposed for compressiv stresses between 0 and 120 ps
(1)	Masonry Society [16]	Proposed	$1.0\sqrt{f'm} \leq 35$	$a/1 \leq 1$ May be increased by 0.20 c (due to dead load)
	Hamid, et al [18]	79	$76 + 1.07\sigma_c$	Ultimate value based on type S mortar
	Copeland/Saxen [17]	64	$70 + 9\sqrt{\sigma_c}$ (fitted)	Ultimate value based on 2630 compressive mortar strength

(1) Values based on inspected workmanship
σ_c = compressive stress.

TABLE 6 RACKING TEST DATA--NONREINFORCED CONCRETE MASONRY WALLS (1)

Construction	Mortar Type	Ultimate Racking Load, psi, Net Mortar Bedded Area	S.F. Act./Allow	Ref.
8" Hollow Units	N	66	2.87	7
	N	58	2.52	7
	N	57	2.48	7
6" 3-Core Hollow	N	69	3.00	8
	N	62	2.70	8
	N	78	3.39	8
8" Hollow Units	N	79	3.43	10
	N	79	3.43	10
	N	73	3.17	10
	N	119	5.17	10
	N	129	5.61	10
	N	109	4.74	10
	S	132	3.88	10
	S	139	4.09	10
	S	129	3.79	10
	S	159	4.68	10
	S	132	3.88	10
	S	159	4.68	10
4-2-4 Cavity Wall of Hollow Units	M	103	3.03	9
	M	108	3.18	9
	M	102	3.00	9

Avg = 3.65

Range = 2.48 - 5.61

(1) From Reference 5

LIST OF REFERENCES
FOR SHEAR (Unreinforced)

[1] Mayes and Clough, "Literature Survey - Compressive, Tensile, Bond and Shear Strength of Masonry," Earthquake Engineering Research Center, University of California, 1975.

[2] ACI Standard, "Building Code Requirements for Concrete Masonry Structures," (ACI 531-79).

[3] Commentary on "Building Code Requirements for Concrete Masonry Structures," (ACI 531-79).

[4] "Specification for the Design and Construction of Load-Bearing Concrete Masonry" - NCMA - 1979.

[5] Research Data and Discussion Relating to "Specification for the Design and Construction of Load Bearing Concrete Masonry" - NCMA - 1970.

[6] Uniform Building Code, Chapter 24 "Masonry" - 1979.

[7] Whittemore, Stang, and Parsons "Structural Properties of Six Masonry Wall Constructions," Building Materials and Structures Report No. 5., NBS - 1938.

[8] Whittemore, Stang, and Parsons "Structural Properties of Two Buch-Concrete Block Constructions and a Concrete Block Wall Construction Sponsored by the National Concrete Masonry Association," Building Materials and Structures Report.

[9] Whittemore, Stang, and Parsons, "Structural Properties of Concrete Block Cavity Wall Construction" Building Materials and Structures Report 21, NBS 1939.

[10] Fishburn, "Effect of Motar Strength and Strength of Unit on the Strength of Concrete Masonry Walls," Monograph 36, NBS, 1961.

[11] ASTM Standard Specification for Brick and Applicable Standard Testing Methods for Units and Masonry Assemblages - May 1975.

[12] Schneider, "Shear in Concrete Masonry Piers," California State Polytechnic College, Pomona, California.

[13] Yokel and Fattal "Failure Hypothesis for Masonry Shear Walls" - Journal of the Structural Division, March 1976.

[14] "A State of the Art Review - Masonry Design Criteria" - Computech - 1980.

[15] "Tentative Provisions for the Development of Seismic Regulations for Buildings" - Applied Technology Council Chapter 12 A - ATC 3-06-1978.

[16] The Masonry Society Standard Building Code Requirements for Masonry Construction, First Draft.

[17] Copeland and Saxer, "Tests of Structural Bond of Masonry Mortars to Concrete Block" - Journal of the Structural Division - November 1964.

[18] Hamid, Drysdale, and Heidebrecht, "Shear Strength of Concrete Masonry Joints," Journal of the Structural Division - July 1979.

5.1.6 TENSION (Unreinforced)

A. Normal to the Bed Joint

A summary of the static monotonic tests performed to determine code allowable stress for tension normal to the bed joint was given in the NCMA Specifications.

Stresses for tension in flexure are related to the type of mortar and the type of unit (hollow or solid). Research used to arrive at allowable stresses for tension in flexure in the veritcal span (i.e. tension perpendicular to the bed joints) consisted of 27 flexural tests of uniformly-loaded single-wythe walls of hollow units. These monotonic tests were made in accordance with ASTM E 72. Table 7 summarizes the test results.

From Table 7 the average modulus of rupture for walls built with Types M and S mortar is 93 psi on net area. For Type N mortar, the value is 64 psi. Applying a safety factor of four (4) to these values results in allowable stresses for hollow units as follows:

Mortar Type	Allowable Tension in Flexure
M&S	23 psi
N	16 psi

These values are consistent with those published in the 1970 ACI Committee 531 Report and which have been only slightly altered in ACI 531-79 Code.

Based upon these tests the minimum factors of safey for each mortar type are:

Mortar Type	Factor of Safety
M	3.87
S	2.60
N	2.81

To establish allowable tensile stresses for walls of solid units, the 8-inch composite walls in Table 8 were used. These walls, composed of 4-inch concrete brick and 4-inch hollow block, were greater than 75% solid, and thus were evaluated as solid masonry construction. Modulus of rupture (gross area) for these walls averaged 157 psi, giving an allowable stress of 39 psi when a safety factor of 4 is applied. The composite wall tests in Table 8 used Type S mortar. To establish allowable stresses for solid units with Type N mortar, the mortar influence established previously for hollow units was used:

$$\frac{23}{16} : \frac{39}{f} ; \; f = 27 \text{ psi}$$

The minimum factor of safety for these tests for Type S mortar was 2.33.

Recent dynamic tests have been performed at Berkeley and the values of tension obtained at cracking at the mid-height of the walls are as follows: 13 psi; 20 psi; 23 psi; 27 psi.

The recommended values have a factor of safety of 2.8 with respect to the lower bound of the static tests for the unfactored loads and are towards the lower limit of the initiation of cracking for the dynamic tests. An increase of 1.67 appeared reasonable for factored loads based on the static tests.

TABLE 7
FLEXURAL STRENGTH—SINGLE WYTHE WALLS OF HOLLOW UNITS—
UNIFORM LOAD—VERTICAL SPAN

Mortar Type Proportion ASTM C 270	Modulus of Rupture psi, Net Area	Reference
M	110	10
M	108	NCMA
M	102	10
M	97	10
M	95	NCMA
S	94	NCMA
M	91	NCMA
M	89	NCMA
N	88	4
S	84	10
S	83	NCMA
S	81	10
S	75	NCMA
S	69	NCMA
N	67	4
N	62	4
S	60	10
N	58	4
N	45	4
O	60	10
O	41	4
O	36	4
O	36	4
O	33	4
O	32	4
O	30	10
O	27	4

TABLE 8 FLEXURAL STRENGTH, VERTICAL SPAN CONCRETE MASONRY WALLS FROM TESTS AT NCMA LABORATORY

ASTM Mortar Type*	Wall				
				Modulus of Rupture	
	Nominal Thickness in.	Max. Uniform Load psf.	Net Section Modulus in 3/ft	Gross Area, psi	Net Mortar Bedded Area, psi
Monowythe Walls of Hollow Units					
M	8	85.15	80.97	61.74	88.73
M	8	87.10	80.97	63.15	90.76
M	8	91.00	80.97	65.97	94.82
M	8	103.35	80.97	74.93	107.69
S	8	62.40	80.97	45.24	69.47
S	8	72.15	80.97	52.31	75.18
S	12	183.3	164.64	57.11	93.94
S	12	161.2	164.64	50.22	82.62
Composite Walls of Concrete Brick & Hollow CMU					
S	8	222.3	103.82	161.16	180.67
S	8	219.7	103.82	159.29	178.55
S	8	187.2	78.16	135.72	202.09
S	8	228.8	103.82	165.88	185.95
S	8	218.4	78.16	158.34	235.77
S	8	223.6	78.16	162.11	241.38
S	12	171.6	139.83	53.46	103.55
S	12	150.8	139.83	46.98	91.00
S	12	156.0	139.83	48.60	94.14
S	12	213.2	139.83	66.42	128.66
Cavity Walls					
S	10	98.8	50.36	158.62	165.55
S	10	156.0	50.36	250.44	261.38
S	10	88.4	48.16	141.91	154.88
S	10	119.6	50.36	192.01	200.40
S	10	114.4	50.36	183.66	191.68
S	10	109.2	48.16	175.30	191.32
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(4-4-4)	145.6	50.36	233.73	243.94
S	12(6-2-4)	135.2	77.80	127.38	146.63
S	12(6-2-4)	119.6	77.80	112.68	129.70

* Mortar type by proportion requirements.

B. Tension Parallel to Bed Joints

Values for állowable tension in flexure for walls supported in the horizontal span are established by doubling the allowables in the vertical span. While it is recognized that flexural tensile strength of walls spanning horizontally is more a function of unit strength than mortar, it is conservative to use double the vertical span values. Table 9 lists a summary of all published tests and indicates an average safety factor of 5.3 for the 43 walls containing no joint reinforcement and 5.6 for the 15 walls containing joint reinforcement.

It is important to note that the factor of safety for those walls loaded at the quarter points, Reference (6), have an average factor of safety of 2.02 with a minimum value of 1.22, while those loaded at the center had an average factor of safety of 6.08 with a minimum value of 3.59. However, it should be noted that the values tested at the ¼ points were also tested at 15 days.

The results associated with the early date of testing and the use of quarter point loading are difficult to explain other than to state they are at variance with all other test results.

An increase in the allowable by a factor of 1.67 is recommended for factored loads. The committee believes that the recommended values could be increased because of the larger factors of safety in the test results; however the value of 1.67 was chosen to be compatible with the increase in other stresses for unreinforced masonry.

The values recommended for stack bonded construction although at variance with current building codes (which allow zero) are thought to be reasonable values for a reevaluation program. In a test program performed by PCA(1) a horizontally spanning stack bonded wall had 1/3 the capacity of an equivalent wall laid in running bond. The recommended values are in accordance with this test data. i.e. two-thirds of the value normal to the bed joint is equivalent to 1/3 the values recommended for parallel to the bed joint.

Reference:

1) Portland Cement Association, "Load Tests of Patterened Concrete Masonry Walls, " Trowel Talk an aid to Masonry Industry, 1963.

TABLE 9 FLEXURAL STRENGTH, HORIZONTAL SPAN, NONREINFORCED CONCRETE MASONRY WALLS

Construction	Mortar Type	Loading Type	Loading psf	Modulus of Rupture Net Area, psi	S.F. Act./Allow	Ref.
Monowythe 8"	N	Uniform	127	1[illegible]2	4.13	4
Hollow, 3-Core	N	"	136	141	4.41	4
	N	"	127	132	4.13	4
	N	"	169	176	5.50	4
	N	"	173	180	5.63	4
	O	"	123	128	4.00	4
	O	"	158	164	5.13	4
Monowythe 8"	N	"	149	155	4.84	4
Hollow, Joint	N	"	160	166	5.19	4
Reinf. @ 16 in.cc	N	"	193	201	6.28	4
	O	"	150	156	4.88	4
	O	"	186	193	6.03	4
Monowythe 8"	N	"	203	211	6.59	4
Hollow Joint	N	"	196	204	6.38	4
Reinf. @ 8 in.cc	O	"	202	210	6.56	4
	O	"	195	203	6.34	4
Monowythe 8"	N	1/4 pt	56	58	1.81	6
Hollow	N	"	38	39	1.22	6
	N	"	61	63	1.97	6
	N	"	60	62	1.94	6
	N	"	69	71	2.22	6
	N	"	93	96	3.00	6
8" Monowythe	M	Center	199	217	4.72	26
Hollow, 2-Core	M	"	176	192	4.17	26
	M	"	151	165	3.59	26
4-2-4 Cavity	M	"	111	210	4.57	26
Wall, Hollow	M	"	135	255	5.54	26
Units	M	"	95	180	3.91	26
8" Monowythe	M	"	159	173	3.76	26
Hollow 2-Core	M	"	159	173	3.76	26
Joint Re. @ 8"oc	M	"	191	208	4.52	26
4-2-4 Cavity of	M	"	159	300	6.52	26
Hollow Units Tied	M	"	159	300	6.52	26
w/Joint Re. @ 8"oc	M	"	159	300	6.52	26

TABLE 9 (Continued)

Construction	Mortar Type	Loading Type	Loading psf	Modulus of Rupture Net Area, psi	S.F. Act./Allow	Ref.
4" Hollow Monowythe	N	Center	138	365	11.41	25
	N	"	157	415	12.97	25
	N	"	101	268	8.38	25
8" Hollow Monowythe	M	"	268	202	4.39	25
	M	"	314	237	5.15	25
	M	"	314	237	5.15	25
8" Hollow Monowythe	N	"	277	210	6.56	25
	N	"	314	237	7.41	25
	N	"	314	237	7.41	25
8" Hollow Monowythe	O	"	259	195	6.09	25
	O	"	277	210	6.56	25
	O	"	277	210	6.56	25
8" Hollow Monowythe	M	"	268	202	4.39	25
	M	"	297	224	4.87	25
	M	"	277	210	4.56	25
8" Hollow Monowythe	N	"	277	210	6.56	25
	N	"	259	195	6.09	25
	N	"	297	224	7.00	25
8" Hollow Monowythe	O	"	360	271	8.45	25
	O	"	297	224	7.00	25
	O	"	268	202	6.31	25
12" Hollow Monowythe	N	"	352	142	4.44	25
	N	"	314	127	3.97	25
	N	"	333	134	4.19	25

5.1.7 SHEAR AND TENSILE BOND STRENGTH OF MASONRY COLLAR JOINT

The collar joint shear and tensile bond strength is a major factor in the behavior of multi-wythe masonry construction, particularly with respect to weak axis bending. A widely stated position is that for composite construction the collar joint must be completely filled with mortar. However, even if this joint is filled, there must be a transfer of shearing stress across this joint without significant slip in order for full composite interaction of the multiple wythes to be realized. Since the cracking strength, moment of inertia, and ultimate flexural strength, of the wall cross section are significantly influenced by the interaction of multiple wythes, it is crucial to establish the collar joint shear bond strength.

The only applicable published data on the shear bond strength of collar joints is that determined by Bechtel on the Trojan Nuclear Power Plant. A number of $^3/_8$" collar joints were tested and the accepted NRC allowable for the shear bond strength was 12 psi. Based on this information 12 psi is the recommended value for factored loads.

There is conflicting data available on the relationship between the shear and tensile bond strengths. In most tests performed on mortar bed joints (couplet tests) the shear bond strength was approximately twice the tensile bond strength. In a more recent method of evaluation by means of centrifugal force the shear bond strength was found to be 60% of the tensile bond strength. The authors of the report consider the test procedure to be an improvement over present methods since joint precompression is essentially eliminated as a result of the testing procedure.

Because of the conflict in the test data the committee recommended that the values for tensile bond strength be the same as for shear bond.

Unless metal ties are used at closely spaced intervals (less than 16 inches on center) it is recommended that their contribution to shear and tensile bond strength be neglected.

Reference:

(1) Hatzinkolas, M., Longworth, J., and Wararuk, J., "Evaluation of Tensile Bond and Shear Bond of Masonry by Means of Centrifugal Force," Alberta Masonry Institute, Edmonton, Alberta.

5.1.8 BOND (reinforced)

Values for bond stress are taken directly from the ACI Code. Due to the sensitivity of workmanship, degradation under cyclic load and the implications of a bond mode of failure it is recommended that these values be increased by 33 1/3% for factored loads.

5.1.9 GROUT CORE TENSILE STRESS

The tensile value recommended for the grout core tensile stress is taken from ACI 318 for concrete with a factor of safety of three. An increase of 1.67 was deemed reasonable for the factored loads.

5.2 DAMPING

The damping values for unreinforced walls are based on judgment and include a differentiation for the OBE and SSE force levels. This is based on the premise that damping increases as the stress level increases.

The damping values for reinforced walls are based on the accepted values for reinforced concrete.

There is no test data available in the literature to validate or refute these damping values.

6.0 ANALYSIS AND DESIGN

6.1 STRUCTURAL RESPONSE OF UNREINFORCED WALLS

6.1.1 OUT OF PLANE EFFECTS

The steps given in this section provide a logical conservative evaluation methodology to determine the stress levels in a masonry wall subjected to out of plane forces. The first two steps provide a lower bound estimate on the frequency of the wall since it assumes the wall spans in only one direction. For a wall with two or more sides capable of acting as boundaries the stresses resulting from one way or beam action will be conservative compared to those obtained from a more rigorous plate analysis.

If the stresses resulting from the analysis exceed the allowable stresses or the wall contains significant openings the beam analysis is not appropriate and the full effect of the actual boundary conditions must be accounted for in a plate analysis. For walls with openings it is recommended that a finite element plate analysis be performed to correctly model the effect of the opening. For walls without openings either a finite element analysis can be performed or standard test book formulae for plates may be used. If a multimode analysis is not performed it is recommended that the moments and stresses be increased by 1.05 to account for higher mode effects. Many parameter studies have been performed that indicate that in most cases the first mode of vibration contributes 98% or more to the total response of the wall. Thus the 1.05 factor is considered adequate.

6.1.2 FREQUENCY VARIATIONS OUT OF PLANE

This section acknowledges the fact that there will be variations in the frequency of the wall as a result of uncertainties in the mass of the wall and attached equipment, material and section properties and the modulus of elasticity of the masonry. The method selected to account for these uncertainties was a variation in the modulus of elasticity. The range of ± 25% for ungrouted walls and ± 20% for grouted walls is conservative when coupled with the use of a smoothed spectrum. If the frequency of a wall falls on the low frequency side of the amplified region of the response spectrum adequate provisions are included to ensure that the determination of the stress in the wall is conservative.

6.1.3 IN PLANE AND OUT OF PLANE EFFECTS

The plant FSAR provides for the design of a two-direction (one horizontal and one vertical) earthquake. The provisions of this section are consistent with the FSAR. The vertical component of motion is not included in the analysis procedure because the positive effect of the dead load on bed joint stresses is not included in the evaluation criteria. It should be noted however that the effect of vertical acceleration is included in determining the pipe and equipment loads on the wall.

6.2 STRUCTURAL RESPONSE OF REINFORCED MASONRY WALLS

6.2.1 OUT OF PLANE EFFECTS

The comments in Sec. 6.1.1 are applicable to the uncracked condition of a reinforced wall. If the wall cracks in either the vertical or horizontal direction cracked section properties of the wall are used to determine the frequency of either the beam or the plate. If a plate analysis is performed an orthotropic analysis must be performed in which different section properties in the horizontal and vertical directions are used.

6.2.2 EQUIVALENT MOMENT OF INERTIA

6.2.2.1 CRACKED CONDITION

The recommended value of I_e is taken from ACI 318. The formula was developed for slender columns and was considered to be appropriate for the out of plane analysis of masonry walls. The formula was checked against the test results of Dickey and Mackintosh[(1)] and reasonable agreement was obtained. It should be noted that if this formula is used it should be used over the total length of the wall and not over the cracked section.

The fully cracked section moment of inertia provides a lower limit and can be used over the cracked section of the wall. It is very conservative to use it over the full length of the wall.

Reference:

(1) Dickey, W. L., and Mackintosh, A., "Results of Variation of "b" or Effective Width in Flexure in Concrete Block Panels," Masonry Institute of America, 1971.

6.2.3 FREQUENCY VARIATIONS

See Sec. 6.1.2 for comments.

6.2.4 IN PLANE AND OUT OF PLANE EFFECTS

See Sec. 6.1.3 for comments.

6.3 ACCELERATIONS

The masonry walls are analyzed in a manner similar to that of equipment and piping systems. It is therefore conservative to use the envelop of the floor level spectra to which the wall is attached. If the wall is not attached at its top, forces will be induced from the floor level of the base of the wall and this should be used in the analysis.

6.5 IN PLANE EFFECTS

Load bearing structural masonry walls shall be evaluated on an allowable stress basis. The shear stress on the wall is determined from seismic analysis of the building and evaluated as in conventional design.

The majority of the masonry walls are not intended to be primary structural elements and for the purposes of this specification a non-load bearing or non structural wall is defined as follows.

1. It does not carry a significant part of the building's story shear or moment.

2. It does not significantly modify the behavior of adjacent structural elements.

In other words, the expected behavior of the building must be substantially the same whether such walls are present or not.

In-plane effects may be imposed on these masonry walls by the relative displacement between floors during seismic events. However, the walls do not carry a significant part of the associated story shear, and their stiffness is extremely difficult to define. In addition, since the experimental evidence to date demonstrates that the apparent in-plane strength of masonry walls depends heavily upon the in-plane stress boundary conditions, load or stress on the walls is not a reasonable basis for an evaluation criteria.

However, examination of the test data provided by the list of references for this section indicates that the gross shear strain of walls is a reliable indicator for predicting the onset of significant cracking. A significant crack is considered to be a crack in the central portion of the wall extending at least 10% of a wall's width or height. Cracking along the interface between a block wall and steel or concrete members does not limit the integrity of the wall, and is not addressed here. The gross shear strain is defined to be:

$$\gamma = \frac{\Delta}{H}$$

where: γ = strain

Δ = relative displacement between top and bottom of wall

H = height of wall

Test results indicate that to predict the initiation of significant cracking, masonry walls must be divided into two categories:

1. Unconfined Walls - not bounded by adjacent steel or concrete primary structure. Significant "confining" stresses cannot be expected.

2. Confined Walls - at a minimum, bounded top and bottom or bounded on three sides.

For unconfined concrete block masonry walls the works of Fishburn (2) and Becica (1) yield an allowable shear strain as defined above of 0.0001. It should be noted that Fishburn's test specimens were 15 days old, on average.

For confined walls, the most reliable data appears to be that of Mayes et al (4). In static and dynamic tests of masonry piers (confined top and bottom) varying block properties, mortar properties, reinforcement, vertical load and grout conditions, significant cracking was initiated at strains exceeding about $\gamma = 0.001$. It should be noted here that reinforcement can have no significant effect on the behavior prior to cracking. Similarly, the presence of cell grout should have no effect on stress or cracking in the mortar joints at a given strain. Both predictions are confirmed by the data in reference (4). In addition, the data shows that the onset of cracking is not sensitive to the magnitude of initial applied vertical load.

Klingner and Bertero (3) performed a series of cyclic tests to failure and found excellent correspondence with a non-linear analysis in which the behavior of an infilled frame prior to cracking is determined by an equivalent diagonal strut. While the equivalent strut technique has been used by many investigators to study the stiffness and load-carrying mechanisms of infilled frames, Klingner and Bertero found that the quasi-compressive failure of the strut could be used to predict the onset of significant cracking.

After some simplification of the relations in reference (3), the strength of the strut corresponds to a strain at cracking

$$= \frac{1 + \left(\frac{B}{H}\right)^2}{1000B/H} \qquad (1)$$

in which

B = wall width

H = wall height

assuming $E = 1000f'_m$

In summary, the recommended value for permissible in plane strain for service loads in unconfined walls is:

$$\gamma_u = 0.0001$$

and in confined walls

$$\gamma_c = 0.001$$

For factored loads these strains may be increased by 1.67.

For non-load bearing walls that are subjected to both in plane shear stresses and interstory drift effects the combination equation specified limits the combined effect such that the sum of the proportion of stress induced by each is less than 1. The complexity of this type of loading has not been validated by tests and the procedure recommended is deemed reasonable.

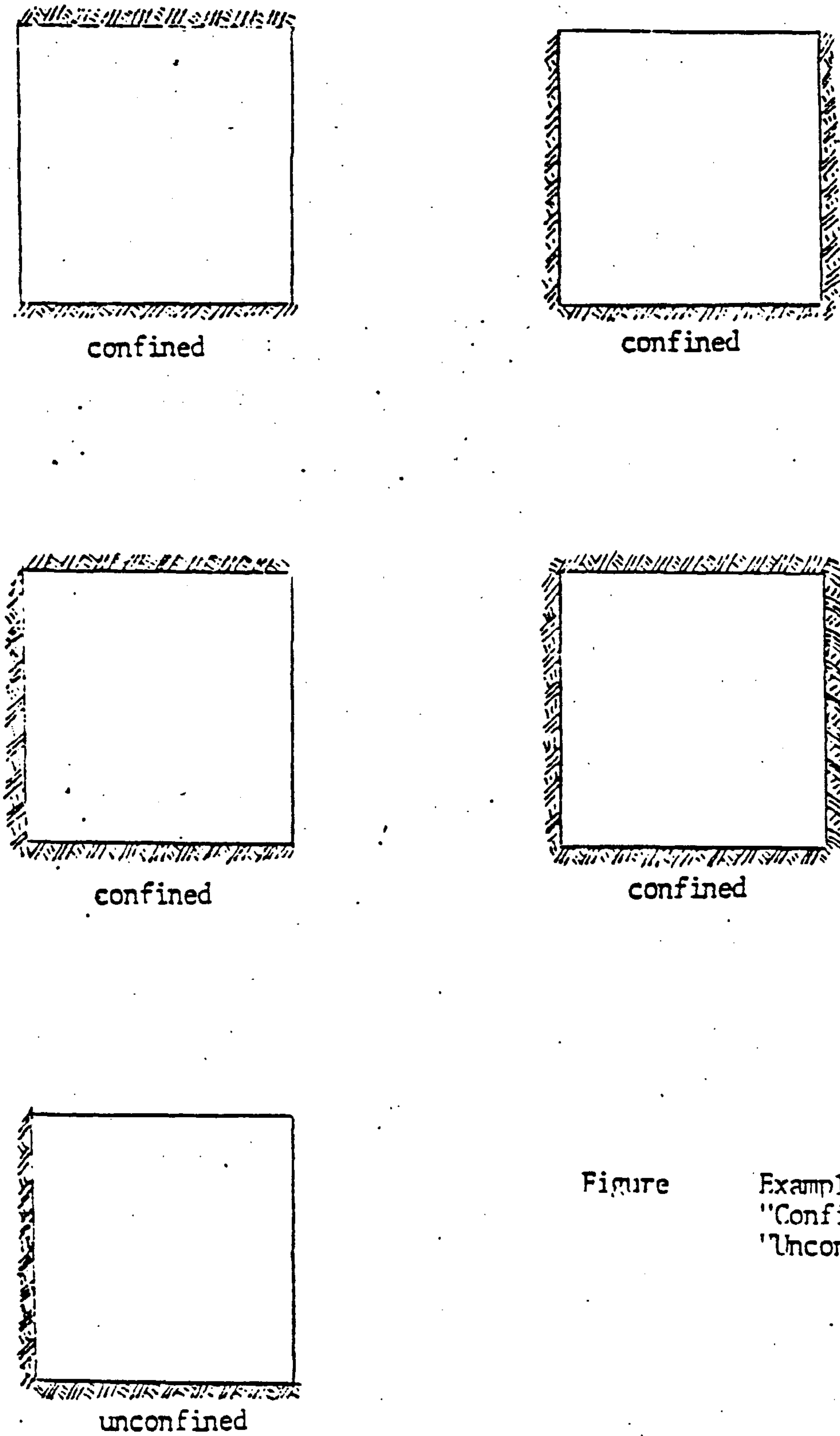

Figure Examples Defining "Confined" and "Unconfined" Walls

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6.6 EQUIPMENT

The method specified to account for the effect of equipment is conservative. The effect of equipment mass is included in the frequency calculation of the wall and thus the inertia effect of the mass of the equipment is included in the determination of the stress in the wall. This procedure by itself may not be sufficient because it does not account for any amplification of the equipment. Thus it is recommended that the fully amplified effect of the equipment be included by applying a static load and combining the resulting stresses with the stresses from the inertia loads. The combination shall be performed by the absolute sum method.

Refinement to this procedure is permitted if the frequency of the equipment is known and the SRSS method of combining stresses can be justified.

6.7 DISTRIBUTION OF CONCENTRATED OUT OF PLANE LOADS

The criteria for distributing concentrated out of plane loads is taken from the Uniform Building Code and is applicable to both reinforced and unreinforced construction. The limitation on stresses for beam or one way action is specified to ensure that these are not lower than those obtained from plate or two way action.

The allowable stresses for block pullout are based on the shear bond strength of a block since this is the mode of failure for unconfined block pullout. The discussion given in Sec. 5.1.5 for the allowable values for unreinforced shear walls indicates that these values are in accordance with the available test data on the shear bond strength of concrete masonry.

7.0 ALTERNATIVE ACCEPTANCE CRITERIA

7.1 REINFORCED MASONRY

Reinforced masonry walls which are well anchored and supported can undergo large ductile inelastic and out of plane flexural deformations (1). An approximate analysis method of determining the out of plane inelastic seismic response is the "energy balance" technique. This analysis technique is, in essence, similar to Blume's (2) reserve energy technique and is analogous to Newmark's (3) inelastic seismic response spectrum technique.

References:

(1) Dickey, W.L. and Mackintosh, A., "Results of Variation in "b" the Effective Width in Flexural Concrete Block Panels," Masonry Institute of America, 1971.

(2) Blume, J.A., Newmark, N.M. and Corning, L.H., "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions," Portland Cement Association, 1961.

(3) Newmark, N.M., "Current Trends in the Seismic Analysis and Design of High-Rise Structures," Chapter 16, Earthquake Engineering, Edited by R. L. Weigel, McGraw-Hill, 1970.

7.2 UNREINFORCED MASONRY

An extensive test program performed by Gabrielson (1) on blast loading of masonry walls provides validation of the concept of arch action of masonry walls subjected to loads that exceed those that cause flexural cracking of an unreinforced masonry wall. An analytical procedure was developed to predict with reasonable accuracy the ultimate capacity of the unreinforced walls tested. With a factor of safety of 1.5 the procedure is used to determine the ultimate or collapse capacity of masonry walls.

Reference:

(1) Gabrielson, G., Wilton, C. and Kaplan, K., "Response of Arching Walls and Debris from Interior Walls Caused by Blast Loading," URS Report 2030-23, URS Research Co., 1975.

Description	S Allowable (psi)	S Maximum (psi)	U Allowable (psi)	U Maximum (psi)
Compressive				
Axial(1)	$0.22f'_m$	1000	$0.44f'_m$	2000
Flexural	$0.33f'_m$	1200	$0.85f'_m$	2400
Bearing				
On full area	$0.25f'_m$	900	$0.62f'_m$	1800
On one-third area or less	$0.375f'_m$	1200	$0.95f'_m$	2400
Shear				
Flexural members(2)	$1.1\sqrt{f'_m}$	50	$1.7\sqrt{f'_m}$	75
Shear Walls(3,4)				
Masonry Takes Shear				
$M/Vd \geq 1$	$0.9\sqrt{f'_m}$	34	$1.5\sqrt{f'_m}$	56
$M/Vd = 0$	$2.0\sqrt{f'_m}$	74	$3.4\sqrt{f'_m}$	123
Reinforcement Takes Shear				
$M/Vd \geq 1$	$1.5\sqrt{f'_m}$	75	$2.5\sqrt{f'_m}$	125
$M/Vd = 0$	$2.0\sqrt{f'_m}$	120	$3.4\sqrt{f'_m}$	180
Reinforcement				
Bond				
Plain Bars		60		80
Deformed Bars		140		186
Tension				
Grade 40		20,000		$0.9F_y$
Grade 60		24,000		$0.9F_y$
Joint Wire		$.5F_y$ or 30,000		$0.9F_y$
Compression		$0.4F_y$		$0.9F_y$

Table 2 Allowable Stresses in Unreinforced Masonry

Description	S		U	
	Allowable (psi)	Maximum (psi)	Allowable (psi)	Maximum (psi)
Compressive				
Axial(1)	$0.22f'_m$	1000	$0.44f'_m$	2000
Flexural	$0.33f'_m$	1200	$0.85f'_m$	3000
Bearing				
On full area	$0.25f'_m$	900	$0.62f'_m$	2250
On one-third area or less	$0.375f'_m$	1200	$0.95f'_m$	3000
Shear				
Flexural members(2, 3)	$1.1\sqrt{f'_m}$	50	$1.7\sqrt{f'_m}$	75
Shear walls(2)	$0.9\sqrt{f'_m}$	34	$1.35\sqrt{f'_m}$	51
Tension				
Normal to bed joints				
Hollow units	$0.5\sqrt{m_o}$	25	$0.83\sqrt{m_o}$	42
Solid or grouted	$1.0\sqrt{m_o}$	40	$1.67\sqrt{m_o}$	67
Parallel to bed joints(4)				
Hollow units	$1.0\sqrt{m_o}$	50	$1.67\sqrt{m_o}$	84
Solid or grouted	$1.5\sqrt{m_o}$	80	$2.5\sqrt{m_o}$	134
Grout Core	$2.5\sqrt{f'_c}$		$4.2\sqrt{f'_c}$	
Collar joints				
Shear		8		12
Tension		8		12