DESIGN OF CONCRETE STRUCTURES

9-11 JULY 1986 - ROOM E7

BUI.MAD249.0469.2

Venue and Dates

Christchurch, 9-11 July 1986, at Room E7, School of Engineering, University of Canterbury.

Closing Date of Applications

Wednesday 18 June 1986.

As enrolment is limited, applications will be accepted in order of receipt.

Fee

$160 payable on application using the attached form.

The fee covers coffee, lunches, social hours and seminar notes for Sessions 2, 7, 9, 10 and 11.

Copies of additional notes "Applications of New Zealand Standard Code of Practice for the Design of Concrete Structures, NZS 3101:1982", New Zealand Concrete Society Technical Report No. 2, August 1983, which cover Sessions 1, 2, 3, 4, 5, 6 and 8, are also available to seminar participants at an additional cost of $28.

Design of Concrete Structures

9 - 11 July 1986

Joint Seminar of University of Canterbury—New Zealand Concrete Society

NZCS

Organised by the Department of Civil Engineering of the University of Canterbury.

DESIGN OF CONCRETE STRUCTURES

Joint University of Canterbury
New Zealand Concrete Society Seminar
Organised by the Department of Civil Engineering of the University of Canterbury.

The seminar will offer a state-of-the-art covering of the design of concrete structures with particular emphasis on the requirements for seismic loading. Topics considered to be well established will be covered only briefly during the seminar, while in the other areas, particularly where more recent research information has come to hand, an extended treatment will be offered.

The seminar will assist engineers to become more familiar with the New Zealand Standard Code of Practice for the Design of Concrete Structures NZS 3101:1982 and its applications. In some sessions of the seminar the presentations will be similar to those given in the seminars on NZS 3101:1982 organised in 1983 by the New Zealand Concrete Society. For this reason Technical Report No. 2 of the New Zealand Concrete Society, entitled "Applications of the New Zealand Standard Code of Practice for the Design of Concrete Structures, NZS 3101:1982", which includes a number of design examples, will be found useful. Special notes for this seminar will be prepared only for some sessions. A session on design charts and computer programs is also included.

In addition the seminar will include sessions on the design of concrete structures for the storage of liquids and on the design of concrete masonry, which will explain the background to the new codes DZ 3106 and NZS 4203P: 1985 recently issued by the Standards Association of New Zealand.

Lecturers

R. Park. Professor of Civil Engineering, University of Canterbury, Christchurch.

T. Paulay, Professor of Civil Engineering, University of Canterbury, Christchurch.

M. J. N. Priestley, Reader in Civil Engineering, University of Canterbury, Christchurch.

L. Gaerty, Assistant Director, New Zealand Concrete Research Association, Porirua.

(continued on back panel)

Seminar Programme and Lecturers

DESIGN OF CONCRETE STRUCTURES

Time	Topic	Lecturer
Wednesday 9 July 1986		
9.00-9.05	Introduction	
9.05-10.30	Session 1: General Design Requirements and Capacity Design Principles	T. Paulay
Coffee		
11.00-12.30	Session 2: Reinforced Concrete Members With Flexure With and Without Axial Load	R. Park
Lunch		
2.00-3.00	Session 3: Reinforced Concrete Members With Shear and Torsion	T. Paulay
Coffee		
3.30-5.00	Session 4: Reinforced Concrete Beam-Column Joints	R. Park
5.15-6.15	Social Hour	
Thursday 10 July 1986		
9.00-10.45	Session 5: Reinforced Concrete Structural Walls	T. Paulay
Coffee		
11.15-12.30	Session 6. Floor Slabs and Diaphragms	R. Park
Lunch		
2.00-3.30	Session 7: The Capacity Design of Hybrid Structures	T. Paulay
Coffee		
4.00-5.30	Session 8: Prestressed Concrete	R. Park
5.45-6.45	Social Hour	
Friday 11 July 1986		
9.00-10.30	Session 9: Design Charts and Computer Programs for Reinforced Concrete	L. Gaerty
Coffee		
11-00-12.30	Session 10: Structures for the Storage of Liquids	M. J. N. Priestley
Lunch		
2.00-3.30	Session 11: Masonry Structures	M. J. N. Priestley
Coffee		
4.00-5.30	Discussion	

D Harding.

DZ 3106
(CPT KMcI 6)

DZ 3106
Draft New Zealand Standard

CODE OF PRACTICE
FOR CONCRETE STRUCTURES FOR THE STORAGE OF LIQUIDS

CONTENTS

RELATED DOCUMENTS

In this document reference is made to the following:

NEW ZEALAND STANDARDS

NZS 1900:0000	Model building bylaw
NZS 3101:1982	The design of concrete structures
NZS 3103:1976	Sands for mortars and internal and external plasters
NZS 3104:1983	Concrete production - High grade and special grade
NZS 3109:1980	Concrete construction
NZS 3112:0000	Methods of test for concrete Part 2:1980 Tests relating to the determination of strength of concrete
NZS 3121:1980	Water and aggregate for concrete
NZS 3402P:1973	Hot-rolled steel bars for concrete reinforcement
NZS 3422:1975	Welded fabric of drawn steel wire for concrete reinforcement
NZS 4203:1984	General structural design and design loadings for buildings

NZS 4441:1972	Swimming pools

BRITISH STANDARDS

BS 1485:1983	Zinc coated hexagonal steel wire netting
BS 4102:1971	Steel wire for fences
BS 5337:1976	The structural use of concrete for retaining aqueous liquids

AMERICAN CONCRETE SOCIETY

ACI 344R-70	Design and construction of circular pretressed concrete structures

COMMITTEE REPRESENTATION

This draft for comment was prepared under the direction of the Building and Civil Engineering Divisional Committee (30/-) of the Standards Association of New Zealand.

The Concrete Structures for the Storage of Liquids Committee (31/10) was responsible for the preparation of the draft and consisted of the following persons:

Mr P J North
Dr M J N Priestley
Mr J Vessey.

FOREWORD

In 1978, two documents entitled 'Code of Practice for Concrete Structures for the Storage of Liquids' Parts 1 and 2 were promulgated, intended to replace the technical requirements of NZS 1900: Chapter 11.1 : 1964. The first of these, NZS 3106P Part 1:1978, a provisional standard, represented comparatively minor changes to the approach and substance of the 1964 document. The second document, published by SANZ as DZ 3106 Part 2, a draft for comment, contained a radically different 'crack control' approach, where design was basically controlled by calculated crack widths under service load conditions. This second document also contained much more detailed advice on seismic loading, temperature stresses, and effects of creep and shrinkage.

A small committee was formed in 1983 to reconcile the two approaches, and produce a new, single draft for comment, replacing both the earlier publications. In doing so, the committee was mindful of considerable resistance in the design profession to the 'crack control' approach. One of the reasons for this resistance was the uncertainty of crack control equations: different available equations result in a wide range of predicted crack widths, and the applicability of any of the equations to cracking of, for example, circular shell structures has yet to be established experimentally.

However, the committee also recognised that the approach presented in NZS 3106P: Part 1 1978 was largely outdated, and did not reflect advances in the state of knowledge of behaviour of concrete liquid retaining structures, made over the past 20 years.

Consequently this current draft for comment is more than just a meshing together of the previous two documents. It represents a new approach, the committees believes, one that is closer to traditional methods of design for these structures while still taking advantage of improved knowledge of their design. Design is solely based on working stress methods. Ultimate strength design using factored loads is seen as inappropriate when the basic design load (that of the contained liquid) is known to a comparatively high precision. Thus a load factor reflecting the uncertainty of magnitude of fluid loading would be close to unity. Conversely, performance under service loads is of prime importance, and a detailed control of service conditions is essential. Two categories of load combinations are listed: permanent or long duration loads, and combinations including infrequent combination of transient loads. For the two categories, different limiting stresses are allowed.

The format of this draft has also been changed somewhat, and is close to that adopted in recent SANZ structural design codes.

APPENDIX A
CYLINDRICAL TANK THERMAL TABLES

Tables are given for 3 base conditions: PINNED

FIXED

SLIDING

SFACT = $H^2/D.t.$

In each case, the top is assumed to be free.

Stresses are given for three thermal conditions.

(1) <u>Average Temperature Change</u>

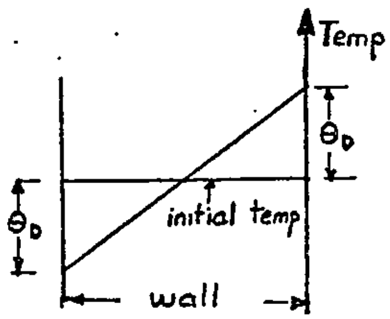
$\equiv$ Uniform temperature change of θ_A

Tables assume θ_A is a temperature increase

Reverse sign for temperature decrease

(2) <u>Differential Temperature Change</u>

$\equiv$ Differential temperature change of $\pm\ \theta_D$ i.e:



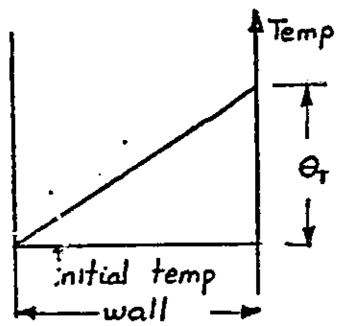
Note total gradient through wall $= 2\theta_D$

Tables assume outside hotter than inside

Reverse sign for inside hotter than outside.

(3) <u>Total Temperature Change</u>

$\equiv$ Temperature variation on <u>outside surface only</u> $= \theta_T$ i.e:



Tables assume outside hotter than inside.

Reverse sign for outside colder than inside.

<u>Stresses</u>

$f = C.E\alpha\theta$ where C = coefficient from appropriate table

E = Mod. of Elast.

α = Lin. coeff. of thermal expansion

$\theta = \theta_A$; θ_D or θ_T as appropriate.

<u>Note:</u> Sign Convention is tension +ve
Vertical stress given for <u>Inside</u> surface
(reverse sign for Outside surface)

PINNED-BASE CONDITION

RTICAL THERMAL STRESS - AVERAGE TEMPERATURE CHANGE

ACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	-0.022	-0.079	-0.175	-0.274	-0.379	-0.478	-0.526	-0.492	-0.348	0.000
3.000	0.000	-0.014	-0.065	-0.144	-0.227	-0.331	-0.457	-0.547	-0.551	-0.400	0.000
4.000	0.000	-0.005	-0.034	-0.077	-0.158	-0.274	-0.398	-0.523	-0.566	-0.442	0.000
5.000	0.000	0.000	-0.006	-0.036	-0.096	-0.204	-0.342	-0.480	-0.564	-0.468	0.000
6.000	0.000	0.000	0.000	-0.014	-0.058	-0.137	-0.281	-0.446	-0.562	-0.490	0.000
8.000	0.000	0.000	0.000	0.019	0.000	-0.067	-0.192	-0.365	-0.547	-0.518	0.000
10.000	0.000	0.000	0.000	0.024	0.012	-0.024	-0.132	-0.300	-0.516	-0.540	0.000
12.000	0.000	0.000	0.000	0.014	0.029	0.000	-0.072	-0.245	-0.461	-0.562	0.000
14.000	0.000	0.000	0.000	0.017	0.017	0.017	0.000	-0.202	-0.437	-0.554	0.000
16.000	0.000	0.000	0.000	0.000	0.019	0.038	0.077	-0.154	-0.422	-0.557	0.000

RTICAL THERMAL STRESS-DIFFERENTIAL TEMP. CHANGE

ACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.072	0.234	0.417	0.592	0.697	0.728	0.665	0.531	0.306	0.032
3.000	0.000	0.109	0.346	0.605	0.832	0.969	1.007	0.921	0.727	0.425	0.052
4.000	0.000	0.139	0.428	0.733	0.978	1.129	1.165	1.066	0.838	0.484	0.041
5.000	0.000	0.165	0.494	0.825	1.080	1.226	1.255	1.153	0.910	0.522	0.027
6.000	0.000	0.189	0.549	0.895	1.143	1.282	1.305	1.203	0.956	0.551	0.016
8.000	0.000	0.233	0.644	0.991	1.218	1.335	1.327	1.260	1.028	0.603	0.002
10.000	0.000	0.277	0.725	1.060	1.251	1.335	1.349	1.286	1.082	0.653	-0.001
12.000	0.000	0.320	0.797	1.116	1.266	1.321	1.335	1.297	1.125	0.702	-0.002
14.000	0.000	0.359	0.861	1.159	1.272	1.304	1.319	1.302	1.161	0.745	-0.001
16.000	0.000	0.397	0.916	1.191	1.274	1.287	1.301	1.304	1.187	0.786	-0.001

PINNED-BASE CONDITION

OOP THERMAL STRESS-INSIDE, AVERAGE CHANGE

FACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.205	0.156	0.107	0.141	-0.038	-0.134	-0.267	-0.426	-0.609	-0.812	-1.000
3.000	0.074	0.076	0.069	0.049	0.008	-0.054	-0.163	-0.319	-0.524	-0.762	-1.000
4.000	0.017	0.036	0.047	0.053	0.040	-0.004	-0.093	-0.241	-0.455	-0.723	-1.000
5.000	-0.008	0.014	0.034	0.050	0.052	0.025	-0.045	-0.180	-0.399	-0.690	-1.000
6.000	-0.011	0.003	0.023	0.040	0.053	0.041	-0.012	-0.137	-0.354	-0.661	-1.000
8.000	-0.015	-0.004	0.009	0.027	0.043	0.052	0.026	-0.069	-0.277	-0.607	-1.000
10.000	-0.008	-0.005	0.000	0.015	0.030	0.048	0.042	-0.024	-0.215	-0.564	-1.000
12.000	-0.002	-0.003	-0.003	0.005	0.022	0.041	0.051	0.006	-0.163	-0.524	-1.000
14.000	0.030	-0.002	-0.003	0.002	0.011	0.034	0.059	0.025	-0.127	-0.487	-1.000
16.000	0.002	0.000	-0.002	-0.001	0.006	0.028	0.064	0.036	-0.100	-0.557	-1.000

OOP THERMAL STRESS-OUTSIDE, AVERAGE CHANGE

FACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.205	0.164	0.135	0.205	0.060	0.002	-0.035	-0.236	-0.431	-0.686	-1.000
3.000	0.074	0.082	0.093	0.101	0.090	0.066	0.001	-0.123	-0.326	-0.619	-1.000
4.000	0.017	0.038	0.059	0.081	0.098	0.094	0.051	-0.053	-0.251	-0.565	-1.000
5.000	-0.008	0.014	0.036	0.062	0.086	0.099	0.079	-0.008	-0.195	-0.522	-1.000
6.000	-0.011	0.003	0.023	0.046	0.073	0.091	0.090	0.023	-0.152	-0.485	-1.000
8.000	-0.015	-0.004	0.008	0.021	0.043	0.076	0.096	0.063	-0.081	-0.421	-1.000
10.000	-0.008	-0.005	0.000	0.007	0.026	0.056	0.090	0.084	-0.029	-0.370	-1.000
12.000	-0.002	-0.003	-0.003	-0.001	0.012	0.041	0.077	0.094	0.003	-0.322	-1.000
14.000	0.000	-0.002	-0.003	-0.004	0.005	0.028	0.059	0.097	0.031	-0.327	-1.000
16.000	0.002	0.000	-0.002	-0.001	-0.000	0.014	0.036	0.092	0.052	-0.357	-1.000

PINNED-BASE CONDITION

HOOP THERMAL STRESS-INSIDE,DIFFERENTIAL CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.273	0.647	0.947	1.177	1.343	1.446	1.494	1.487	1.420	1.289	1.076
3.000	0.247	0.676	1.002	1.229	1.379	1.464	1.504	1.493	1.427	1.285	1.037
4.000	0.260	0.777	1.053	1.261	1.382	1.448	1.477	1.472	1.418	1.281	1.013
5.000	0.276	0.815	1.098	1.292	1.376	1.420	1.442	1.446	1.410	1.283	1.002
6.000	0.290	0.816	1.136	1.297	1.364	1.389	1.407	1.420	1.402	1.291	0.997
8.000	0.304	0.884	1.195	1.314	1.336	1.336	1.321	1.378	1.392	1.307	0.996
10.000	0.310	0.938	1.238	1.322	1.313	1.294	1.304	1.347	1.387	1.325	0.997
12.000	0.310	0.952	1.269	1.324	1.293	1.263	1.272	1.323	1.382	1.342	0.999
14.000	0.309	1.019	1.292	1.322	1.277	1.243	1.251	1.305	1.376	1.353	1.000
16.000	0.309	1.050	1.307	1.317	1.264	1.230	1.236	1.291	1.367	1.361	1.000

HOOP THERMAL STRESS-OUTSIDE,DIFFERENTIAL CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-1.728	-1.380	-1.138	-0.974	-0.871	-0.806	-0.769	-0.753	-0.772	-0.822	-0.936
3.000	-1.753	-1.384	-1.123	-0.990	-0.921	-0.885	-0.859	-0.840	-0.835	-0.869	-0.983
4.000	-1.741	-1.274	-1.102	-1.004	-0.971	-0.959	-0.943	-0.912	-0.885	-0.894	-1.002
5.000	-1.725	-1.245	-1.081	-1.015	-1.014	-1.022	-1.010	-0.970	-0.919	-0.906	-1.008
6.000	-1.711	-1.253	-1.062	-1.026	-1.049	-1.073	-1.064	-1.014	-0.943	-0.909	-1.009
8.000	-1.696	-1.200	-1.037	-1.043	-1.103	-1.145	-1.111	-1.077	-0.979	-0.911	-1.006
10.000	-1.691	-1.162	-1.024	-1.061	-1.138	-1.188	-1.186	-1.116	-1.003	-0.911	-1.003
12.000	-1.691	-1.134	-1.018	-1.079	-1.164	-1.213	-1.209	-1.144	-1.024	-0.912	-1.001
14.000	-1.691	-1.111	-1.019	-1.096	-1.182	-1.227	-1.224	-1.164	-1.043	-0.916	-1.000
16.000	-1.692	-1.093	-1.023	-1.113	-1.196	-1.235	-1.233	-1.179	-1.061	-0.922	-1.000

PINNED-BASE CONDITION

VERTICAL THERMAL STRESS-TOTAL EFFECTS

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.025	0.078	0.121	0.159	0.159	0.125	0.070	0.019	-0.021	0.016
3.000	0.000	0.047	0.141	0.231	0.303	0.319	0.275	0.187	0.088	0.012	0.025
4.000	0.000	0.067	0.197	0.328	0.410	0.427	0.383	0.272	0.136	0.021	0.021
5.000	0.000	0.082	0.244	0.394	0.492	0.511	0.457	0.336	0.173	0.027	0.013
6.000	0.000	0.095	0.274	0.441	0.543	0.573	0.512	0.378	0.197	0.031	0.005
8.000	0.000	0.117	0.322	0.505	0.609	0.634	0.582	0.448	0.241	0.042	0.001
10.000	0.000	0.138	0.362	0.542	0.631	0.655	0.609	0.493	0.283	0.056	-0.001
12.000	0.000	0.160	0.398	0.565	0.648	0.661	0.631	0.526	0.332	0.070	-0.001
14.000	0.000	0.179	0.431	0.588	0.645	0.660	0.659	0.550	0.362	0.096	-0.001
16.000	0.000	0.198	0.458	0.595	0.646	0.663	0.689	0.575	0.382	0.114	-0.001

HOOP THERMAL STRESS-INSIDE,TOTAL EFFECTS

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.239	0.401	0.527	0.659	0.652	0.656	0.613	0.531	0.406	0.239	0.038
3.000	0.161	0.376	0.536	0.639	0.694	0.705	0.670	0.587	0.452	0.261	0.018
4.000	0.139	0.407	0.550	0.657	0.711	0.722	0.692	0.616	0.481	0.279	0.007
5.000	0.134	0.414	0.566	0.666	0.714	0.723	0.699	0.633	0.506	0.296	0.001
6.000	0.139	0.410	0.580	0.669	0.708	0.715	0.698	0.641	0.524	0.315	-0.001
8.000	0.145	0.440	0.602	0.671	0.690	0.694	0.674	0.655	0.557	0.350	-0.002
10.000	0.151	0.467	0.619	0.668	0.672	0.671	0.673	0.662	0.586	0.380	-0.001
12.000	0.154	0.490	0.633	0.664	0.658	0.652	0.652	0.665	0.610	0.409	-0.001
14.000	0.155	0.508	0.644	0.662	0.644	0.639	0.655	0.665	0.625	0.433	0.003
16.000	0.155	0.525	0.653	0.658	0.635	0.629	0.650	0.664	0.634	0.402	0.000

HOOP THERMAL STRESS-OUTSIDE,TOTAL EFFECTS

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-0.761	-0.608	-0.501	-0.385	-0.405	-0.402	-0.432	-0.495	-0.802	-0.754	-0.968
3.000	-0.840	-0.641	-0.515	-0.445	-0.416	-0.410	-0.429	-0.481	-0.580	-0.743	-0.992
4.000	-0.862	-0.618	-0.521	-0.461	-0.437	-0.432	-0.446	-0.483	-0.568	-0.729	-1.001
5.000	-0.866	-0.616	-0.522	-0.476	-0.464	-0.462	-0.466	-0.489	-0.557	-0.714	-1.004
6.000	-0.861	-0.625	-0.520	-0.490	-0.488	-0.491	-0.487	-0.495	-0.547	-0.697	-1.005
8.000	-0.856	-0.602	-0.515	-0.511	-0.530	-0.534	-0.508	-0.507	-0.530	-0.666	-1.003
10.000	-0.850	-0.584	-0.512	-0.527	-0.556	-0.566	-0.546	-0.516	-0.516	-0.640	-1.001
12.000	-0.846	-0.568	-0.511	-0.540	-0.576	-0.58[illegible]	-0.566	-0.525	-0.510	-0.617	-1.0[illegible]
14.000	-0.846	-0.557	-0.511	-0.550	-0.589	-0.60[illegible]	-0.583	-0.533	-0.503	-0.601	-1.0[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.61[illegible]	-0.598	-0.544	-0.504	-0.639	-1.000

FIXED-BASE CONDITION

RTICAL THERMAL STRESS - AVERAGE TEMPERATURE CHANGE

ACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	-0.024	-0.086	-0.158	-0.211	-0.214	-0.142	0.046	0.401	0.934	1.726
3.000	0.000	-0.025	-0.094	-0.184	-0.266	-0.328	-0.299	-0.151	0.191	0.803	1.739
4.000	0.000	-0.019	-0.072	-0.158	-0.250	-0.326	-0.360	-0.254	0.062	0.696	1.752
5.000	0.000	-0.012	-0.048	-0.114	-0.210	-0.306	-0.366	-0.312	-0.042	0.606	1.758
6.000	0.000	-0.007	-0.029	-0.079	-0.158	-0.259	-0.353	-0.346	-0.122	0.526	1.742
8.000	0.000	0.000	-0.010	-0.029	-0.077	-0.173	-0.298	-0.365	-0.230	0.384	1.766
10.000	0.000	0.000	0.012	0.000	-0.024	-0.108	-0.252	-0.360	-0.312	0.264	1.764
12.000	0.000	0.000	0.000	0.014	0.000	-0.058	-0.202	-0.346	-0.317	0.173	1.771
14.000	0.000	0.000	0.000	0.000	0.000	-0.034	-0.168	-0.302	-0.353	0.118	1.764
16.000	0.000	0.000	0.000	0.000	0.019	-0.019	-0.115	-0.230	-0.384	0.096	1.747

RTICAL THERMAL STRESS-DIFFERENTIAL TEMP. CHANGE

ACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.070	0.232	0.432	0.633	0.814	0.964	1.020	1.164	1.219	1.252
3.000	0.000	0.100	0.320	0.569	0.797	0.981	1.113	1.198	1.247	1.269	1.272
4.000	0.000	0.129	0.397	0.680	0.916	1.087	1.193	1.249	1.270	1.271	1.261
5.000	0.000	0.156	0.465	0.770	1.005	1.157	1.237	1.269	1.271	1.261	1.247
6.000	0.000	0.183	0.527	0.847	1.072	1.203	1.260	1.272	1.264	1.249	1.236
8.000	0.000	0.232	0.633	0.964	1.164	1.252	1.272	1.263	1.246	1.231	1.222
10.000	0.000	0.277	0.722	1.049	1.216	1.270	1.268	1.248	1.232	1.222	1.219
12.000	0.000	0.320	0.797	1.113	1.247	1.272	1.257	1.237	1.224	1.219	1.218
14.000	0.000	0.359	0.861	1.159	1.263	1.269	1.247	1.229	1.220	1.218	1.219
16.000	0.000	0.397	0.916	1.193	1.270	1.261	1.238	1.224	1.218	1.218	1.219

FIXED-BASE CONDITION

HOOP THERMAL STRESS-INSIDE,AVERAGE CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.253	0.140	0.025	-0.100	-0.232	-0.371	-0.511	-0.647	-0.742	-0.777	-0.689
3.000	0.160	0.107	0.044	-0.035	-0.136	-0.263	-0.403	-0.568	-0.708	-0.774	-0.687
4.000	0.085	0.070	0.044	0.000	-0.048	-0.172	-0.319	-0.493	-0.667	-0.770	-0.685
5.000	0.037	0.042	0.038	0.021	-0.023	-0.106	-0.241	-0.427	-0.629	-0.763	-0.684
6.000	0.010	0.023	0.033	0.031	0.005	-0.061	-0.185	-0.368	-0.592	-0.756	-0.686
8.000	-0.011	0.005	0.020	0.031	0.030	-0.005	-0.101	-0.278	-0.522	-0.742	-0.682
10.000	-0.011	-0.002	0.012	0.023	0.035	0.021	-0.049	-0.206	-0.465	-0.726	-0.682
12.000	-0.006	-0.003	0.003	0.017	0.031	0.033	-0.014	-0.151	-0.405	-0.707	-0.681
14.000	-0.003	-0.002	0.000	0.007	0.022	0.034	0.005	-0.105	-0.359	-0.685	-0.682
16.000	0.000	-0.001	-0.001	0.003	0.018	0.029	0.019	-0.066	-0.319	-0.662	-0.686

HOOP THERMAL STRESS-OUTSIDE,AVERAGE CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.253	0.148	0.057	-0.042	-0.156	-0.295	-0.461	-0.663	-0.886	-1.113	-1.311
3.000	0.160	0.117	0.078	0.031	-0.040	-0.145	-0.300	-0.514	-0.776	-1.064	-1.313
4.000	0.085	0.076	0.070	0.058	0.042	-0.054	-0.139	-0.401	-0.689	-1.020	-1.315
5.000	0.037	0.046	0.056	0.063	0.053	0.004	-0.109	-0.315	-0.613	-0.981	-1.316
6.000	0.010	0.025	0.043	0.059	0.063	0.033	-0.057	-0.244	-0.548	-0.946	-1.314
8.000	-0.011	0.005	0.024	0.041	0.058	0.057	0.007	-0.146	-0.440	-0.890	-1.318
10.000	-0.011	-0.002	0.002	0.023	0.043	0.059	0.041	-0.076	-0.353	-0.822	-1.318
12.000	-0.006	-0.003	0.003	0.011	0.031	0.053	0.053	-0.027	-0.291	-0.769	-1.319
14.000	-0.003	-0.002	0.000	0.007	0.022	0.046	0.055	0.003	-0.231	-0.727	-1.318
16.000	0.000	-0.001	-0.001	0.003	0.012	0.035	0.061	0.016	-0.181	-0.696	-1.314

FIXED-BASE CONDITION

HOOP THERMAL STRESS-INSIDE;DIFFERENTIAL CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.308	0.635	0.891	1.077	1.203	1.279	1.318	1.331	1.327	1.313	1.295
3.000	0.308	0.699	0.983	1.168	1.273	1.321	1.331	1.320	1.299	1.277	1.256
4.000	0.308	0.750	1.050	1.225	1.308	1.331	1.321	1.297	1.271	1.249	1.233
5.000	0.307	0.792	1.101	1.263	1.324	1.328	1.305	1.276	1.250	1.233	1.222
6.000	0.308	0.829	1.142	1.289	1.331	1.319	1.288	1.258	1.236	1.223	1.217
8.000	0.308	0.891	1.203	1.318	1.327	1.295	1.260	1.235	1.221	1.216	1.215
10.000	0.308	0.941	1.244	1.329	1.314	1.274	1.241	1.222	1.216	1.215	1.217
12.000	0.308	0.983	1.273	1.331	1.299	1.256	1.228	1.217	1.215	1.217	1.219
14.000	0.308	1.019	1.294	1.327	1.284	1.243	1.221	1.215	1.216	1.218	1.220
16.000	0.308	1.050	1.308	1.321	1.271	1.233	1.218	1.215	1.217	1.219	1.220

HOOP THERMAL STRESS-OUTSIDE;DIFFERENTIAL CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-1.693	-1.391	-1.194	-1.079	-1.026	-1.015	-1.030	-1.059	-1.093	-1.126	-1.156
3.000	-1.693	-1.338	-1.133	-1.038	-1.015	-1.033	-1.070	-1.112	-1.151	-1.181	-1.203
4.000	-1.693	-1.297	-1.094	-1.020	-1.023	-1.061	-1.109	-1.153	-1.187	-1.209	-1.222
5.000	-1.693	-1.265	-1.067	-1.015	-1.038	-1.089	-1.141	-1.182	-1.208	-1.222	-1.228
6.000	-1.693	-1.237	-1.048	-1.016	-1.056	-1.115	-1.166	-1.201	-1.220	-1.228	-1.229
8.000	-1.693	-1.193	-1.026	-1.030	-1.093	-1.156	-1.199	-1.221	-1.228	-1.228	-1.226
10.000	-1.693	-1.160	-1.017	-1.049	-1.124	-1.184	-1.216	-1.228	-1.228	-1.225	-1.223
12.000	-1.693	-1.133	-1.015	-1.070	-1.150	-1.203	-1.225	-1.229	-1.226	-1.223	-1.221
14.000	-1.693	-1.111	-1.017	-1.091	-1.171	-1.215	-1.228	-1.228	-1.224	-1.221	-1.220
16.000	-1.693	-1.094	-1.023	-1.109	-1.187	-1.222	-1.229	-1.226	-1.222	-1.220	-1.220

FIXED-BASE CONDITION

VERTICAL THERMAL STRESS-TOTAL EFFECTS

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.023	0.073	0.137	0.211	0.300	0.411	0.563	0.782	1.076	1.489
3.000	0.000	0.037	0.113	0.192	0.265	0.327	0.407	0.523	0.719	1.036	1.506
4.000	0.000	0.055	0.162	0.261	0.333	0.380	0.417	0.497	0.666	0.984	1.507
5.000	0.000	0.072	0.208	0.328	0.398	0.425	0.436	0.478	0.615	0.934	1.502
6.000	0.000	0.088	0.249	0.384	0.457	0.472	0.454	0.463	0.571	0.887	1.499
8.000	0.000	0.116	0.312	0.468	0.544	0.539	0.437	0.449	0.508	0.807	1.494
10.000	0.000	0.138	0.367	0.525	0.596	0.581	0.508	0.444	0.460	0.743	1.491
12.000	0.000	0.160	0.398	0.564	0.623	0.607	0.528	0.446	0.453	0.696	1.494
14.000	0.000	0.179	0.431	0.580	0.631	0.618	0.539	0.463	0.434	0.668	1.491
16.000	0.000	0.198	0.458	0.597	0.645	0.621	0.562	0.497	0.417	0.657	1.483

HOOP THERMAL STRESS-INSIDE;TOTAL EFFECTS

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.280	0.387	0.458	0.489	0.485	0.454	0.403	0.342	0.292	0.268	0.303
3.000	0.234	0.403	0.513	0.566	0.568	0.529	0.462	0.376	0.296	0.251	0.285
4.000	0.196	0.410	0.547	0.613	0.630	0.580	0.501	0.402	0.302	0.240	0.274
5.000	0.172	0.417	0.570	0.642	0.651	0.611	0.532	0.424	0.311	0.235	0.269
6.000	0.159	0.426	0.583	0.660	0.668	0.629	0.552	0.445	0.322	0.233	0.265
8.000	0.148	0.448	0.611	0.674	0.679	0.645	0.580	0.478	0.349	0.237	0.267
10.000	0.148	0.469	0.628	0.676	0.674	0.647	0.596	0.508	0.375	0.244	0.267
12.000	0.151	0.490	0.638	0.674	0.665	0.644	0.607	0.533	0.405	0.255	0.269
14.000	0.152	0.508	0.647	0.667	0.653	0.638	0.613	0.555	0.429	0.267	0.269
16.000	0.154	0.524	0.653	0.662	0.645	0.631	0.618	0.574	0.449	0.279	0.267

HOOP THERMAL STRESS-OUTSIDE;TOTAL EFFECTS

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-0.720	-0.621	-0.569	-0.561	-0.591	-0.655	-0.745	-0.861	-0.990	-1.120	-1.233
3.000	-0.767	-0.611	-0.528	-0.503	-0.527	-0.559	-0.685	-0.813	-0.963	-1.122	-1.258
4.000	-0.804	-0.610	-0.512	-0.481	-0.490	-0.558	-0.649	-0.777	-0.938	-1.115	-1.269
5.000	-0.828	-0.609	-0.506	-0.476	-0.493	-0.543	-0.625	-0.748	-0.911	-1.102	-1.272
6.000	-0.842	-0.606	-0.502	-0.479	-0.497	-0.541	-0.612	-0.722	-0.884	-1.087	-1.271
8.000	-0.852	-0.594	-0.501	-0.494	-0.518	-0.549	-0.596	-0.684	-0.834	-1.054	-1.272
10.000	-0.252	-0.581	-0.504	-0.513	-0.541	-0.562	-0.588	-0.652	-0.791	-1.023	-1.272
12.000	-0.850	-0.568	-0.506	-0.529	-0.560	-0.575	-0.583	-0.628	-0.759	-0.996	-1.270
14.000	-0.848	-0.557	-0.509	-0.542	-0.574	-0.584	-0.581	-0.612	-0.728	-0.974	-1.269
16.000	-0.847	-0.547	-0.512	-0.553	-0.588	-0.593	-0.584	-0.605	-0.701	-0.958	-1.267

10

SLIDING-BASE CONDITIONS

ERTICAL THERMAL STRESS – AVERAGE TEMPERATURE CHANGE

FACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

'ERTICAL THERMAL STRESS-DIFFERENTIAL TEMP. CHANGE

;FACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.032	0.068	0.176	0.292	0.377	0.407	0.377	0.292	0.176	0.068	0.032
3.000	0.052	0.149	0.346	0.547	0.689	0.742	0.689	0.547	0.346	0.149	0.052
4.000	0.041	0.181	0.447	0.709	0.889	0.954	0.839	0.709	0.447	0.181	0.041
5.000	0.027	0.198	0.516	0.819	1.022	1.093	1.022	0.819	0.516	0.198	0.027
6.000	0.016	0.212	0.571	0.899	1.113	1.186	1.113	0.899	0.571	0.212	0.016
8.000	0.002	0.243	0.659	1.007	1.216	1.283	1.216	1.007	0.659	0.243	0.002
10.000	-0.001	0.279	0.734	1.077	1.264	1.320	1.264	1.077	0.734	0.279	-0.001
12.000	-0.002	0.318	0.800	1.130	1.283	1.325	1.283	1.130	0.800	0.318	-0.002
14.000	-0.001	0.356	0.861	1.168	1.290	1.318	1.290	1.168	0.861	0.356	-0.001
16.000	-0.001	0.394	0.914	1.197	1.288	1.303	1.288	1.197	0.914	0.394	-0.001

SLIDING-BASE CONDITIONS

HOOP THERMAL STRESS-INSIDE, AVERAGE CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

HOOP THERMAL STRESS-OUTSIDE, AVERAGE CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

SLIDING-BASE CONDITION

HOOP THERMAL STRESS-INSIDE, DIFFERENTIAL CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.383	0.729	0.997	1.188	1.301	1.338	1.301	1.188	0.997	0.729	0.383
3.000	0.344	0.755	1.062	1.268	1.384	1.422	1.384	1.268	1.062	0.755	0.344
4.000	0.321	0.779	1.101	1.302	1.409	1.442	1.409	1.302	1.101	0.779	0.321
5.000	0.309	0.805	1.132	1.319	1.409	1.435	1.409	1.319	1.132	0.805	0.309
6.000	0.304	0.832	1.158	1.327	1.399	1.417	1.399	1.327	1.158	0.832	0.304
8.000	0.303	0.887	1.204	1.332	1.367	1.371	1.367	1.332	1.204	0.887	0.303
10.000	0.305	0.936	1.240	1.332	1.335	1.327	1.335	1.332	1.240	0.936	0.305
12.000	0.306	0.980	1.268	1.328	1.308	1.292	1.308	1.328	1.268	0.980	0.306
14.000	0.307	1.017	1.290	1.323	1.286	1.266	1.286	1.323	1.290	1.017	0.307
16.000	0.308	1.049	1.305	1.316	1.269	1.246	1.269	1.316	1.305	1.049	0.308

HOOP THERMAL STRESS-OUTSIDE, DIFFERENTIAL CHANGE

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-1.629	-1.297	-1.067	-0.918	-0.836	-0.809	-0.836	-0.918	-1.067	-1.297	-1.629
3.000	-1.676	-1.299	-1.064	-0.930	-0.865	-0.846	-0.865	-0.930	-1.064	-1.299	-1.676
4.000	-1.695	-1.287	-1.061	-0.954	-0.912	-0.902	-0.912	-0.954	-1.061	-1.287	-1.695
5.000	-1.701	-1.267	-1.055	-0.977	-0.960	-0.959	-0.960	-0.977	-1.055	-1.267	-1.701
6.000	-1.702	-1.245	-1.048	-0.997	-1.003	-1.011	-1.003	-0.997	-1.048	-1.245	-1.702
8.000	-1.699	-1.202	-1.034	-1.031	-1.072	-1.092	-1.072	-1.031	-1.034	-1.202	-1.699
10.000	-1.696	-1.165	-1.025	-1.057	-1.121	-1.149	-1.121	-1.057	-1.025	-1.165	-1.696
12.000	-1.694	-1.136	-1.021	-1.079	-1.155	-1.186	-1.155	-1.079	-1.021	-1.136	-1.694
14.000	-1.693	-1.112	-1.021	-1.098	-1.179	-1.209	-1.179	-1.098	-1.021	-1.112	-1.693
16.000	-1.693	-1.094	-1.025	-1.115	-1.196	-1.223	-1.196	-1.115	-1.025	-1.094	-1.693

SLIDING-BASE CONDITION

VERTICAL THERMAL STRESS-TOTAL EFFECTS

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.016	0.034	0.088	0.146	0.188	0.204	0.188	0.146	0.088	0.034	0.016
3.000	0.026	0.074	0.173	0.273	0.345	0.371	0.345	0.273	0.173	0.074	0.026
4.000	0.021	0.090	0.223	0.354	0.445	0.477	0.445	0.354	0.223	0.090	0.021
5.000	0.013	0.099	0.258	0.409	0.511	0.547	0.511	0.409	0.258	0.099	0.013
6.000	0.008	0.106	0.285	0.450	0.556	0.593	0.556	0.450	0.285	0.106	0.008
8.000	0.001	0.121	0.329	0.503	0.608	0.642	0.608	0.503	0.329	0.121	0.001
10.000	-0.001	0.140	0.367	0.539	0.632	0.660	0.632	0.539	0.367	0.140	-0.001
12.000	-0.001	0.159	0.400	0.565	0.642	0.662	0.642	0.565	0.400	0.159	-0.001
14.000	-0.001	0.178	0.431	0.584	0.645	0.659	0.645	0.584	0.431	0.178	-0.001
16.000	-0.001	0.197	0.457	0.598	0.644	0.651	0.644	0.598	0.457	0.197	-0.001

HOOP THERMAL STRESS-INSIDE, TOTAL EFFECTS

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	0.191	0.364	0.499	0.594	0.650	0.669	0.650	0.594	0.499	0.364	0.191
3.000	0.172	0.378	0.531	0.634	0.692	0.711	0.692	0.634	0.531	0.378	0.172
4.000	0.160	0.390	0.550	0.651	0.705	0.721	0.705	0.651	0.550	0.390	0.160
5.000	0.155	0.402	0.566	0.659	0.705	0.718	0.705	0.659	0.566	0.402	0.155
6.000	0.152	0.416	0.579	0.664	0.699	0.709	0.699	0.664	0.579	0.416	0.152
8.000	0.151	0.443	0.602	0.666	0.683	0.685	0.683	0.666	0.602	0.443	0.151
10.000	0.152	0.468	0.620	0.666	0.667	0.664	0.667	0.666	0.620	0.468	0.152
12.000	0.153	0.490	0.634	0.664	0.654	0.646	0.654	0.664	0.634	0.490	0.153
14.000	0.154	0.508	0.645	0.661	0.643	0.633	0.643	0.661	0.645	0.508	0.154
16.000	0.154	0.524	0.653	0.658	0.634	0.623	0.634	0.658	0.653	0.524	0.154

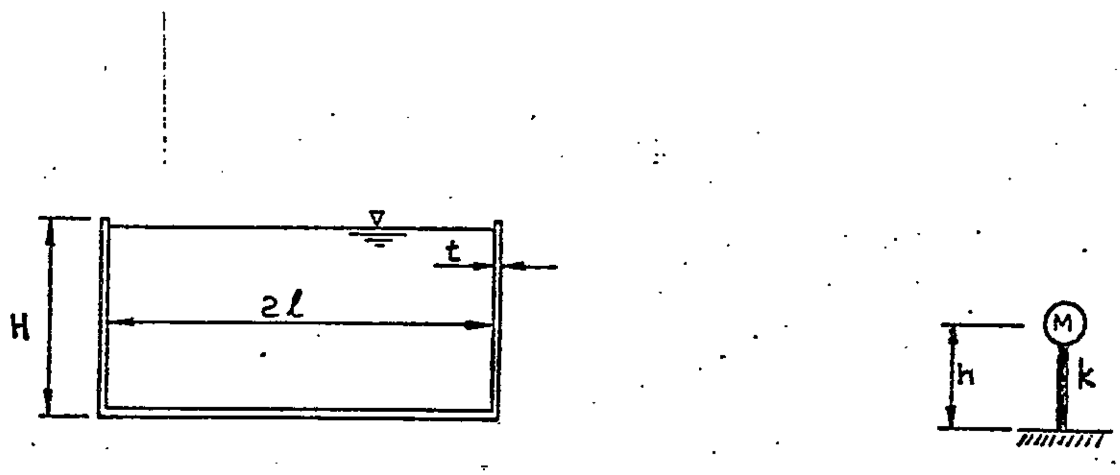
HOOP THERMAL STRESS-OUTSIDE, TOTAL EFFECTS

SFACT	TOP	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	BTM
2.000	-0.815	-0.648	-0.533	-0.459	-0.418	-0.405	-0.418	-0.459	-0.533	-0.648	-0.815
3.000	-0.838	-0.649	-0.532	-0.465	-0.432	-0.423	-0.432	-0.465	-0.532	-0.649	-0.838
4.000	-0.848	-0.643	-0.530	-0.477	-0.456	-0.451	-0.456	-0.477	-0.530	-0.643	-0.848
5.000	-0.851	-0.633	-0.527	-0.488	-0.480	-0.479	-0.480	-0.488	-0.527	-0.633	-0.851
6.000	-0.851	-0.622	-0.524	-0.499	-0.501	-0.505	-0.501	-0.499	-0.524	-0.622	-0.851
8.000	-0.849	-0.601	-0.517	-0.515	-0.536	-0.546	-0.536	-0.515	-0.517	-0.601	-0.849
10.000	-0.848	-0.583	-0.513	-0.529	-0.560	-0.574	-0.560	-0.529	-0.513	-0.583	-0.848
12.000	-0.847	-0.568	-0.510	-0.540	-0.578	-0.593	-0.578	-0.540	-0.510	-0.568	-0.847
14.000	-0.847	-0.556	-0.511	-0.549	-0.590	-0.605	-0.590	-0.549	-0.511	-0.556	-0.847
16.000	-0.846	-0.547	-0.512	-0.558	-0.598	-0.612	-0.598	-0.558	-0.512	-0.547	-0.846

APPENDIX B

FUNDAMENTAL PERIOD OF VIBRATION OF THE INERTIA COMPONENT OF A RECTANGULAR TANK (T_I)

T_I for an at-grade rectangular tank can be roughly estimated using a lumped mass/hypothetical column type model.



(a) Rectangular tank of width B (b) Hypothetical column model.

FIGURE B1: TANK/MODEL SYSTEM

The fundamental period T_I is given by:

$$T_I = 2\pi\sqrt{M/k}$$

where $M = M_W' + M_I'$

M_W' = mass of water per unit width (kg/m)

$= Ht\rho_c$ (ρ_c = density of wall)

M_I' = half of the impulsive mass of contained liquid per unit width (kg/m)

$$= \frac{(W_I)}{W} H \ell \rho_L \quad (\rho_L = \text{density of liquid})$$

$\frac{W_I}{W}$ can be obtained from figure 2.2

k = flexural stiffness of a unit width cantilever

$$= \frac{E}{4}\left(\frac{t}{h}\right)^3 \quad (N/m)$$

h = effective height of M

$$= \frac{h_W M_W' + h_I M_I'}{M}$$

h_W = centre of gravity of wall = H/2

h_I = height of impulsive mass

($\frac{h_I}{H}$ can be obtained from figure 2.2)

WORKED EXAMPLE

liquid = water ρ_L = 1000 kg/m^3

concrete tank ρ_c = 2400 kg/m^3

2ℓ = 20 m H = 6.0 m t = 300 m E_c = 25 GPa

$$M_W' = Ht\rho_c = 6 \times .3 \times 2400 = 4300 \text{ kg/m}$$

$$M_I' = \frac{(W_I)}{(W)} H \ell \rho_L = 0.36 \times 6 \times 10 \times 1000 = 21\ 600 \text{ kg/m}$$

$$\frac{W_I}{W} = 0.36 \text{ for } \frac{\ell}{H} = 1.67 \text{ in figure 2.2)}$$

$$M = M_W' + M_I = 25\ 900 \text{ kg/m}$$

$$h_I = 0.385H \text{ (from figure 2.2 for } \frac{\ell}{H} = 1.67)$$

$$= 0.385 \times 6 = 2.31$$

$$h_W = 0.5H = 3.0 \text{ m}$$

$$h = \frac{h_W M_W' + h_I M_I'}{M} = \frac{3 \times 4300 + 2.31 \times 21\ 600}{25\ 900} = 2.42 \text{ m}$$

$$k = \frac{E}{4}\left(\frac{t}{h}\right)^3 = \frac{25 \times 10^9}{4} \cdot \left(\frac{.3}{2.42}\right)^3 = 11.9 \times 10^6 \text{ N/m}$$

$$T_I = 2\pi\sqrt{\frac{M}{k}} = 2\pi\sqrt{\frac{25\ 900}{11.9 \times 10^6}} = \underline{0.29 \text{ seconds}}$$

ETABS 84

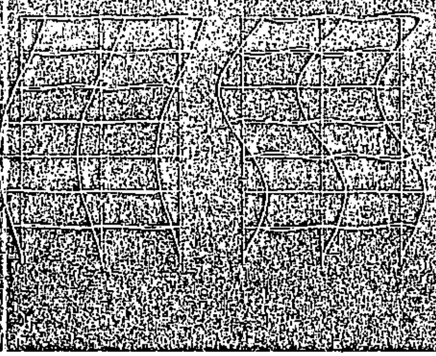
Three-Dimensional Static and Dynamic Analysis of Building Systems

APPLICATIONS
- Symmetric and non symmetric, planar and non planar, three dimensional frame and shear wall structures
- Buildings with cantilevers, setbacks, mezzanine floors, or atriums
- Complete three dimensional common column compatibility for highrise tubular structures
- Braced frames (X-braced, K-braced, A-frames, eccentrically braced frames)
- Shear wall-frame interaction
- Shear wall, pier-spandrel systems
- Discontinuous shear walls
- Shear walls with arbitrary openings
- Spring support conditions
- Pin ended columns and beams

STATIC ANALYSIS
- Vertical Load Analysis
  1. Superimposed (point or distributed) dead or live load analysis
  2. Self weight analysis
- Lateral Load Analysis
  1. Automated UBC Seismic Loading
  2. Automated ATC Seismic Loading
  3. Automated UBC Wind Loading
  4. General lateral loading
- Static displacements and member forces

DYNAMIC ANALYSIS
- Periods and mode shapes
- Multidirectional lateral earthquake Response Spectrum analysis
- Lateral Time History analysis
- Dynamic displacements and member forces

GENERAL FEATURES
- Large capacity
- Building oriented output formats, story-by-story
- Free format input
- Easy to use
- Extensive generation options
- Rigid floor diaphragms and no floor diaphragms
- Built-in AISC property table, automated rectangular section property calculation
- Load combinations (statics and dynamics)
- Good Diagnostics



SOLUTION PROCEDURES
- Blocked active column equation solution
- Geometric stiffness modification for direct (non-iterative) inclusion of P-delta effects
- Automated frame joint stiffness corrections due to finite beam and column dimensions
- Axial, shear and bending deformations included in all members

DESIGN POSTPROCESSORS

Postprocessors for the AISC stress check of steel buildings and for the ACI design of concrete structures (June 1984) are available under separate license.

[redacted]

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REGISTRANTS FOR SEMINAR "DESIGN OF CONCRETE STRUCTURES"
TO BE HELD AT THE UNIVERSITY OF CANTERBURY 9-11 JULY 1986

M.J. Abernethy	MWD, Wellington
W.R. Andrew	Worseldine & Wells, Nelson
S. Ashby	Firth Stresscrete, Panmure, Auckland
M. Bloxham	Murray North & Partners, Hamilton
H. Brookie	Brickell Moss Raines & Stevens, Wellington
G. Bycroft	Lamont Bycroft & Partners, Wanganui
P. Charlton	Beca Carter Hollings & Ferner, Auckland
S. Clark	Murray North & Partners, Rotorua
I. Connor	Morrison Cooper & Partners, Christchurch
G. Dempsey	Frederick Sheppard & Partners, Wellington
F. Dennis	R.W. Morris, Christchurch
M.A. Gordon	Waimairi District Council, Christchurch
I.W. Goss	R.W. Morris & Assoc., Christchurch
L. Greenfield	Christchurch City Council
P. Greenway	Frederick Sheppard & Partners, Christchurch
R. Gross	Auckland City Council
S. Guillemin	Blewett Waite Jeffs and Carter, Auckland
D. Harding	Alan Reay Consultants, Christchurch
G.M. Hughson	Wellington City Council (Works Department)
R.S. Jarrat	Smith Leuchars Ltd, Wellington
D. Kirkland	Holmes Wood Poole & Johnstone, New Plymouth
B.S. Lobb	Milward Fougere Finlay & Lobb, Timaru
S. Macdonald	Macdonald & Associates, Blenheim
M. Marinan	Spencer Holmes Miller Partners Ltd, Wellington
A. McGaughran	Timaru City Council
N.R. Melhop	Consulting Engineer, Christchurch
R. Modgill	New Plymouth City Council
N.J. Morgan	Structon Group, Wellington
A. Mortimer	Brickell Moss Raines & Stevens, Auckland
C. Mundy	MWD, Ballantrae Place, Wellington
A.F.N. Nightingale	Consulting Engineer, Lower Hutt
W. Page	Sanders Lane & Page, Nelson
D. Preston	Royds Garden, Christchurch
J. Smialowski	Bruce Henderson Consultants, Tauranga
M.H. Smith	F.R. Smith & Associates, Christchurch.
D.C. Stewart	MWD, Auckland
T.B. Steven	Murray North & Partners, Auckland
R.D. Sullivan	Consulting Engineer, Christchurch
P. Thompson	Civil Engineer, Lower Hutt
R.J. Twiname	D.C. Airey & Partners, Takapuna, Auckland
N.M. Taylor	Royds Garden, Christchurch
C. Welling	Hutt Valley Drainage Board, Lower Hutt
J. Wemyss	Sanders Lane & Page, Nelson

Paulay : Concrete Shear Walls

① Design for Flexure

(i) $0.7/f_y < \rho_e < 16/f_y$

(ii) when $v_i > 0.3\sqrt{f'_c}$ use two layers.

(iii) $d_b < b_w/10$.

(iv) $b > l_n/10$ in end region only.

- out of plane buckling can occur at relatively low loads.

(v) use load redistribution if desired $\Delta M \leq 0.3 M_{max}$.

(vi) From $\frac{P_u}{A_g f'_c}$ find ϕ.

$0.7 \leq \phi \leq 0.9$ but by 10.5 $\phi = 0.9$.

(viii) $M_i = \frac{M_u}{\phi}$, $P_i = \frac{P_u}{\phi}$ $e = \frac{M_i}{P_i}$

(viii) With trial & error, find (c) and area of vertical bars.

(ix) find flexural overstrength of base section

M_o based on $f_y^x = 1.25 \times 275$
$= 1.4 \times 380$

(x) $\phi_o = \frac{M_o}{M_{code}}$ for each wall

Check for strength $\phi_o > 1.25/\phi = 1.39$
$> 1.40/\phi = 1.56$

(xi) compare $\underline{c}$ and $\underline{c_c}$

If $c_c = 0.10\,\phi_o\, S\, l_w > c$ no confinement
$< c$ confine concrete.

- when $c > c_c$
- confined length $\geq \alpha c$
- $\alpha = 1 - 0.7\frac{c_c}{c} \geq 0.5$.
- Also use ϕ_o^x = global overstrength factor
  ie whole bldg not just this wall

(xii) curtailment of vertical flexural reinforcement
- will not be same as 3rd order parabola implied by Code loads

if cut of steel to match envelope, p hinge can occur anywhere, so need extra confinement steel throughout the wall – cheaper to use extra vert steel to force p'hinge to occur at base, ⇒ $v_c > 0$ in body of wall, less shear reinf reqd in body of wall.

(5) Define end region $l_p > l_w$
$> h_w/6$.

(6) Confine end region.

(i) when $c > c_c$ confine outer ½

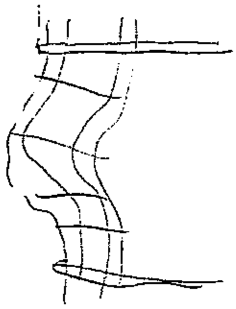
(ii) $A_{sh} = 0{\cdot}12\ s_h h'' \frac{f'_c}{f_{yh}}\left(0{\cdot}5 + {\cdot}9\frac{c}{l_w}\right)$

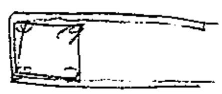
or Eq 10-5

(iii) $A_{te} = \frac{\Sigma A_b f_y}{16 f_{yt}} \cdot \frac{s_h}{100}$ whenever $\varepsilon_c > \varepsilon_y$

and when $p_e = \frac{A_s}{b s_v} > \frac{2}{f_y}$.

don't space horiz shear reif too far $\not> 300$? b_w?





(7) Design for shear.

(i) $V_{wall} = \omega_v \phi_o V_{code} < 4 V_{code}/S$.

(ii) $v_i = \frac{V_{wall}}{.8 l_w b_w}$ $\phi = 1.0$ for capacity design procedures.

(iii) $v_i < (0.3\phi_o S + .16)\sqrt{f'_c} \leq 0.9\sqrt{f'_c}$

if $S = 0.8$, $\phi_o = 1.4$ $v_i \leq 0.4\sqrt{f'_c}$
necessary to avoid shear failure.

→ this is very often critical in a c'lever shear wall.

iv $v_c = 0.6\sqrt{\frac{P_e}{A_g}}$

(v) $A_v = v_i - v_c\ b_w s / f_y$

may use deformed bars with appropriate hooks.

(8) Splices

(i) not more than 33% over l_p - unavoidable to have them but should be staggered

(ii) stagger by more than $2l_d$

(iii) provide transv. reinf ties if $d_b \geq 16$

$\frac{A_{tr}}{s} \geq \frac{8d_b}{f_y t}$

shear-friction to transfer tie force & keep cracks down

(9) Elastic regions of wall

(i) Check flexural capacity with P_i against linear moment diagram

(ii) reduce wall thickness

$l_n/b > 10$

$v_i < 0.2 f'_c$ or 6.0 MPa.

(iii) reduce shear reinforcement

$v_c \geq 0.27\sqrt{f'_c}$

dramatic shortf. reduction normal.

(iv) reduce transverse ties.

$\rho_e > 2/f_y$ & $f_y > f_s > 0.5 f_y$ } $S_h < 16 d_b$, $< 48 d_s$ in.

(v) use nominal ties when $f'_s < 0.5 f_y$
(construction ties) and when $s \geq 4$.

STRUCTURAL WALLS : A SUMMARY

Intent of design
- control of failure modes
  - upper storey shear
  - concrete crushing
  - diag. tension failure
  - sliding shear c.j.s.
- good energy dissipation.

Coupled Walls
- critical areas (27) (21)
- coupling beams (58) (60)
- application. (123)

FLOORSLABS & DIAPHRAGMS

Prof. Park.

Flat slabs · should not be used for seismic resistance
· DCTLL shear already is high
- difficult to get ductile connections, strength degrades rapidly

Many approaches to design.

① 11.5 Elastic thin plate theory.
need elastic computer programs - useful for complex systems
computer will give M_x, M_y & M_{xy}
M_{xy} should not be ignored.
provide M_x capacity = $M_x + |M_{xy}|$ as in commentary.
simple procedures for allocating reinforcement.

② Moment coefficient tables.
∴ elastic theory gives large moment variations & it is difficult to model these with steel cutoffs.
moment coeff[t] tables allowed for some redistribution.

③ Limit Design
- look at ultimate strength
(a) yield line theory - Johansson
(b) strip method - Hillerborg
Hillerborg better for design Johansson for analysis.
- not told what ratios to use for m_x' & m_x
if ratios used diff to elastic, can get significant cracking under service loads.
- code says somewhere between 1 & 2 (C 11.6), slabs are very tolerant & wide variations will give satisfactory designs.

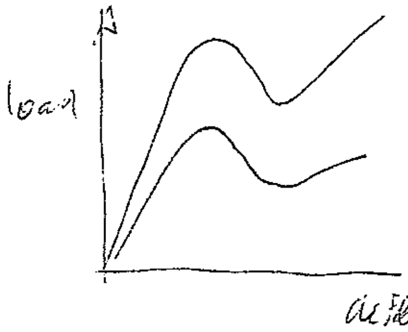
④ Code Bending Moment Coefficients
- are derived from yield line theory.

Direct Design Method
Equivalent frame Method } used widely in USA.

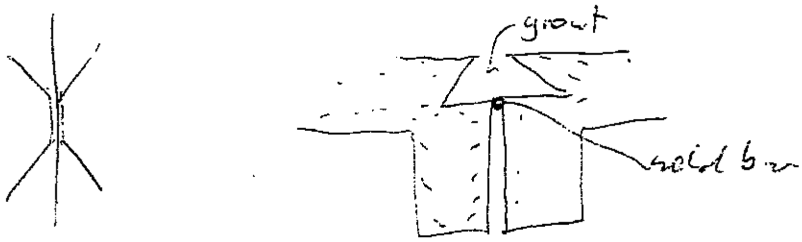
Section 11 of "Applications of NZCOP ... 3101:1982" includes design examples by all methods & shows variation in design BM's which result on p31.

indicates these are tolerant animals. - don't worry about these different answers.
but how much difference in service load deflection expected!

actual ultimate slab capacities often higher than yield line theory in a building, due to membrane action due to support, tensile and compressive, from the surrounding diaphragm. Deflections may be 2 or 3 times slab depth.



Stressconcrete: Auckland : connection between p/c slabs



can prestress slabs to lower levels to minimise differential deflection. Can prop the slabs to equal deflections before topping.

Applications of Design Charts & Computer Programs for Reinf. Conc.

(1) NZ Reinf Conc Handbook — L Caerty

Column Design section : new steel strength coming with revised charts to follow

Development section : note amendment to 3101: 5.3.7.2(c) which gives l_d less than (a), (b) etc sometimes.

(2) Minicomputer programs — Leathem Andrews.

Concol 1 — Handbook holder $175

Concol 2

Direct design for rectangular, circular section

input column dimensions, M, P, gives As output.

(3) David King program : Concrete Column Programs.

NZCRA trying to decide what to do with it.

Uniaxial, Biaxial, square, rectangular, circular.

Can use the "Mander model" for stress / strain curve instead of ACI stress model. – gives less steel, smaller columns. for confined concrete, not unconfined conc.

Can use any column section.

(4) ETABS — Jeff Clendon HW/POJ.

developed in Berkeley Calif.

HW/PO J are selling $7000-$9000 in

640 KB RAM Harddisc & floppy disc drive.

- not an interactive system
- use editor to read data file.
- gives "echo" file & then runs it

Frames, sh walls, 25 storeys   running time ½ hr – 4 hrs. (25 storey)

IBM PC 20mB hard disk - storage problem so not enough space to queue jobs up overnight. needs 5mB for to run completely.

Post programs programs from USA available for steel design, concrete walls - will design beams, walls & columns based on America Codes.

ETABS '84
SUPER ETABS has more options.

Post processor plotter is 3-D.

SAP '80 also available.
Flat slab programme SAFE which does Finite element slab analysis.

(5) ~~EXEC~~ Philip. Thompson
XSEC

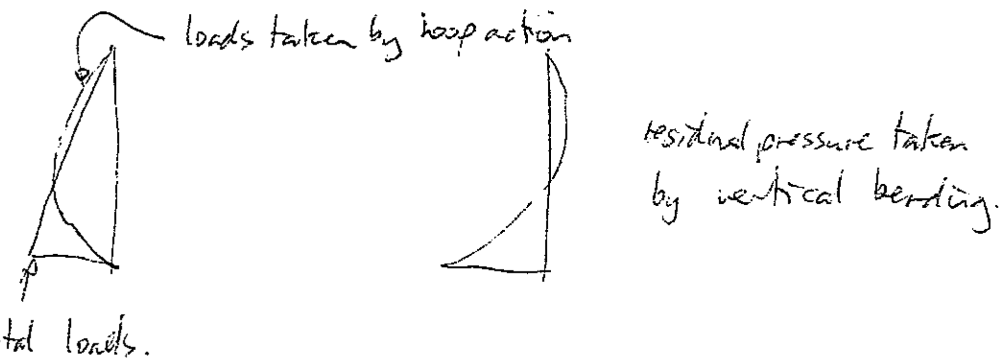
Concrete cross section design
decide on a cross section, programme will put in steel.

Structures for storage of liquids — Priestley.

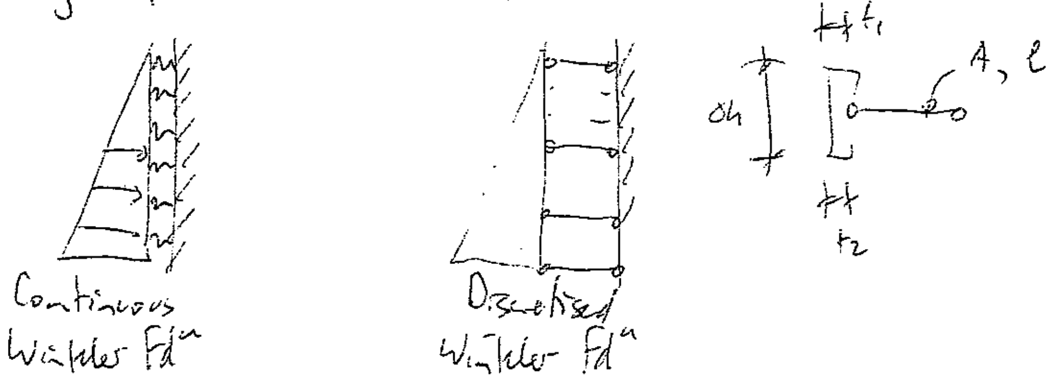
- Free sliding base walls often used overseas, not in NZ as seismic loading. all ~~stresses~~ pressures carried by Hoop Action.

- pin or fixed base deflection pattern is modified giving vertical curvatures and BM's. Usually refer to tables to get design B.M's. Resist part of load by hoop tension.

Fluid loads:



- can do specific design on computer, rather than use tables. design for a Winkler foundation.



-frame analogy up to 10 struts.

$$\frac{A_i}{\ell_i} = \frac{t_1+t_2}{2a^2}\cdot\delta h$$

Vertical BM's $= M_v$, $f_v = \frac{\pm 6}{t^2}\cdot M_v$

Thermal loads:

(i) inside temp. reasonably stable = water temp.
outside ≈ radiant heat on sunny side

Pump hot water into storage tank, inside temp will be higher than outside which gives opposite temp gradient.

Temp change induces deformations, which are restrained & so stresses are induced
stresses are dependent on Material of construction,

$$f = E\,\alpha\,\theta\,[I, A]$$

MoE. ↗ coeff of expt ↑ temp change ↖ section properties ↖

Thermal Moment : $M = \frac{E}{I_c} I \,\phi\left(\frac{\alpha\theta}{t}\right)$

(I) $I = I_{gross}$ or $I_{cracked}$
& $I_{cracked}$ varies with temperature, history, position.

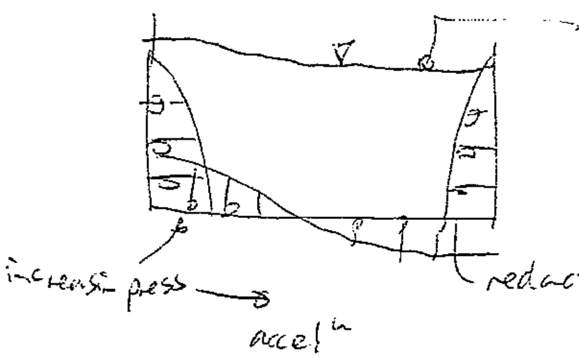
Fig 3 provides crude approximation for BM relief by cracking.

(E) $E_c = 4700\sqrt{f'_c}$ is a lower bound.
$= 5500\sqrt{f'_c}$ more likely, with $f'_c \approx 1.4 \times$ spec. after 2yrs.

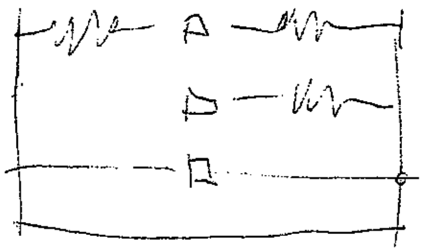
(α) 6×10^{-6} per °C to 14×10^{-6} per °C

Shrinkage & Swelling
- cause stresses, somewhat offset by creep as they occur slowly.
some experts say OK to ignore any nett effect.

Seismic response
based on current recommendations of NZSEE.



→ change in water level & consequent change in base pressure



upper masses not rigidly linked to walls
various sloshing modes

lower masses effectively bound to move with walls - rigidly linked

Table gives ratios for fluid masses and heights.

large periods typical for natural sloshing, 6 to 12 secs Fig 6.
advantage to use new code to use lower spectral accel?

Practicalities

- need to develop shear and uplift forces at wall base for seismic loadings.

- thermal loads can be greater than water loads
- seismic loads also of similar to water load, but normally higher at the top of the wall.

- more sensible load combinations & load cases included in the code

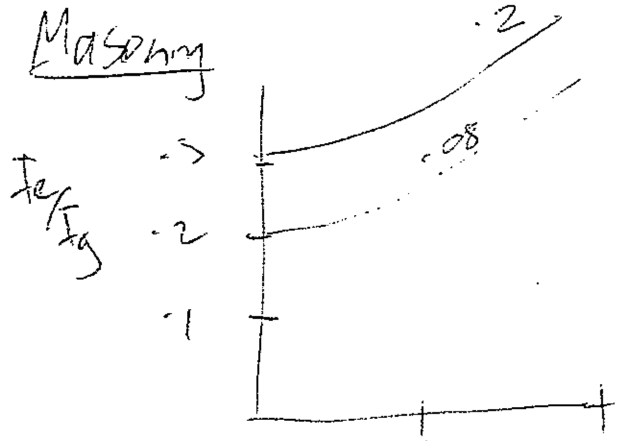
- prestressing: stress wall to move it in at base, then fix it at base. with creep, top will continue to move but base will not → vertical BM & some reduction in prestress at the base

- may consider insulating the tank to reduce thermal stresses imposed

Masonry

Priestley

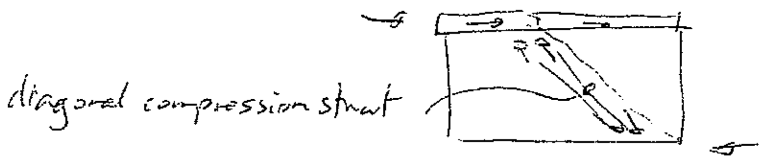
(12)



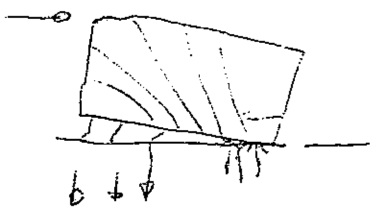
$P_u = \frac{0.1\ f}{f'_c\ A_g}$

Walls of limited ductility

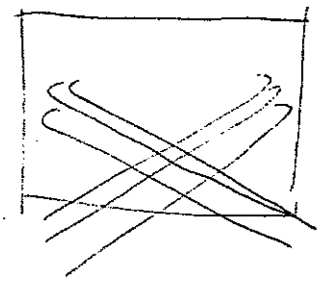
- squat shear walls
  how is shear introduced along the top edge?

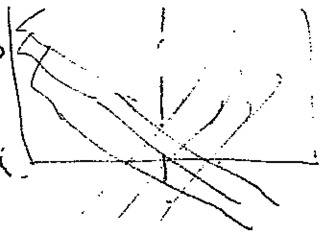


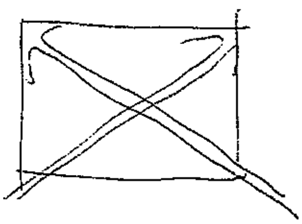
put enough shear reinf in ∴ it will fail in flexure



flanged wall not necessarily better as extra flexural steel means if fails in shear.

this  simply use diagonal reinforcement.

~~or~~ this is better → 
as it doesn't add
to the flexural strength.

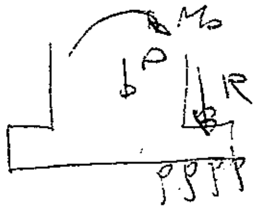
or
this 

this adds to flexural strength also. ??

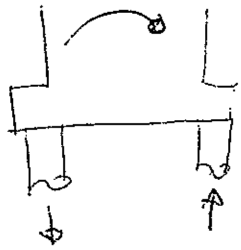
Foundations

Paulay

if reaction is within depth of wall can use pads



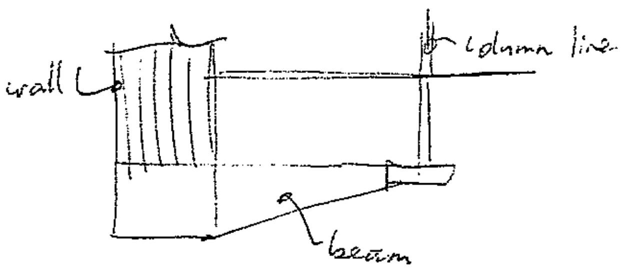
if have piles these can resist moment



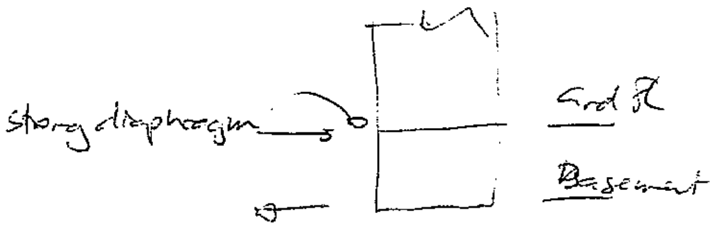
decide to have yielding in superstructure, calculate flexural overstrength of structure & apply to foundations, so that it ideally remains elastic.

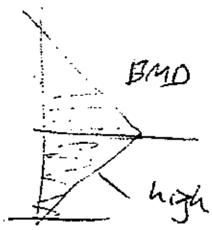
Doesn't like idea of yielding occurring under a moderate EQ, where it is difficult to repair or inspect.

if on a boundary, hook in first row of columns



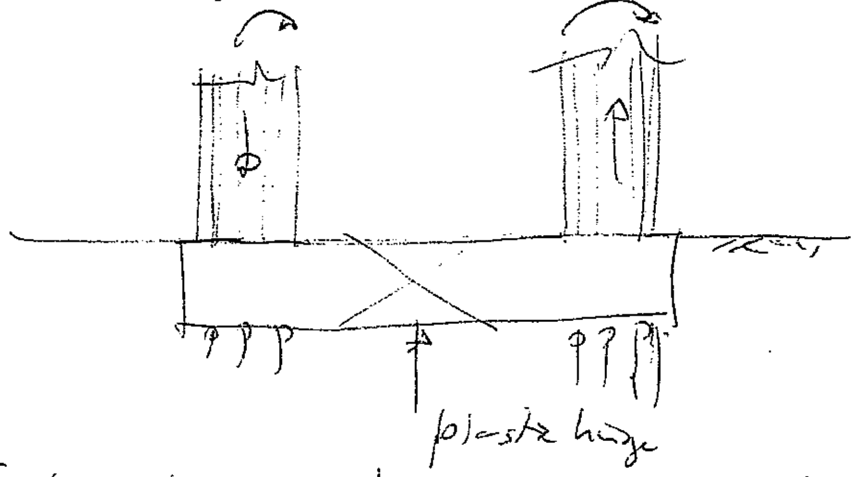
can get fixity from a basement wall





high BM gradient
high shear
maybe diagonal reinforcement

possible to design hinges into the foundation beam.



This footing designed to yield – no ductility design needed in the wall.

wouldn't like to see it sitting permanently below water table.

Repair

don't expect major damage after moderate EQ.

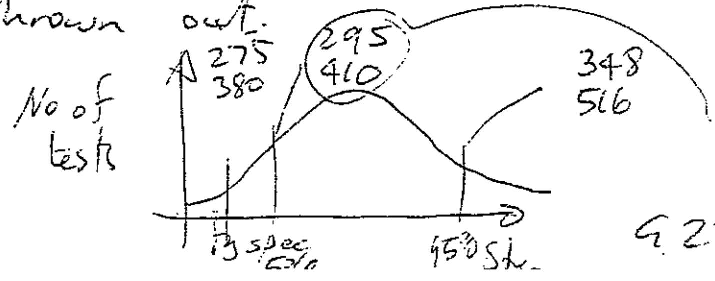
bodies wrecked in lab have been given $\mu = 6$ or 10 so no wonder they are in a bad way.

After EQ may have moderate cracks, - for a large EQ, may have some cover concrete spalled off.

This can readily be repaired, epoxied. Quite often the really wrecked specimens in the lab have been repaired, and retested. They have performed again better than the originals!

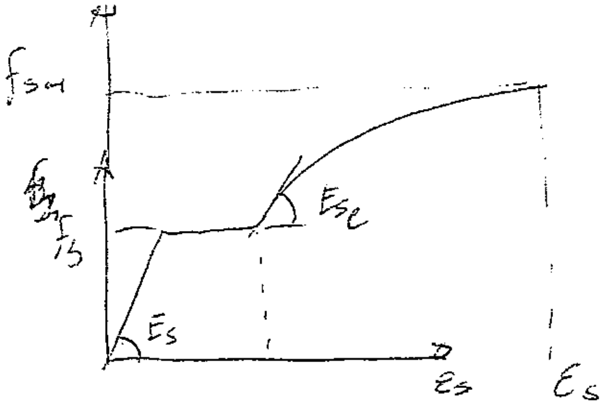
New steel tests

Pacific St. all steel is tested, all is at least that specified, that below is thrown out.



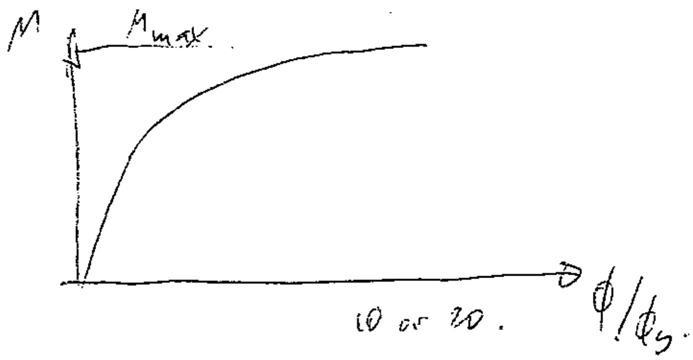
9.275

Paulay says could use 5% as characteristic strength not the absolute minimum.



actual values of yield strength.

Can do moment-curvature analysis.



Flexural overstrength factor M_o $\frac{M_{max}}{M_{ideal}}$

based on 5% lower-tail f_y
ie 295 or 410.

for G 275 $M_o = 1.19$ at $\phi/\phi_y = 10$.
$= 1.30$ at $\phi/\phi_y = 15$.

G 380 $M_o = 1.52$ at $\phi/\phi_y = 10$
$= 1.60$ at $\phi/\phi_y = 15$.

HERA New Zealand Heavy Engineering Research Association

BULLETIN

HERA INFORMATION CENTRE

★**NEW RELEASE**

New Zealand Heavy Engineering Industry Achievements in Recent New Zealand Major Development Projects. This publication reflects the development of heavy engineering since 1981 and illustrates industry accomplishments during recent major projects. A complementary publication **"Heavy Engineering Facilities in New Zealand"** provides information of company capabilities and main products, manufacturing resources and contact in formation. Both publications are now available FREE from HERA. **Ref. 2**

Publications from Australian Institute of Steel Construction.

Safe load tables for structural steel — metric units 5th ed. $36.
The tables cover the range of structural sections generally available and commonly used taken from the Australian Standards:—
AS1131: Dimensions of hot-rolled structural sections.
AS1163: Steel hollow sections. **Ref. 3**

Source book for the Australian steel structures code AS1250 by M.C. Lay. 3rd ed. $18.
The purpose of this book is to provide back-ground data, explanations and reference sources for those Rules of the Australian Steel Structures Code, AS1250, whose origins and reasons for inclusion are not immediately apparent. **Ref. 4**

Standarized structural connections. 3rd ed. $52.50
The object of this manual is to provide a rationalized approach to the design, detailing and fabrication at structural steelwork connections. **Ref. 5**

Design of structural connections by T.J. Hogan and I.R. Thomas. 2nd ed. $40
This manual sets out a design basis for a range of commonly used structural steel connections. **Ref. 6**

Bolting of steel structures by A. Firkins, and T.J. Hogan. 2nd ed. $9
This publication is intended to provide a state-of-the-art summary of the use of bolts in steel structures. **Ref. 7**

PERSONAL COMPUTER SOFTWARE FOR STRUCTURAL ENGINEERS

HERA is currently negotiating with two Californian programmers involved in the development of structural software packages. The intention is to adapt existing programmes for use in New Zealand and a major announcement on this subject can be expected shortly.

BACKGROUND

A designer in California has access to software for individual steel member design, to the relevant U.S. codes. This software includes capacity design methods for high ductility demand structure (e.g. ductile moment resisting frames).
Furthermore the strong column weak beam philosophy is generally adhered to, with emphasis placed on flexural type inelastic structural systems rather than shear type. Thus their seismic design philosophy is similar to ours, with differences in detail only.
Also researchers in the United States are at the forefront in development of seismic design procedures and recommendations for braced frames. Therefore design procedures and recommendations for braced frames are largely based on U.S. recommendations. Thus a programme written for braced frame member design to U.S. codes and design practice, as well as for ductile rigid frame member design, is likely to be adaptable to New Zealand codes and design practice without too much difficultly.

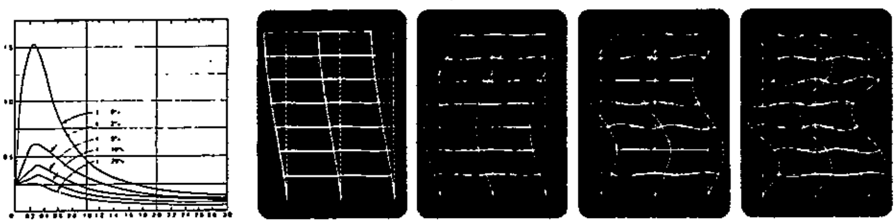
U.S. DEVELOPMENTS

Computers and Structures Inc. of Berkeley, California are regarded as market leaders pertaining to state-of-the-art structural analysis and design software for personal computers.
They have developed SAP80 a general purpose static and dynamic finite element analysis programme, and ETABS, a special purpose static and dynamic building analysis programme.
Along with the recent advances in microprocessor technology there has been a dramatic increase in the use of the SAP80 and ETABS programmes. These software packages are now routinely used on personal computers for the analysis of a wide variety of small and large systems, having anywhere from 10 to 10,000 equations.
Part of the ETABS System is a steel design postprocessor programme called STEELER. The programme performs a member design analysis based upon AISC recommendations and incorporates current United States seismic design philosophy in the calculations. It can be used on a very wide variety of structures, including concentrically and eccentrically braced frames, ductile moment resisting frames and would have immediate application to all types of steel structures currently designed in New Zealand.
The programme operates by taking the output files from ETABS. It then performs the design process, based on the user specified member properties used in the analysis of the frame. The output is a stress check of all members, plus any stiffener, doubler and continuity plate requirements. The user can then change members if desired and re-run STEELER without having to re-run the ETABS programme.
The hardware requirements to operate the system are as follows:—

OPERATING SYSTEM MS/DOS 2.0 or later by Microsoft

COMPUTER (Minimum requirements for CSI programmes)IBM, PC/AT, IBMPC/XT or IBM PC with one 360k floppy disk drive and one 10,20 or 30 megabyte hard disk; or IBM compatible machine.
RAM (Random Access Memory) — 640K
Math coprocessor chip — 8087 for IBM PC/XT,80287 for PC/AT

PRINTER
Programme output can be printed on any printer which is compatible with your computer. A hard copy of the screen graphics produced by the graphics programme may be obtained on dot matrix printers which are "IBM graphics compatible". **Ref. 8**



Response Spectrum Analysis

Compumod Pty. Ltd.
COMPUTER MODELLING IN ENGINEERING

10th Floor
20 Martin Place
Sydney 2000
Tel: (02) 27 7405

Box 4853
GPO
Sydney 2001
Telex: AA70537

Release of MSC/pal 2

MSC announces MSC/pal 2, a three-dimensional finite element analysis software package for engineering applications that runs on IBM PC XTs, ATs, and compatible. MSC/pal 2 is an advanced version of MSC/pal, the popular PC-based finite element analysis software released by MSC in the latter half of 1984.

MSC/pal 2 performs static and dynamic analysis for models with up to 1000 grid points. Its element library includes: quadrilateral and triangular plate elements; several types of straight and curved beams; discrete springs, masses, and dampers; shear panels; and plane-stress and plane-strain two-dimensional plate elements. Its solution capabilities include statics, normal modes, transient response, and frequency response analysis. A bandwidth minimizer is integrated into MSC/pal 2 to reduce memory requirements and to increase execution speed. Double-precision arithmetic in all operations ensures numerical accuracy.

Interactive post-processing capabilities include: stress and displacement output for all solution types; structure geometry plots with rotation, scaling, and element shrink features; deformed structure plots with stress and displacement contours and animation; and XY plots for dynamic displacements, velocities, accelerations, and response ratios.

Advanced features with the MSC/pal 2 package allow the user to graphically sort static stress output for easy visual identification of highly-stressed elements, as well as file input/output features to allow transferring MSC/pal 2 data to/from external programs. The ability to convert a model file from MSC/pal 2 to MSC/NASTRAN format is included to allow running a model on larger computers; all static and dynamic load files may be translated as well. The entire MSC/NASTRAN data file is generated, including the Executive Control, Case Control, and Bulk Data decks, making MSC/pal 2 an ideal educational tool for learning MSC/NASTRAN.

Machine requirements include an IBM (or IBM-compatible) PC XT or AT (a hard disk is required), PC DOS 2.0 (or higher), 512K memory, and an 8087 (80287 for the AT) numeric coprocessor chip. An IBM (or compatible) color graphics card is required for graphical output, and an Epson (or compatible) graphics printer is required for hard-copy of graphics. Any dot matrix, letter quality, or laser printer is acceptable for hard-copy of tabular output.

1986 Modular Microbee: more memory for less money

microbee

The advantages of a modular approach

Now more than ever before, Australia's own Microbee personal computers provide the most practical, low cost solution for the needs of Australian business, education and the home.

Our range of 1986 Microbees are the culmination of over four years of continuous product development. They provide increased memory capacity, coupled with much greater flexibility in selecting the optimum disk drive configuration for your needs. The Premium Series models also provide enhanced graphics, additional keys for cursor control plus inbuilt colour video and Videotex communications.

Whether you're looking for a floppy-disk based workstation, a ROM-based entry level system or a professional hard disk system, our 1986 Microbees are the ideal choice. And they're made right here in Australia!

Premium or standard models

With most configurations, you have a choice between the Standard model Microbee computer and its Premium Series counterpart. The Premium Series models offer all of the features of Standard models, plus

- Videotex and colour video as standard
- Greatly enhanced graphics
- 4 extra keys for cursor control
- Volume control for the internal speaker

Having Videotex facilities built-in makes the Premium Series Microbees ideal for use as an intelligent data terminal. All you need for data communications is a low cost modem like our Bee Modem.

The greatly expanded colour graphics capability of our Premium Series models provides them with the ability to display a total of 131,072 individually controllable picture elements or 'pixels' – many more than competing machines. This makes them highly compatible with the latest ultra-friendly graphics driven software.

Disk systems now with 128K of RAM

Thanks to falling chip prices, our new disk-based models now all come with a full 128K bytes of RAM memory – for the same price as previous 64K models. As well as providing increased storage capacity for programs and data, the increased memory size speeds up operation by reducing the need for transfers of information to and from disk.

14-DAY MONEY BACK TRIAL OFFER

Microbee understands that buying a computer is a major investment. Our 14-day money back offer is designed so you can make that investment with full confidence. In the unlikely event that your new Microbee system doesn't meet your needs, you can return it to us within 14 days of purchase. Providing it's in original condition, we'll give you a full refund.

There couldn't be a safer way to invest in a computer!

Updating your existing 'Bee

If you're already the proud owner of an earlier Microbee, we offer you the opportunity to update your system to one of the latest Premium Series models at a very attractive price.

For any earlier Microbee computer console, we'll give you $200 trade-in allowance on any of our New Premium Series and for floppy disk units, we'll give you a further $100 (single drive) or $150 (dual drive) off the purchase of a new disk system of the same or higher level.

This means you can save up to $300 on a new Premium Series computer and disk unit – and still keep your existing monitor, printer and other peripherals!

Microbee values its loyal customers, and always makes it possible for them to update their computer to the latest system at low cost.

Buy only the software you really need

Everyone's needs are different when it comes to applications software for your computer. Although some may need the full range of word processor, spreadsheet, and various other packages, others may need only one or two of these – at least initially.

With the new 1986 Microbee disk systems you have the freedom to buy only the applications software you want. Included with the systems themselves are the operating system plus basic utilities for disk formatting and copying, data communications, setting up the system and so on. There's also Microworld BASIC and a small "get you started" word processor – but no expensive bundled packages. Then we can provide all the other applications packages you're likely to need.

Wordstar: World's most widely used word processor

For word processing, Microbee offers the full Wordstar Professional Pack from Micropro International. Wordstar is generally acknowledged as the world's most widely used word processing package, combining ease of use with great flexibility. In the Professional Pack it comes complete with Spellstar, Mailmerge and StarIndex – plus all necessary manuals.

Multiplan: Easy to use spreadsheet

Microbee offers Microsoft Corporation's Multiplan spreadsheet package, one of the most widely used and very easy to use. Manipulate data in worksheets of up to 255 rows by 63 columns, which may be edited and used to look at "what if" possibilities. Sheets can be saved on disk, printed out and linked together to have data "rippling through" from level to level. A very powerful business tool.

Programming languages

For those who wish to write their own software programs, Microbee offers a range of powerful programming languages. These include Microsoft BASIC, Borland Turbo Pascal and Research Logo.

ORDER FORM

Use this form to select: 1. The computer configuration you want
2. Your choice of disk unit and
3. Hardware options and software required.

If you are updating from an existing Microbee, you can calculate your trade-in allowance and deduct it from the total of (1) (2) & (3).

	ITEM	CAT. NO.	UNIT PRICE	MY ORDER
1.	COMPUTERS			
	Standard 32K ROM	970.500	$ 499	$
	Premium 32K ROM	970.550	$ 599	$
	Standard 128K RAM	970.520	$ 499	$
	Premium 128K RAM	970.570	$ 599	$
	Premium 128k RAM HD unit	970.580	$ 599	$
2.	FLOPPY DISK UNITS			
	3.5 inch single FD unit	970.600	$ 599	$
	5.25 inch single FD unit	970.620	$ 599	$
	3.5 inch dual FD unit	970.610	$ 799	$
	5.25 inch dual FD unit	970.630	$ 799	$
	HD Unit 10M + 1.6M	970.640	$2900	$

	ITEM	CAT. NO.	UNIT PRICE	MY ORDER
3.	OPTIONS			
	ESE Mono monitor (Green)	990.125	$ 189	$
	ESE mono monitor (Amber)	990.126	$ 199	$
	Hi-resolution RGB colour mon.	990.127	$ 849	$
	Thompson/ESE RGB col. mon.	990.128	$ 549	$
	DP-100 NLQ dot matrix printer	990.079	$ 499	$
	300/300 + 1200/75 Bee Modem	970.032	$ 299	$
	Wordstar Professional WP Pack	119.061/159.061	$ 135	$
	Microsoft Multiplan Spreadsheet	112.111/152.111	$ 225	$
	Microsoft BASIC Interpreter	112.101/152.101	$ 225	$
	Turbo Pascal	117.191/157.191	$ 169	$

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BANKCARD NO. [ ][ ][ ][ ] [ ][ ][ ] [ ][ ][ ][ ] [ ][ ][ ][ ][ ][ ]
Signature ____________________

Please allow 14 days for order processing and 7 days for delivery depending on location.
☐ 14 DAY MONEY BACK TRIAL OFFER

My order TOTAL: $______
UPDATE ALLOWANCE
I will be trading in
a ________ Microbee Console ________ $200
and a ________ in disk system ________ ______
($100 for single/$150 for dual)
TOTAL TRADE IN ALLOWANCE $______
(Deduct from Order Sales total)
PAY ONLY $______

Please make cheques payable to Microbee Systems Ltd & mail order to:
Microbee Systems New Zealand Limited
P.O. Box 71107 Rosebank, Auckland 7.
OR take your order to one of the addresses shown above.

MICROBEE SYSTEMS N.Z. LTD.
438 B ROSEBANK RD,
AVONDALE,
AUCKLAND, 7.
TELEPHONE: 881-139

P.O. BOX 26045,
NEWLANDS,
WELLINGTON, 4.
TELEPHONE 785-548

microbee

Choice of disk units

Our disk-based systems now offer you a much broader range of choices when it comes to disk drive configuration. With floppy disk systems you can choose either the established 5.25 inch (130mm) disk format, or the new 3.5 inch (90mm) format. In either format you can also choose between a single disk drive, for economy, or dual drives for greater operating speed and convenience.

All disk systems come with the widely-used CP/M operating system, enhanced with Microbee's unique icon-menu shells and operating utilities. This makes them particularly easy to use, even for those with no previous computer experience.

Four different video monitors

With the 1986 Microbees you have a choice of no less than four different video monitors. Our two ESE monochrome monitors provide excellent performance at an economy price, with either green or amber display as desired. For more demanding applications we offer a higher resolution monochrome (green) monitor, and a high resolution RGB colour monitor with 18 MHz bandwidth and 0.38 mm pitch CRT.

Built-in serial & parallel ports

All Microbee models feature built-in serial communications and parallel data interface ports as standard – unlike many other computers where these facilities are expensive "optional extras". This means that basic peripherals like printers and data modems plug directly in.

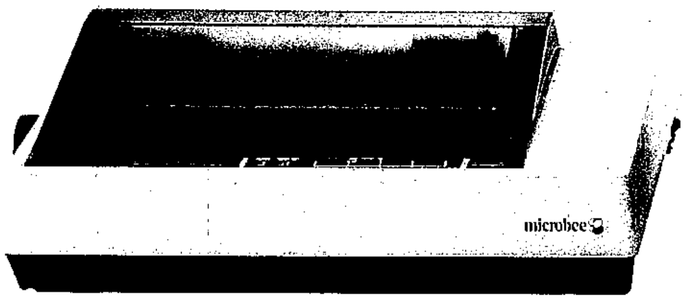
The Microbee serial port is fully software controlled, and operates over a wide range of data communication rates.

VIDEO MONITOR

Four different monitors are available: monochrome in green or amber, high resolution green monochrome, and high resolution RGB colour.

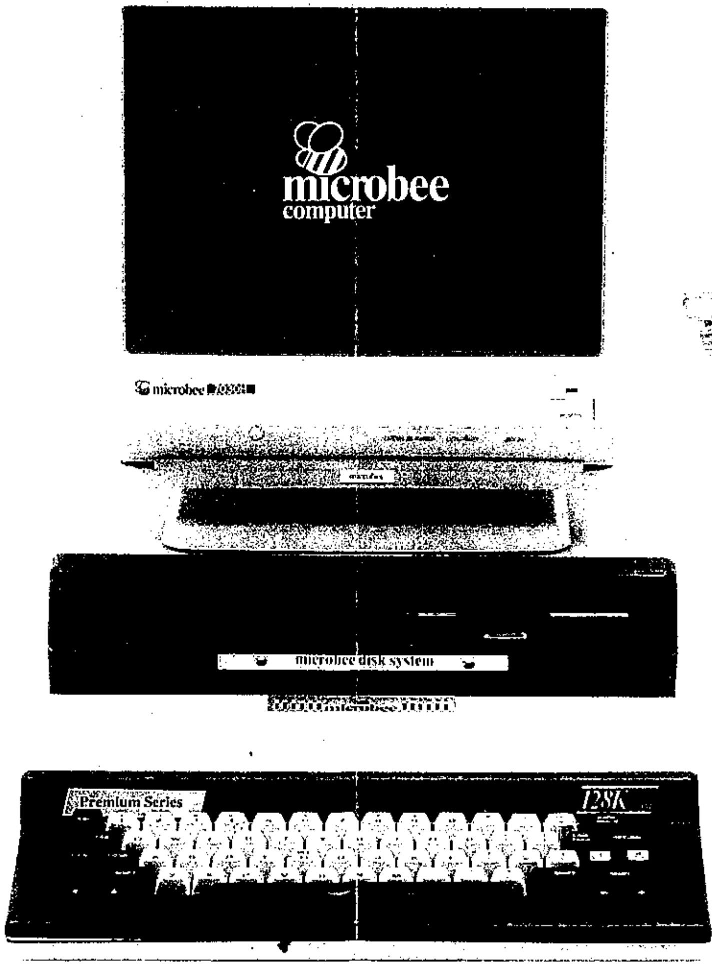
NLQ PRINTER

Microbee's optional low cost DP-100 printer offers both 100 cps draft quality printing and multi-pass NLQ (near letter quality) printing, making it ideal for most applications.



Optional NLQ printer

Microbee's optional low cost DP-100 printer offers the speed of dot-matrix draft quality printing at 100 cps, coupled with the ability to produce multi-pass NLQ (near letter quality) output for word processing. It makes an excellent choice for most home and office applications.



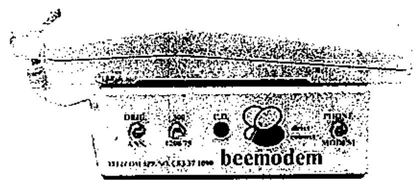
Data modem with 'phone

Another important option available with all 1986 Microbees is our Bee Modem, for communication with other computers and data services via the telephone network. Data communications extends the uses and power of your computer system enormously, giving you access to a vast range of databases and message handling resources.

The Bee Modem is a highly reliable direct connect, full duplex unit operating at both commonly used data rates: 300/300 bps and 1200/75 bps. It comes complete with a push-button electronic telephone, and plugs directly into a standard telephone socket. It is fully-authorised by Telecom (C83-37-1090).

MODEM

Microbee's optional low cost Bee Modem lets you access data services like Videotex and Information Express, plus Telememo and other message handling services.



DISK UNIT

Floppy disk units are available in either 5.25-in or 3.5-in formats, and with either single or dual drives. Formatted storage capacity is 400K bytes per diskette, in each case. The alternative Hard Disk model provides 20M of formatted storage, with an 800K 5.25-in floppy drive for backup.

COMPUTER

You have a choice of five: Premium or Standard 32K ROM-based models, Premium or Standard 128K floppy disk based models, or the 128K Premium Hard Disk model.

Entry level ROM models

For those wishing to become familiar with computers at low cost, Microbee offers ROM-based models (both Standard and Premium). These feature 32K of user RAM plus 88K of ROM containing a suite of built-in programs: a word processor, a spreadsheet, a database manager, our Telcom communications package, a desk calculator program, business graphics and the MicroWorld BASIC programming language.

These models feature battery backup to preserve your files in memory, but programs and data may also be stored using a normal cassette tape recorder.

'Top of the line' 20M hard disk model

Our Premium Series 128K Hard Disk System is designed for applications which need much more storage capacity than is available with floppy disks. It comes with 20 megabytes of formatted storage, together with a high-density 5.25 inch floppy disk drive for data input and backing up.

Unlike many other hard disk systems, the Microbee Hard Disk System is exceptionally easy to use. The operating system and utilities feature the same friendly icon-menu shells used on the floppy disk system, making the system very suitable for use by non-technical people.

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SECTION 1
COMMENTARY

C1.1.3
For liquids having a detrimental effect on concrete, appropriate special precautions may include the provision of linings impervious to the liquid to be contained. Particular attention should be paid to any joints in the lining which must remain impervious for the life of the structure, for much damage and even collapse can occur before a leak is detected. In tanks where such impervious lingings are used, the allowable stresses of the alternative design method of NZS 3101 may be used in lieu of those of this Code.

Code of practice for
CONCRETE STRUCTURES FOR THE STORAGE OF LIQUIDS

SECTION 1
GENERAL

1.1
Scope

1.1.1
This New Zealand Standard Code of Practice sets out requirements for concrete, including cement mortar, structures for the storage of liquids and is approved as a means of compliance with the relevant requirements of NZS 1900.

1.1.2
Sections 1,2,3, and 9 of this Standard apply to all concrete structures for the storage of liquids. The remaining sections apply specifically to structures constructed in either reinforced concrete, prestressed concrete or cement mortar.

1.1.3
Appropriate special precautions additional to the requirements of this Standard shall be taken where the structure is to contain highly penetrating liquids, or liquids having detrimental effect on concrete.

1.1.4
Swimming pools shall in addition comply with NZS 4441.

1.2
Interpretation

1.2.1
In this Standard the word "shall" indicates a requirement that is

to be adopted in order to comply with the Standard, while the word "should" indicates a recommended practice.

1.2.2
Cross-references to other clauses or subdivisions within this Standard quote the number only, for example: "... as required by 6.4.1".

1.2.3
The full titles of reference documents cited in this Standard are given in the "List of related documents" immediately preceding the Foreword.

1.3
Support structures

1.3.1
Support structures for elevated tanks shall be designed to the requirements of NZS 4203 together with an appropriate design code for the material to be used.

1.4
Roofs

1.4.1
Every structure containing water intended for potable water supply without further treatment shall have a roof designed to prevent contamination of its contents.

1.5
Notation

1.5.1
In this Standard, symbols shall have the following meanings, pro-

vided that other meanings for symbols that are defined immediately adjacent to formulae or diagrams shall apply in relation to those formulae or diagrams only:

a	Radius of circular tank, m
A	The design loads, or their related internal moments and forces, resulting from a combination of group A loads.
A_o	Peak horizontal ground acceleration coefficient (see 2.2.9.3)
b	One half of the width of a rectangular tank, perpendicular to the direction being considered, m
B	The design loads, or their related internal moments and forces, resulting from a combination of group B loads.
C	Temporary loads, or their related internal moments and forces, occurring during construction.
C_C	The horizontal convective seismic coefficient.
C_I	The horizontal impulsive seismic coefficient.
C_T	The axial force induced in a circular wall due to a temperature gradient through the wall, N.
D	Dead loads, or their related internal moments and forces.
E	Earthquake loads, or their related internal moments and forces.
E_c	Modulus of elasticity of concrete, MPa.

E_H The horizontal component of E.

E_s Modulus of elasticity of prestressed reinforcement, MPa.

E_V The vertical component of E.

EP The loads, or their related moments and forces, resulting from backfilled earth pressure.

f'_c Specified compressive strength of concrete or cement mortar, MPa.

$\sqrt{f'_c}$ The square root of the specified compressive strength of concrete or cement mortar, MPa.

f_i The fibre stress on the inside face of a tank wall or roof resulting from a temperature gradiant through the wall or roof, MPa.

f_o The fibre stress on the outside face of a tank wall or roof resulting from a temperature gradient through the wall or roof, MPa.

f_y The specified yield strength of non prestressed reinforcement, MPa.

F Liquid load, the internal static force exerted by the stored contents when filled to overflow level or its related moments and forces.

F_T A reduction factor modifying temperature stresses for section rigidity (see C2.2.8.1).

h The height above the base of the wall of the centre of gravity of the combined (convective plus implusive) horizontal seismic force exerted by the contained liquid, m.

h_C The height of the centre of gravity of the convective horizontal force exerted by the contained liquid, m.

h_I The height of the centre of gravity of the impulsive horizontal force exerted by the contained liquid, m.

h_S The height to the centre of gravity of the tank shell, m.

H The height of the tank wall to the surface level of the liquid, m.

ℓ One half of the length of a rectangular tank in the direction being considered, m.

L Live loads, or their related internal moments and forces.

M The total overturning moment acting on the foundation or support structure, N m.

M_B Overturning moment on the floor of a tank resulting from hydrodynamic seismic pressures, N m.

M_{BC} The convective component of M_B, N m.

M_{BI} The impulsive component of M_B, N m.

M_W Overturning moment acting at the foot of the tank wall, N m.

M_{WC} The convective component of M_W, N m.

M_{WI} The impulsive component of M_W, N m.

M_T The moment induced in an element due to a temperature gradient across the element, N m.

P_b The hydrodynamic seismic pressure of the contained liquid at the bottom of the tank wall, MPa.

P_t The hydrodynamic seismic pressure of the contained liquid at the top of the tank wall, MPa.

P The prestressing load, or its related moments and forces, provided to counteract forces and deformations induced by other loads.

r The radius of a dome roof, m.

R Risk factor as defined in NZS 4203.

S_h Loads, or their related moments and forces, resulting from shrinkage.

S_w Loads, or their related moments and forces, resulting from swelling.

t The thickness of the wall or roof, m.

T Loads or their related moments and forces resulting from variations in temperature.

T_C The natural period of the first mode of sloshing, s.

T_I Fundamental period of oscillation of a tank, s.

V_C The convective component of V_H, N.

V_H The total horizontal seismic force per unit length exerted on the wall of a tank, N.

V_I The impulsive component of VH, N.

w Maximum load (dead plus live) for unit area on dome roof, N/m2.

W Wind loads, or their related internal moments and forces.

W_C The equivalent weight of the convective contents, N.

W_I The equivalent weight of the impulsive contents, N.

W_S The weight of the tank shell, including the roof if connected to the walls, N.

α (alpha) Linear coefficient at thermal expansion.

β (beta) A constant depending on the shape of a tank used in determining seismic pressure (see 2.2.9.5).

γ_L (gamma) The weight density of liquid, kN/m3.

ε_{sh} (epsilon) Shrinkage strain.

ε_{sw} Swelling strain.

θ (theta) Change in temperature at a point in a tank wall, °C.

1.6 Definitions

1.6.1
The following terms are defined for general use in this Code. Specialized definitions appear in individual sections.

ADMIXTURE. A material other than portland cement, aggregate, or water, added to concrete to modify its properties.

AGGREGATE. Inert material which is mixed with portland cement and water to produce concrete.

ANCHORAGE. See Section 5 of NZS 3101. Also, the means by which the prestress force is permanently transferred to the concrete.

BONDED TENDON. Prestressing tendon that is bonded to concrete either directly or through grouting.

CEMENT MORTAR. A mixture of portland cement or any other hydraulic cement, sand and water.

CONCRETE. A mixture of portland cement or any other hydraulic cement, sand, coarse aggregate and water.

CONSTRUCTION JOINT. An intentional joint in concrete work detailed to ensure adequate strength and servicability.

DUCTILITY. The ability to a structure or member to undergo repeated and reversing inelastic deflections beyond the point of first yield while maintaining a substantial proportion of its initial maximum load carrying capacity.

ELASTIC ANALYSIS. Analysis based on the assumption of linear relationships between stress and strain for reinforcement and concrete.

ELEVATED TANK. A liquid retaining structure which is elevated above grade by a support structure.

ENGINEER. The Local Authority's principal Engineer who shall be registered under the Engineers Registration Act 1924 and who is the holder of a current annual practicing certificate; his deputy or assistant appointed by the Local Authority to control the erection of buildings.

FREEBOARD. Additional wall height above tank overflow level.

MOVEMENT JOINT. A specially formed joint intended to accommodate relative movement between adjoining parts of a structure.

PARTIALLY PRESTRESSED CONCRETE. Concrete in which prestressing is used to provide part of the reinforcement requirement, and where cracking of the concrete is permitted under specified design load combinations.

POST-TENSIONING. A method of prestressing in which the tendons are tensioned after the concrete has hardened.

PRECAST CONCRETE. A concrete element cast in other than its final position in the structure.

PRESTRESSED CONCRETE. Concrete in which there has been introduced internal stresses of such magnitude and distribution that the stresses resulting from loads are counteracted to a desired degree.

PRE-TENSIONING. A method of prestressing in which the tendons are tensioned before the concrete is placed.

REINFORCED CONCRETE. Concrete containing steel reinforcement, and designed and detailed so that the two materials act together in resisting forces.

REINFORCEMENT, DEFORMED. Round deformed reinforcing steel conforming to NZS 3402P.

REINFORCEMENT, MESH. Welded mesh formed of hard-drawn round steel bars, welded at intersections, conforming to NZS 3422.

REINFORCEMENT, PLAIN. Round, smooth reinforcing steel conforming to NZS 3402P.

SHRINKAGE. Time dependent compressive strain of concrete, resulting from loss of moisture to the surrounding environment.

SPECIFIED COMPRESSIVE STRENGTH OF CONCRETE. A singular value of strength normally at age 28 days unless stated otherwise, denoted by the symbl f'_c which classifies a concrete as to its strength class for purposes of design and construction. It is that level of compressive strength which meets the production standards required by Section 6 of NZS 3109.

SUPPORT STRUCTURE. A structure supporting a liquid retaining structure at a required height above grade.

SWELLING. Tensile strain of concrete resulting from absorption of moisture.

TANK. Structure designed for the containment and storage of liquids.

TENDON. Steel elements such as wire, cable, bar, rod, or strand used to impart prestress to concrete when the element is tensioned.

UNBONDED TENDONS. Tendons which are not bonded to the concrete either directly or through grouting.

WATERSTOP. Continuous impervious membrane placed across a construction joint to inhibit moisture flow through the joint.

SECTION 2
COMMENTARY

C2.1
Design method
Ultimate strength design methods are considered to be of doubtful relevance to design of concrete storage tanks, as standard load factors specified in NZS 4203 were developed without considering the special types and intensities of loading to which concrete storage tanks are subjected. Servicability of the structure under the design loads is of paramount importance. A working stress approach is therefore required. Provided allowable stress levels are not exceeded under the service load combinations of clause 2.3.2 of this code, ultimate behaviour should be satisfactory.

All connections of wall with floor and roof exert some measure of restraint that affects wall design. Particular attention should be given to the translation and rotation restraint the extent of which varies depending on the type of joint: fixed, hinged or free. Actual details may exhibit properties of one or more types at different stages of construction. Design calculations are generally based on the assumption that joints are either fully fixed or completely unrestrained against rotation and/or displacement. In reality such things as friction, soil movement and foundation deformation result in an intermediate degree of fixity, the implications of which may need to be assessed. Because the effects of edge restraint are of fundamental importance to tank design, detailed information can usually be found in almost any publication on tank design. BS 5337 is a suitable reference on these matters.

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SECTION 2
2.0 DESIGN CONSIDERATIONS

2.1
Design method
Tank design shall be based on elastic analysis methods and shall take into account effects of all loads and conditions of edge restraint at wall junctions with floor and roof. Maximum stresses shall not exceed allowable working stresses given in Sections 5, 6 or 7 as appropriate.

C2.2
Design loads
There are two broad types of loading: that resulting from the application of forces and that resulting from the application of deformation (strain). In a tank the force loading is exemplified by contained fluid pressure; the strain induced loads are temperature and shrinkage.

C2.2.1
Dead load
Dead load definition differs from that given in NZS 4203 which includes such things as earth pressure, temperature effects and construction loads. This code deals with these aspects separately.

C2.2.3
Backfill loading
Earth pressures should take into account the distribution and characteristics of the backfill (symmetrical or asymmetrical) and should be determined by rational methods of soil mechanics based on foundations and soil investigations.

C2.2.4
Construction loads
Examples of construction loads are the stacking, lifting and propping of precast panels.

C2.2.5
Liquid load
Overflow systems usually require a surcharge to initiate operation. In most cases, the surcharge is small and can safely be ignored, that is, the overflow level is taken as the inlet level of the overflow pipe. Where the surcharge is likely to be large, the additional head should be included as a class B design load.

2.2
Design loads
A number of loadings and load combinations need to be considered in the design of a tank. In general, the nature of the loads and their intensity should be as prescribed by NZS 4203. This Section deals with special considerations peculiar to tanks.

2.2.1
Dead load (D)
Dead load is the self weight of the structure including that of equipment and appurtenances.

2.2.2
Live load (L)
Live loads are the superimposed loads such as snow, people, vehicles, plant etc. and shall conform to the requirements of NZS 4203.

2.2.3
Backfill loading (EP)
Backfill loading is the external pressure exerted by backfill, including the effects of surcharge and water table.

2.3.4
Construction loads (C)
Construction loads are temporary loads resulting from equipment, materials and methods to be used during construction.

2.2.5
Liquid load (F)
Liquid load is the internal static pressure exerted by the stored contents when filled to the overflow level.

C2.2.6
Prestress

In circular tanks circumferential prestressing forces are sometimes used to offset content pressures. In domed tanks, circumferential prestress may be applied at the dome ring to counterbalance the horizontal component of the thrust due to dome dead and live load.

Vertical prestressing may be provided to counteract bending stresses in the vertical direction.

C2.2.7
Wind

In general the effects of wind load on ground supported tanks are not severe and will not govern design. The possibility of suction over the entire shell surface should be recognised in the design of dome roofs.

C2.2.8
Temperature

C2.2.8.1
Wall

The temperature distributions of figure 2.1 are appropriate for tanks subject to direct solar radiation. Special consideration should be given to shielded or buried tanks for which lower design gradients will generally be applicable.

Design tables have been developedl (and included as Appendix A) suitable for the common range of circular tanks. Thermal stresses are presented in the form:

$$f_t = CE_c \alpha \beta \qquad \text{.......... (Eq. C2.1)}$$

2.2.6
Prestress (P)

This is the prestressing force provided to counteract forces and deformation induced by other loads.

2.2.7
Wind (W)

Maximum design loading for wind shall conform to the requirements of NZS 4203.

2.2.8
Temperature (T)

2.2.8.1
Wall

Except as required by clause 2.2.8.3 and in the absence of thermal analysis based on known local meteorological conditions, the wall shall be designed for the temperature distributions defined by figure 2.1.

where C is a coefficient dependent on the shape factor H2/Dt, position up the wall, the degree of base restraint, and type of temperature effect (average, differential or combined). The tables are in terms of surface stresses assuming uncracked section stiffness. If the tank is designed on the basis of a cracked section, then these stresses should be modified by a reduction factor F_T to account for the reduced section rigidity that accompanies cracking. The amount of reduction depends on the extent of tension stiffening in the concrete, the reinforcing content and the degree of inplane force.

A method of evaluating F_t for reinforced concrete design is described in Section 5 and the implications for partially prestressed design discussed in Section 6. There is no reduction (F_t = 1.0) for fully prestressed design as the concrete is assumed to remain uncracked.

Temperature moments and axial forces are given by:

$$M_T = F_t\frac{(f_i - f_o)}{2}\cdot\frac{t^2}{6} \quad \text{............ (Eq. C2-2)}$$

$$C_T = F_t\frac{(f_i + f_o)}{2}\cdot t \quad \text{............ (Eq. C2-3)}$$

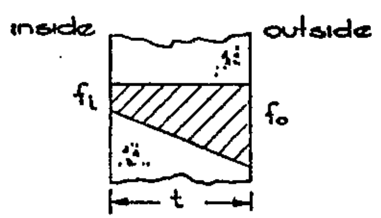


FIGURE C2.1: THERMAL STRESSES

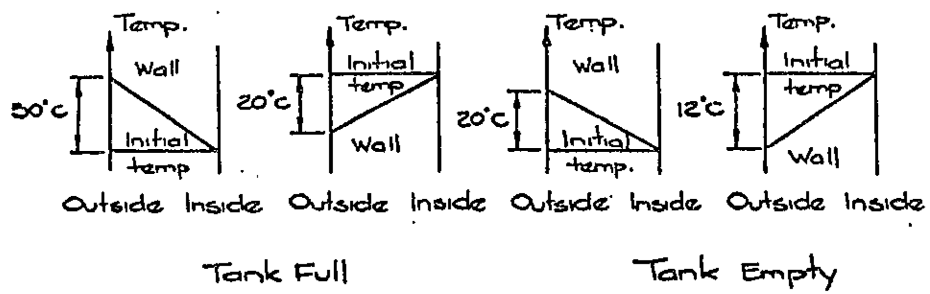


FIGURE 2.1: TEMPERATURE DISTRIBUTIONS IN TANK WALLS

C2.2.8.2

Roof

Temperature effects on the roof are in general small and of little significance unless the roof is cast monolithically with the wall.

In snow regions, appropriate consideration should be given to the effects of a reverse temperature gradient (outside colder than inside).

C2.2.8.

Special structures

The evaluation of the thermal response of concrete tanks subjected to unusual temperature conditions may require sophisticated heat flow analyses. A description of the procedures involved in such analyses is given in reference (2.1). In addition to final temperature gradients, transient thermal conditions may also be an important design consideration. For example, the temporary thermal gradients resulting from the rapid filling of a tank with a relatively hot liquid may be more severe than the eventual equilibrium condition.

C2.2.9

Earthquake

The earthquake analysis should include the inertia forces generated by the horizontal acceleration of the structure itself and the hydrodynamic forces generated by the horizontal acceleration of the contained liquid. The hydrodynamic pressure of the contained liquid can be considered to consist of two components: the "impulsive" (inertia) pressure caused by the portion of the liquid accelerating with the tank and the "convective" pressure caused by the portion of liquid oscillating in the tank.

2.2.8.2

Roof

Allowance shall be made for stresses and movements resulting from:

(a) a ± 20 °C variation in the mean temperature.

(b) a linear temperature gradient of 5 °C per 100 mm of roof thickness (outside hotter than inside).

2.2.8.3

Special structures

A structure containing heated or cooled fluids shall be subject to a special study to establish the range of temperature conditions appropriate for design.

2.2.9

Earthquake (E)

2.2.9.1

General

The structure shall be designed for the forces, shears and moments resulting from earthquake accelerations of liquid mass, dead mass and external mass responding with the structure.

Consideration should also be given to the horizontal acceleration of surrounding backfill. The effects of this can be significant, having been the apparent cause of a number of tank failures. For example, a large underground reinforced concrete tank (part of the Balboa water treatment plant) suffered severe damage in the 1971 San Fernando earthquake, the damage apparently caused by movement of the surrounding ground. Earthquake earth pressures for which tanks are to be designed are given in reference (2.2).

C2.2.9.2
Horizontal earthquake force

For horizontal convective and inertia earthquake coefficients C_C and C_I respectively, see 2.2.9.3.

W_C and W_I are the weights of the convective and impulsive contents respectively and are given in figure C2.2 for both circular and rectangular tanks.

W_S is the weight of the tank shell (including roof if connected to walls).

C2.2.9.3
Horizontal seismic coefficient

(a) General

Note to Commentators - <u>It was originally envisaged that the latest revision of NZS 4203 would be completed before this draft was circulated for comment. Unfortunately this has not happened and it has been necessary to include interim data to facilitate the calculation of earthquake coefficients.</u>

2.2.9.2
Horizontal earthquake force

Unless calculated on the basis of a more rigorous analysis, the total horizontal earthquake force, V_H, exerted on the tank, shall be calculated in accordance with 2.2.9.4 by a combination of two components; a convective component V_C and an inertia component V_I where:

V_C = $C_C W_C$ .... (Eq. 2-1)

and

V_I = $C_I(W_I + W_S)$ .... (Eq. 2-2)

2.2.9.3
Horizontal seismic coefficient

(a) General. Unless a more rigorous analysis is undertaken, the basic horizontal earthquake coefficients for the tank and its contents shall be determined in accordance with 2.3.9.3(b) and (d)

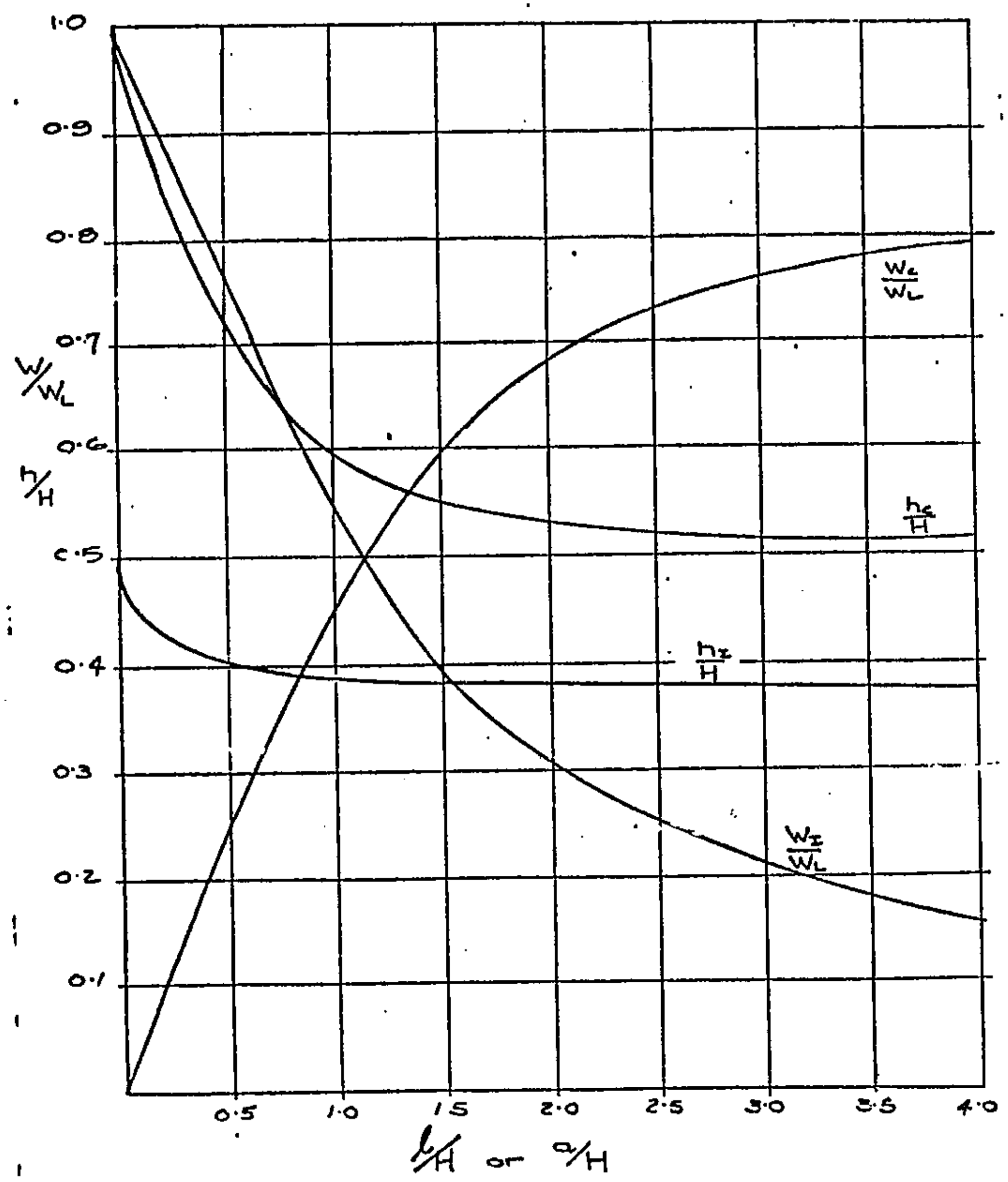


FIGURE C2.2 Rectangular Tank W/W_L and h/H vs l/H
Cylindrical Tank W/W_L and h/H vs a/H

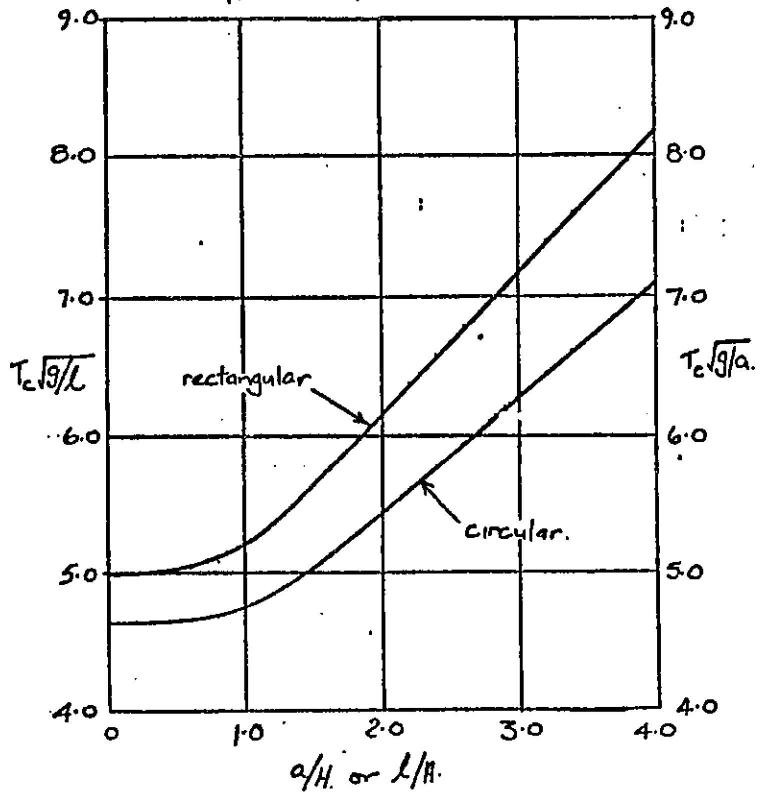


FIGURE C2.3 PERIOD OF FUNDAMENTAL SLOSHING.

Convective and inertia earthquake coefficients should be determined from the revised basic acceleration response spectra given in NZS 4203, taking into account peak ground acceleration, period of vibration, geographical location and the appropriate ductility and damping factors. (In the absence of the loadings code revisions, data recommended by the NZNSEE Study Group on Seismic Design of Storage Tanks(2.3) has been substituted).

Alternatively, the earthquake coefficients may be determined from response spectra established for the specific tank site taking into account the dynamic characteristics of the tank.

(b) Convective coefficient C_c

Equation 2-3 has been derived from the NZNSEE recommended response spectra (2.3) using the following assumptions:

(i) $T_c > 1.0$ second.

(ii) ½% viscous damping.

(iii) Elastic response.

The period of vibration of the convective mode, T_c can be determined from figure C2.3 for both rectangular and circular tanks.

(c) Peak ground acceleration

The peak ground acceleration is a function of the intensity of the design earthquake (earthquake for which the tank is to remain functional). This intensity varies with geographical location and is generally identified in terms of an annual probability of exceedance. To be consistent with NZS 4203, the intensity of the design earthquake is scaled in accordance with the appropriate seismic risk fac-

(b) Convective coefficient C_c. The convective earthquake coefficient shall be determined as a function of peak ground acceleration A_o and natural period of the first mode of sloshing, T_c, in accordance with:

$$C_c = \frac{3A_o}{T_c^{1.4}} \qquad \text{......... (Eq. 2-3)}$$

(c) Peak ground acceleration.

Peak ground acceleration shall be as given in table 2.1.

Table 2.1
PEAK GROUND ACCELERATION

Zone*	Peak Ground Acceleration
A	0.4R
B	0.3R
C	0.2R

* as defined in figure 4 of NZS 4203.

tor R. This risk factor is as defined in NZS 4203 with values and categories appropriate for tank design given in table C2.1 along with the implied probability of exceedance for each category.

Table C2.1
RISK FACTORS AND IMPLIED ANNUAL PROBABILITY OF EXCEEDANCE OF DESIGN EARTHQUAKE

Category	Description	Risk Factor R	Annual Probability of Exceedance
1	Tanks containing highly valuable or hazardous contents, for example, emergency services or toxic chemicals	2.0	.002
2	Tanks which are intended to remain functional in the emergency period for a major earthquake, for example, firefighting water	1.6	.0033
3	Tanks which should be functional in the restoration period for a major earthquake, for example, potable water	1.3	.005
4	Tanks which should be functional in the restoration period for a moderate earthquake	1.0	.01
5*	Tanks which should be functional after a minor earthquake, for example, farm tanks	.075	.02

* Risk category 5 is specific in this code and not part of the

(d) Inertia coefficient C_I

Equation 2-4 states that the inertia earthquake coefficient is equal to the peak ground acceleration, which is also the zero period ordinate on the response spectrum. This assumption is realistic for a circular concrete tank which because of its stiffness, has a very low period of oscillation. Rectangular tanks are generally more flexible than circular tanks which in some instances may lead to structural amplication effects. In lieu of the revised NZS 4203 response spectrum, the inertia coefficient for elastically responding flexible tanks may be calculated from the following expressions:

For $0.05 \leqslant T_I < 0.13$ seconds

$$C_I = A_o (1 + 13T_I) \quad \text{.......... (Eq. C2-4)}$$

For $0.13 < T_I < 0.60$ seconds

$$C_I = 2.7\, A_o \quad \text{.......... (Eq. C2-5)}$$

For $0.6 \leqslant T_I < 10$ seconds

$$C_I = \frac{1.6\, A_o}{T_I} \quad \text{.......... (Eq. C2-6)}$$

Equations C2-4 to C2-6 have been derived from the NZNSEE recommended response spectrum3 using the following assumptions:

(i) Elastic response (μ = I).

(ii) 2 % viscous damping.

Where the tank is supported by a ductile supporting system detailed for ductility in accordance with the appropriate material code, C_I should be taken as the basic coefficient, C_d, applicable for the structural type and material of the support in accordance with NZS 4203.

(d) Inertia coefficient C_I. For rigid tanks with fundamental period of oscillation, T_I less than 0.05 seconds, CI is given by:

$$C_I = A_o \quad \text{.......... (Eq.2-4)}$$

For flexible tanks ($T_I > 0.05$ seconds) CI values shall be in accordance with the basic coefficient given in NZS 4203 taking into account the period of vibration, geographical location and appropriate values of damping and ductility of the tank and its support structure.

The period of vibration of the inertia mode, T_I, should include the effects of flexibility from wall deformations. An approximate method of calculating T_I for a rectangular concrete tank is given in Appendix B.

For an elevated tank, T_I is significantly influenced by the flexibility of the support structure. A method of calculating the period of vibration for the inertia mode for an elevated tank is given by Housner (2.4).

C2.2.9.4
Combinations of horizontal components

The periods of the inertia and convective responses are generally widely separated, the impulsive period being much shorter than the convective period. When responses are widely separated, near-simultaneous occurrence of peak values could occur, and it is thus recommended that the combined impulsive and convective responses be taken as the algebraic sum of the separate components.

For a flexible structure, for example, an elevated tank, it is possible for the response periods for both inertia and convective components, to be of the same order of magnitude. In this instance, the root mean square method of combination better simulates the reduced probability of coincidence of peak inertia and convective responses.

C2.2.9.5
Earthquake pressures
Circumferential earthquake pressure distribution can be represented by a sinusoidal variation for a circular tank (see figure C2.4(a)). Vertical earthquake pressure as presented by Jacobsen (2.5) and Housner (2.6) are of the form shown by the 'exact' curves in figure C2.4(b). The equivalent linear distributions, although a simplification of the actual distribution, are sufficiently accurate for design purposes, and form the basis for equations 2-7 and 2-8. Stresses can be calculated using standard

2.2.9.4
Combinations of horizontal components
Inertia and convective responses shall be combined as follows:

(a) For $T_C/T_I \geq 3$

$$V_H = V_C + V_I \qquad \text{............ (Eq. 2-5)}$$

(b) For $T_C/T_I < 3$

$$V_H = \sqrt{V_C^2 + V_I^2} \qquad \text{............ (Eq. 2-6)}$$

2.2.9.5
Earthquake pressures
In the absence of a more rigorous analysis which takes into account the exact and complex vertical and horizontal variations in hydrodynamic pressures, the tank shall be designed for a horizontally uniform pressure distribution that varies linearly from p_t at the surface of the liquid to p_b at the base,

where

$$p_t = \frac{V_H\,(6h - 2H)}{\ldots} \qquad \text{(Eq. 2-7)}$$

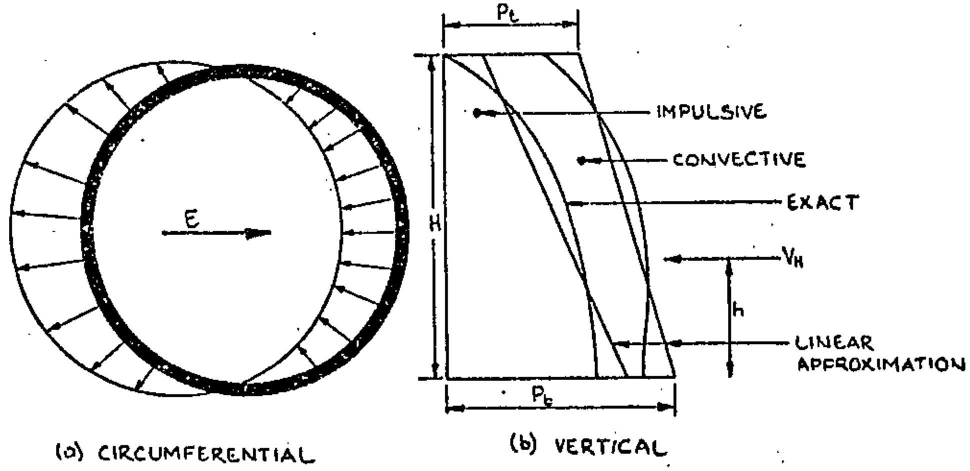


Figure C2.4: EARTHQUAKE PRESSURE DISTRIBUTION IN CIRCULAR TANKS

In figure C2.4(b) h is the height above the base of the centre of gravity of the combined (convective plus inertia) horizontal earthquake force and is given by:

$$h = \frac{V_C h_C + V_I h_I}{V_C + V_I} \quad \text{............ Eq. C2-7)}$$

C2.2.9.6
Overturning moment

(a). Wall. The overturning moment acting at the base of the wall is conveniently expressed in terms of an equivalent weight (W_I, WC, WS) and an equivalent height (hI, hC, hS) where

and

$$P_b = \frac{V_H (4H - 6h)}{\beta H2} \quad \text{........... (Eq. 2-8)}$$

where β is a constant whose value depends on the cross-sectional shape of the tank;

for a circular tank $\beta = \pi a$.
for rectangular tank $\beta =$ 2b.

2.2.9.6
Overturning moment

(a) The overturning moment acting at the base of the wall shall be determined by:

subscripts I, C and S refer to inertia, convective and shell respectively. The variation of these parameters with the a/H ratio (ciruclar) or /H ratio (rectangular) is shown in figure C2.2

(b) Floor. In order to design the tank foundation (or support structure) it is necessary to know the moment M_B arising from the hydrodynamic pressures acting on the base. The variation of this moment with the ratio a/H (circular) or /H (rectangular) is shown in figure C2.5. The moment on the base has been plotted as a dimensionless moment M_B defined by:

(i) Circular tank

$$M_{BI}' = \frac{MBI}{C_I \gamma_L a^2 H^2} \quad \text{.......... (Eq. C2-8)}$$

$$M_{BC}' = \frac{MBC}{C_C \gamma_L a^2 H^2} \quad \text{.......... (Eq. C2-9)}$$

(ii) Rectangular tank

$$M_{BI}' = \frac{MBI}{2C_I \gamma_L \ell b H^2} \quad \text{.......... (Eq. C2-10)}$$

$$M_{BC}' = \frac{MBC}{2C_C \gamma_L \ell b H^2} \quad \text{.......... (Eq. C2-11)}$$

C2.2.9.7
Vertical earthquake acceleration
The effect on vertical ground motion is to alter the internal pressure exerted by the contained liquid; an upward acceleration of the tank will cause an increase in pressure. The incremental

(i) For $T_{C/TI} \geq 3$

$$M_W = CIWIhI + CIWShS + CCWChC \quad \text{.......... (Eq. 2-9)}$$

(ii) For $T_{C/TI} < 3$

$$M_W = \sqrt{(CIWIhI+CIWShS)2+(CCWChC)2} \quad \text{...... (Eq. 2-10)}$$

(b) The overturning moment acting on the floor of the tank shall be determined by:

(i) For $T_{C/TV} \geq 3$

$$M_B = MBI + MBC \quad \text{......... (Eq. 2-11)}$$

(ii) For $T_{C/TI} < 3$

$$M_B = \sqrt{MBI2 + MBC2} \quad \text{......... (Eq. 2-12)}$$

(c) The total overturning moment acting on the foundation or support structure is given by:

(i) For $T_{C/TI} \geq 3$

$$M = M_W + MB \quad \text{......... (Eq. 2-13)}$$

(ii) For $T_{C/TI} < 3$

$$M = \sqrt{(M_{WI}+MBI)2 + (MWC+MBC)2} \quad \text{......... (Eq. 2-14)}$$

2.2.9.7
Vertical earthquake acceleration
The vertical earthquake coefficient shall be as given in table 2.2.

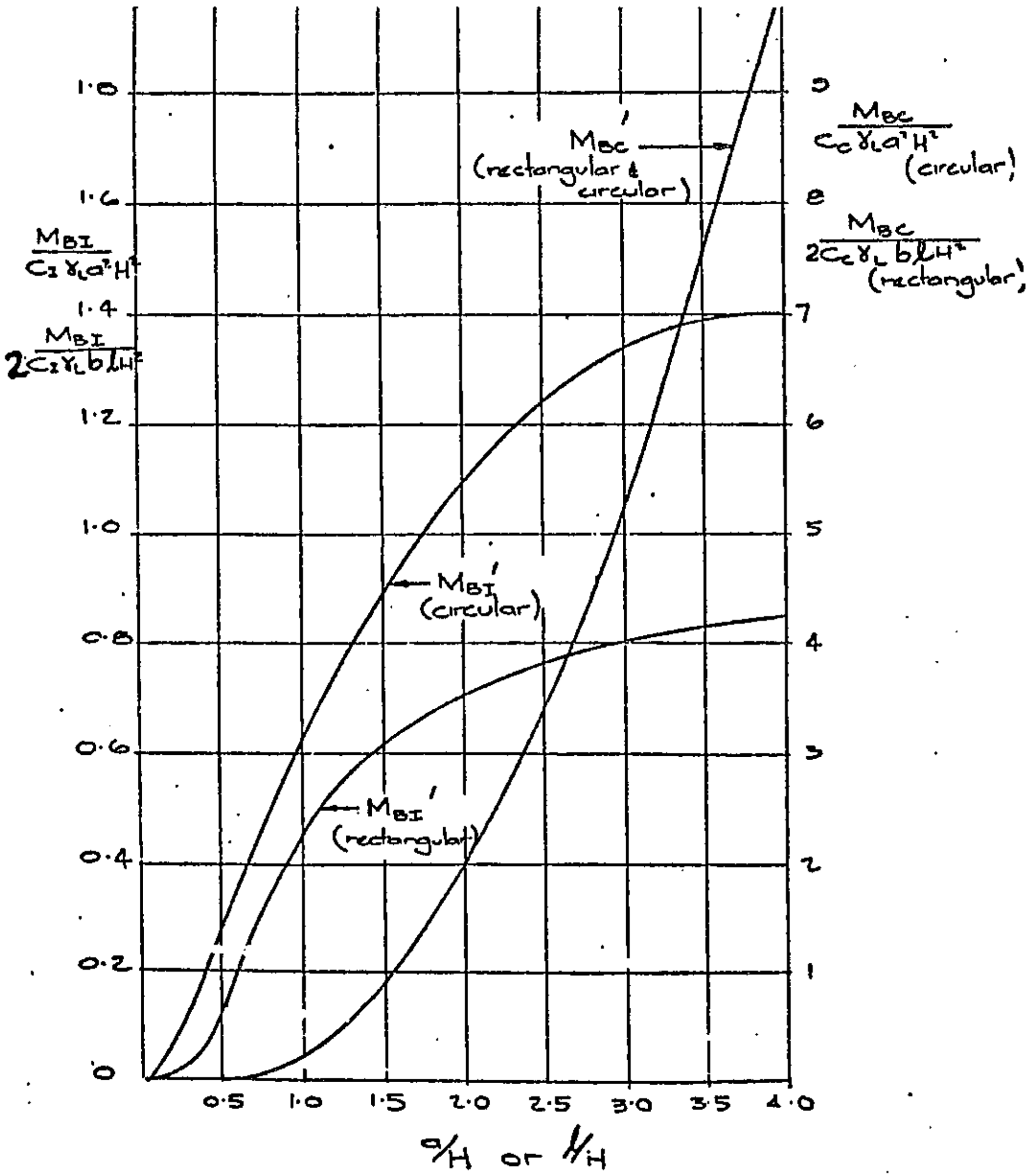


FIG. C 2.5: IMPULSIVE AND CONVECTIVE
MOMENTS ON BASE
(CIRCULAR AND RECTANGULAR TANKS)

Table 2.2
VERTICAL EARTHQUAKE COEFFICIENT

Zone*	Vertical Seismic Coefficient Cv
A	0.27R
B	0.20R
C	0.13R

stresses caused by a vertical acceleration are identical in distributin to those produced by the static liquid load while their magnitudes are some proportion thereof. For example, an upward earthquake acceleration of 0.25 g produces incremental stresses whose magnitude is 25 % of the static liquid containment stresses.

The earthquake coefficients given in table 2.2 correspond to the peak horizontal accelerations for the zone and magnitude of earthquake considered, but reduced by a factor of 0.67.

C2.2.9.8
Combination of horizontal and vertical responses
The stresses from peak horizontal and vertical ground accelerations are combined by their root mean square to account for the reduced probability of their concurrence. E_H is the stress caused by the horizontal component of earthquake acceleration. By way of explanation, considering the example of an upward ground acceleration of 0.25 g, the incremental stress E_V is equal to 0.25F where F is the stress caused by the static containment pressure.

$$E = \sqrt{E_H^2 + (0.25F)^2}$$

C2.2.9.9
Shear transfer
The horizontal earthquake force V_H generates shear forces between the wall and footing and the wall and roof. In rectangular tanks, the earthquake shear is transmitted directly by reaction to vertical bending. In circular tanks, the earthquake shear is transmitted partly by membrane shear and the rest by reaction to vertical bending. For a tank with a height to diameter ratio of 1:4 approximately 20 % of the earthquake shear force is transmitted by the radial base reaction to vertical bending. The remaining 80 % is resisted by membrane shear transfer Q:

2.2.9.8
Combination of horizontal and vertical responses
Concurrence of horizontal and vertical motions shall be considered. Stresses shall be combined according to:

$$E = \sqrt{E_H^2 + E_V^2} \qquad \text{........... (Eq. 2-15)}$$

2.2.9.9
Shear transfer
Earthquake shear forces shall be considered in the design of wall to footing and wall to roof joints:

$$Q = 0.8\ V_H \qquad \text{.......... (Eq. C2.12)}$$

To transmit this shear Q, a shear flow q is required at the wall/footing interface where:

$$q = \frac{Q \sin \delta}{\pi a} \qquad \text{.......... (Eq. C2-13)}$$

The distribution is illustrated in figure C2.6.

The maximum shear occurs at 90 degrees to the earthquake direction and is given by:

$$q_{max} \quad \frac{Q}{\pi a} = \frac{0.8\ V_H}{\pi a} \qquad \text{.......... (Eq. C2-14)}$$

In general the wall/footing interface has sufficient reinforcement through the joint to transmit this shear. However, for precast tank construction the wall panels may be located in a preformed slot in the ring beam footing. Friction between the wall base and footing will generally be insufficient to resist the earthquake shear, thereby requiring some form of mechanical restraint such as galvanized steel dowels.

Failure to provide a means for shear transfer around the circumference will cause circumferential sliding of the wall. The shear resistance is transferred to the principal diagonal, inducing high membrane stresses at the wall junction, balanced by high radial reactions as shown in figure C2.7.

The roof to wall joint is subject to earthquake shear from the horizontal acceleration of the roof. Where dowels are provided to transfer this shear, the distribution will be the same as shown in figure C2.6 with maximum shear given by:

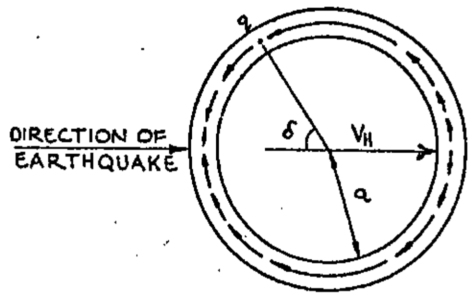
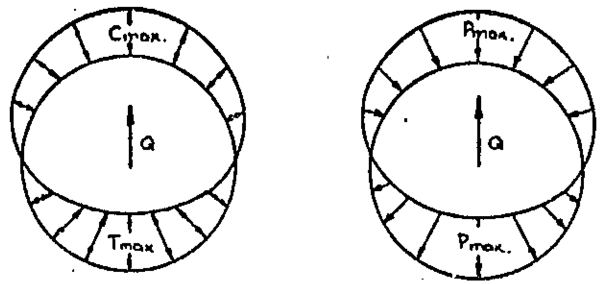


Figure C2.6: MEMBRANE SHEAR DISTRIBUTION



(a) WALL BASE HOOP FORCES $C_{max.} = T_{max.} = \frac{Q}{\pi}$

(b) WALL BASE REACTIONS $P_{max.} = \frac{Q}{\pi a}$

FIGURE C2.7: FORCES INDUCED BY BASE SHEAR TRANSFER FOR A WALL FREE TO SLIDE CIRCUMFERENTIALLY, BUT RESTRAINED RADIALLY

$$q_{max} = \frac{0.8\ F_R}{\pi a} \quad \text{.......... (Eq. C2-15)}$$

Where F_R is the force from the horizontal acceleration of the roof.

For tanks with roof overhangs, the concrete nib can be designed to withstand the earthquake force. Because the roof is free to slide on top of the wall, the shear transfer will be reacted over that portion of the circumference where the nib overhang comes into contact with the wall. Typically, the distribution of forces and wall reactions will be similar to that shown in figure C2.7(b) but reacting on only half of the circumference. The maximum reaction force will be given by:

$$P_{max} = \frac{2\ F_R}{\pi a} \quad \text{.......... (Eq. C2-16)}$$

C2.2.9.10
Freeboard
The horizontal earthquake acceleration causes the contained fluid to slosh with vertical displacement of the fluid surface. The maximum vertical displacement, d_{max}, may be calculated from the following expressions for circular and rectangular tanks.

(a) Circular tank

$$d_{max} = aC_c \quad \text{.......... (Eq. C3-17)}$$

(b) Rectangular tank

$$d_{max} = \ell C_c \quad \text{.......... (Eq. C3-18)}$$

The amount of freeboard required for design will vary. Where overtopping is tolerable no freeboard provision is necessary. Where loss of liquid must be prevented (for example, tanks for the storage of toxic liquids), where overtopping may result in

2.2.9.10
Freeboard
Provision shall be made to accommodate the maximum wave oscillation generated by earthquake acceleration.

scouring of the foundation materials or cause damage to pipes and/or roof, then provisions should be made by:

(i) freeboard allowance, and/or

(ii) designing the roof structure to resist the resulting uplift pressures.

C2.2.10
Shrinkage and swelling

C2.2.10.1
Walls
The shrinkage and swelling strains given in table 2.3 were derived using the CEB-FIP (1978) Model Code, except that predicted shrinkages were doubled in accordance with the recommendations of reference (2.7).

The following assumptions were made:

(a) Shrinkage commences immediately after casting

(b) Shrinkage regain is 100 % and occurs immediately the tank is filled

(c) Precast wall panels are subject to free shrinkage until they are erected, 50 days after casting. Shrinkage continues until the tank is filled, a further 50 days after erection

(d) For cast in-situ tanks, filling occurs 100 days after the walls are cast

(e) Shrinkage strains are reduced by a creep reduction factor given by:

2.2.10
Shrinkage (S_h) and swelling (Sw)

2.2.10.1
Walls
In the absence of a rational analysis to determine shrinkage and swelling strains appropriate to the expected construction/loading history, the tank walls shall be designed for the shrinkage and swelling strains given in table 3.3.

$1 - e^{-\phi}$

where ϕ is the creep factor for the concrete between the time shrinkage stresses commence (that is, when shrinkage movement is restrained) to the time the tank is filled.

(f) The creep reduction factor used to assess long term (500 days after filling) swelling (implied by the load combinations) is given by: $e^{-\phi_\infty}$, *where ϕ_∞ is the long term creep factor.*

For cast insitu tanks, shrinkage stresses develop between the time the walls are cast until the tank is filled. Precast panels on the other hand do not develop shrinkage stresses until the panels are locked into position by which time a significant amount of shrinkage has already occurred.

On filling, there is a rapid shrinkage regain or swelling of the concrete. Swelling strains generally exceed creep-reduced shrinkage strains because the swelling rate is much faster than the shrinkage rate and hence in the short term less affected by creep relaxation. (The swelling strains given in table 2.3 are net strains, that is, counteracting shrinkage strains have been deducted). Although swelling strains are generally similar for both types of construction, because the counteracting shrinkage strains are lower for precast panels than those for cast in-situ construction, the net swelling strains are correspondingly higher.

The initial swelling strains caused by the shrinkage regain that occurs when the tank is filled are in time reduced by creep relaxation. This reduction is taken into account in the value given to the load factor used in the load combinations in 2.3.

Exposure of a tank to wind and sun casues the outside surface to dry out resulting in a shrinkage gradient through the wall. Few

Table 3.3
SHRINKAGE AND SWELLILNG STRAINS (CREEP ADJUSTED)

Wall Thickness (mm)	Strain (x 10-6)			
	Shrinkage (ε_{sh})		Swelling (ε_{sw})	
	Precast	Instiu	Precast	Insitu
100	70	120	300	250
125	55	105	265	215
150	50	85	205	170
175	50	75	175	150
200	45	70	160	135
225	40	65	145	120
250	35	60	135	110

2 2.10.2
Roof
The roof shall be designed for the shrinkage and swelling strains given in table 2.3.

data are available relating to the extent of the differential and to the distribution through the wall thickness. It appears however that the gradient is low for much of the wall thickness with most of the differential occurring in the outer 15 %. This results in crazing of the outer surface which, while relieving the shrinkage stresses, has negligible effect on the serviceability of the tank. Consequently differential shrinkage gradients between the inside and outside faces of sections of the walls do not require specific design.

The stresses caused by volumetric changes in concrete are characteristically similar to those caused by thermal effects. Shrinkage is directly analogous to an average temperature decrease while swelling corresponds to an average temperature increase. The similarity of thermal and shrinkage effects means that the method of analysis developed for temperature stresses can also be used for calculating shrinkage stresses. The thermal equivalent is derived by dividing the shrinkage (or swelling) strain by the coefficient of thermal expansion for concrete:

$$T = \frac{\varepsilon_s}{\alpha} \qquad \text{.......... (Eq. C2-19)}$$

C2.2.10.2

Roof

Shrinkage (or swelling) of the roof will not produce significant stresses unles the shrinkage (or swelling) movement is restrained, for example, where the roof is cast monolithically with the walls.

C2.2.11

Non-symmetric loads for circular tanks

Ambient thermal loads and hydrodynamic seismic pressures are not rotationally symmetric, but vary continually around the tank's

2.2.11

Non-symmetric loads for circular tanks

For thin walled circular tanks ($t/a < 0.03$) non-symmetric loads may be considered to be rotationally symmetric and equal to the value at the section under consideration.

perimeter. Analysis have shown that this variation is generally low enough for stresses at any given section to depend only on the local temperature or pressure distribution.

C2.3
Load combinations

C2.3.2
Transient loads that should be omitted if beneficial, are earth pressure (EP), shrinkage (S_h), swelling (Sw) and temperature (T).

The prestress force may vary between P_{max} and Pmin, the maximum and minimum due to in-time losses, respectively. To ensure that the more adverse condition is incorporated in design, both P_{max} and P_{min} should be considered in the load combinations.

(a) Group A loads. Group A load cases are permanent loads plus variable loads of long duration; or permanent loads plus frequently repetitive loads. Shrinkage is a long during load. Swelling can be either short or long duration; this is accounted for in the load factor.

Load case 2-16 equally applies for shrinkage and swelling; shrinkage applies when the tank is empty prior to filling, swelling applies when the tank is emptied for maintenance.

(b) Group B loads. Group B load cases are permanent loads plus infrequent combinations of transient loads.

Load case 2-21 applies equally to shrinkage and swelling; shrinkage - tank empty prior to filling, swelling - tank empty for maintenance.

The earthquake component in load case 2-25 refers to the pressure exerted on the roof by sloshing of the contained liquid.

2.3
Load combinations

2.3.1
Structures and members shall be designed in accordance with the allowable stresses to resist the loading combinations specified in 2.3.2 as applicable.

2.3.2
The loads described in 2.2 shall be combined in groups as defined below. In any group, if a worse effect is obtained by omitting one or more of the transient items, this case shall also be considered.

(a) Group A loads

wall $A = D + EP + P + (S_h \text{ or } 0.5\ Sw)$ .... (Eq. 2-16)
$A = D + F + EP + P + 0.5\ S_w$ .... (Eq. 2-17)

roof $A = D + P + S_h$ .... (Eq. 2-18)

(b) Group B loads

wall $B = D + F + EP + P + 0.8E + 0.5\ S_w$ .... (Eq. 2-19)
$B = D + F + EP + P + 0.7\ S_w + T$ .... (Eq. 2-20)
$B = D + EP + P + T + (0.7S_h \text{ or } 0.35Sw)$ .... (Eq. 2-21)

roof $B = D + L + P$ .... (Eq. 2-22)
$B = 0.8D + W + P$ .... (Eq. 2-23)
$B = D + T + P + (0.7S_h \text{ or } 0.7Sw)$ .... (Eq. 2-24)
$B = 0.8D + 0.8E$ .... (Eq. 2-25)

REFERENCES

(2.1) Priestley, M.J.N., "Ambient Thermal Stresses in Circular Prestressed Concrete Tanks", ACI Journal, Vol. 73, No. 10, Oct. 1976, pp 553 - 560.

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(2.3) New Zealand National Society of Earthquake Engineering Draft Report., "Seismic Design of Storage Tanks", 1984.

(2.4) Housner, G.W., "Dynamic Pressure on Fluid Containers", Chapter 6 in Nuclear Reactors and Earthquakes, Atomic Energy Commission, Report No TID-7024, 1963.

(2.5) Jacobsen, L.S., "Impulsive Hydrodynamics of Fluid Inside a Cylindrical Tank, and of a Fluid Surrounding a Cylindrical Pier", Bulletin Seismological Society of America, Vol 39, 1949, pp 189-204.

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(2.7) Vadhanavikkit, C. and Bryant, A.H., "Creep and Shrinkage of Concrete", University of Auckland, School of Enginering, Department of Civil Engineering, Report No 334, March 1984.

(2.8) Priestly, M.J.N., Vessey, J. and North, P.J., "Concrete Structures for the Storage of Liquids" - A new Draft SANZ Code", New Zealand Concrete Construction, Feb 1985.

SECTION 3
COMMENTARY

C3.1
General
Guidance is given in this section on the materials to be used in construction of concrete structures for the storage of liquids, and on their properties to be used for design purposes.

C3.2
Concrete

C3.2.2
The comparatively high strengths and low water/cement ratios specified for concrete are to ensure high quality dense concrete with low shrinkage, to minimise cracking and permeability.

C3.2.4
Minimum cement contents are specified to ensure dense durable concrete with low permeability. Maximum cement contents are specified in order to minimise shrinkage, and shrinkage cracking in thin sections, and heat-of-hydration cracking in thick sections.

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SECTION 3
MATERIALS

3.1
General
This Section applies to materials to be used in the construction of reinforced or prestressed concrete or cement mortar elements in liquid retaining structures.

3.2
Concrete

3.2.1
Mix constituents. All concrete shall comply with requirements of NZS 3109, and NZS 3104.

3.2.2
Compressive strength. The specified compressive strength shall not be less than:

(a) reinforced concrete elements, ... $f'_c = 25$ MPa

(b) prestressed concrete elements, ... $f'_c = 35$ MPa

3.2.3
Water/cement ratio. The water/cement ratio of concrete used for liquid-retaining elements shall not exceed 0.55.

3.2.4
Cement content. For all concrete subject to continuous or frequent water contact or condensation, such as reservoir floors walls and roof, the following limits to cement contents shall apply:

The limits correspond to those specified by BS 5337 for concrete of 20 mm maximum aggregate size. Note that cement contents approaching the upper limit for prestressed concrete should only be adopted for very high strength concrete (50-60 MPa). Concrete roofs are specifically included, because condensation will mean the inside surface will be continuously wet. This conforms with the approach taken by BS 5337.

C3.2.5

Air entrainment reduces water demand and therefore reduces shrinkage. It also changes the nature of the voids between the particles in the concrete matrix from interconnected tubular voids to discrete spherical voids, thereby decreasing permeability. Pending further experience, waterproofing additives are not recommended for use in water retaining structures.

C3.2.6

Stresses induced in storage tanks by thermal loading and creep and shrinkage effects are directly proportional to the value of the modulus of elasticity, E, pertaining at the time the action is applied. Consequently it is important that realistic values should be adopted. Wherever possible, locally based test data should be used to assist in assessing the design value of E_c, and recognitic.. ..de of the fact that the insitu compression strength f'_c will normally exceed the specified strength by 20 % or more. This will result in increased values of E_c when using the following equations, which are duplicated from NZS 3101 Section 3.3.4.

$$E_c = 0.043 w_c^{1.5} \sqrt{f'_c} \text{ MPa} \quad \text{.............. (Eq. C3-1)}$$

where $1400 \leqslant w_c \leqslant 2500$ is the concrete density in kg/m3

minimum cement content, reinforced or
prestressed concrete : 300 kg/m3
maximum cement content, reinforced concrete
: 400 kg/m3
maximum cement content prestressed concrete
: 550 kg/m3

3.2.5

Admixtures. Air entrainment in accordance with the requirements of NZS 3104 shall be used wherever practical for all concrete subject to continuous or frequent water contact, or condensation.

3.2.6

Modulus of Elasticity. The design value of the modulus of elasticity of concrete shall be determined in accordance with provisions of NZS 3101, unless otherwise established by testing.

For normal weight concrete the value

$$E_c = 4700 \sqrt{f'_c} \text{ MPa} \quad \text{(Eq. C3-2)}$$

may be used.

C3.2.7
Thermal stresses in tanks are also proportional to the concrete coefficient of thermal expansion α_c which can vary between $5 \times 10^{-6}/°C$ and $15 \times 10^{-6}/°C$ depending primarily on aggregate type, with Andesites and Limestones giving lowest values and Quartzites typically giving highest values. Some typical values are listed in table C3.1. Limited data on New Zealand concretes is available in Reference (3.1).

Table C3.1
TYPICAL COEFFICIENTS OF THERMAL EXPANSION FOR WATER-CURED CONCRETE MADE FROM DIFFERENT AGGREGATE TYPES

Aggregate	Coefficient of Thermal Expansion $\times 10^{-6}/°C$
Andesite	6.5
Basalt	9.5
Dolerite	8.5
Foamed slag	9
Granite	9
Greywacke	11
Limestone	6
Pumice	7
Quartzite	13
Sandstone	10

3.2.7
Coefficient of thermal expansion. The design value of the coefficient of thermal expansion shall be determined with due regard for the constituent materials to be used for the construction.

In the absence of information on the aggregate type to be used in construction of the tank, a reasonably conservative value of α_c = 11 x 10-6 should be used.

C3.3

Cement mortar

Modulus of elasticity and coefficient of thermal expansion of cement mortars should be assessed on the behaviour of representatives testing wherever possible. Where characteristics of constituents are not known at the time of testing, E_c and should be assessed as for equivalent strength concrete. It should, however, be recognised that because of high cement-aggregate ratios required for cement mortar structures, values for α_c will tend to be higher than for equivalent concretes. It is recommended that values listed in table C3.1 should all be increased by 1 x 10-6 for cement mortar as a consequence.

3.3

Cement mortar

3.3.1

Pneumatically placed mortar.

3.3.1.1

The aggregate for penumatically placed mortar shall, unless otherwise approved comply with the requirements of NZS 3121 for sand, and in addition shall have a fineness modulus lying between the limits of 2.50 and 3.30.

3.3.1.2

Except for special works requiring other mixes, the proportions for pneumatic mortar shall be one part of cement to not more than four parts of moist sand (3 % to 5 % moisture content) volumetrically batched, or weigh-batched to equivalent proportions.

3.3.1.3

Compressive strength. The specified compressive strength of nominal 100 mm x 50 mm diameter cores taken from representative work panels shall not be less than:

(a) for reinforced concrete elements, ... f'_c = 25 MPa

(b) for prestressed concrete elements, ... f'_c = 35 MPa

The panels shall be fabricated, cured and cored as described in 3.3.1.4 and testing of cores shall be in accordance with NZS 3112, Part 2, Section 9. Test results shall meet the requirements of NZS 3109.

3.3.1.4
Representative work panels of such number and at such times as the Engineer may specify shall be fabricated by gunning on to horizontal rigid plywood forms. These panels shall be approximately 300 mm square and 125 mm thick. Immediately after manufacture the panels shall be covered with impermeable plastics sheeting to prevent water loss, and shall be protected from direct sunlight. After not less than 16 hours they shall then for forwarded to the laboratory in a container which prevents damage or loss of moisture. The panels shall be stored in the laboratory at a temperature of 21 ±2 °C under moist conditions such that free water is maintained on their surfaces at all times. Between 3 and 7 days after manufacture the panels shall be cored. Three cores shall be taken from near the centre of each panel, and each set of three cores shall constitute a test sample. The cores shall remain in moist storage at 21 ±2 °C until tested at 28 days after forming the panel.

3.3.1.5
Modulus of Elasticity. The design value of the modulus of elasticity of pneumatically placed mortar shall be determined in accordance with provisions of NZS 3101 for concrete of equivalent strength and density, unless otherwise established by testing.

3.3.1.6
Coefficient of thermal expansion. The design value of coefficient of thermal expansion shall be determined with due regard for the constituent materials to be used for the construction.

3.3.2
Hand-placed and mechanically-placed mortars.

3.3.2.1
The aggregates for hand-placed and mechanically placed mortars shall comply with the requirements for sands for mortars and external rendering as specified in NZS 3103.

3.3.2.2
The proportions of hand-placed and mechanically placed mortar shall be two parts of cement to not more than five parts of moist sand (3 % to 5 % moisture content) volumetrically batched, or weight-batched to equivalent proportions.

3.3.2.3
Compressive strength. The specified compressive strength of nominal 100 mm x 50 mm diameter cylinders moulded, cured and tested in accordance with NZS 3112 shall be not less than $f'_c = 25$ MPa. Test results shall meet the requirements of NZS 3109.

3.3.2.4
Modulus of elasticity. The design value of the modulus of elasticity of hand-placed or mechanically placed mortar shall be determined in accordance with provisions of NZS 3101 for concrete of equivalent strength and density, unless otherwis established by testing.

3.3.2.5
Coefficient of thermal expansion. The design value of the coefficient of thermal expansion shall be determined with due regard for the constituent materials to be used for the construction.

3.4
Non-prestressed reinforcement

3.4.1
All hot rolled reinforcing steel and welded wire fabric shall comply with requirements of NZS 3402P, or NZS 3422 as appropriate.

3.4.2
Galvanized or zinc coated netting and twisted or loosely linked steel wire shall comply with BS 1485 or BS 4102, as appropriate.

3.4.3
Modulus of elasticity of hot-rolled reinforcement shall be taken as 200 GPa, unless otherwise established by testing.

3.5
Prestressed reinforcement

3.5.1
Prestressed reinforcement shall comply with the requirements for prestressing steel of NZS 3109.

3.5.2
Modulus of elasticity of prestressed reinforcement shall be established by tensile testing.

C3.5
Prestressed reinforcement
Modulus of elasticity of prestressed reinforcement E_s is generally lower than the value of 200 GPa commonly used for normal strength reinforcement, and may be as low as 150 GPa. Values of E_s should be based on manufacturers' test results, or on results from independent testing.

REFERENCES

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SECTION 4
COMMENTARY

C4.1
The requirements set out in this Section are those which are additional to the requirements of NZS 3109:1980 Specification for Concrete Construction. The additional requirements are related to the unusual service conditons of reservoirs and other water retaining structures and the need to provide watertight construction and durability.

C4.2.2
Construction joints
A construction joint is a joint in the concrete introduced for convenience in construction at which measures are taken to achieve subsequent continuity with no provision for further relative movement. Joints prepared in accordance with all of the requirements of NZS 3109 and with sufficient reinforcement through them to prevent all relative movement should perform satisfactorily. If complete continuity cannot be obtained the joint should be treated as a movement joint and taken into account in the design (See 2.1).

SECTION 4
CONSTRUCTION REQUIREMENTS

4.1
General

4.1.1
This section applies to the construction of all reinforced and prestressed concrete liquid retaining structures covered by this Standard. The requirements for the construction of cement mortar structures are covered in Section 7.

4.1.2
Except where otherwise specified the requirements of NZS 3109 also apply.

4.2
Concrete placing

4.2.1
The placing of concrete for water retaining structures shall be carried out with particular care but generally in accordance with the requirements of NZS 3109.

4.2.2
Construction joints
Construction joints shall be prepared as specified in NZS 3109.

Construction joints should be located in positions selected by the design engineer and shown on the drawings. Typical applications for construction joints are in floor joints, and between successive lifts in a reservoir wall. Generally all joints should be either vertical or horizontal.

Waterstops are not usually required at construction joints but may be included at the discretion of the design engineer. Before a waterstop can be effective some relative movement (debonding) must take place first and strictly speaking such joints are movement joints.

A special case of construction joints is the infill between precast wall elements of circular prestressed reservoirs. It is not good practice to use dry pack mortar in the joint as uneven compaction can cause uneven bearing stresses on the joint faces. It is now common practice to make such joints with an insitu concrete infill placed in one lift using a super plasticiser additive to aid placing. The dimensions of the infill should be sufficient for adequate placing of the concrete. The joint between the precast and insitu concrete should be planar and square to the panel face. Shear keys, grooves or other stress raisers should be avoided as they can lead to uneven stresses on the joint faces. Such details have been known to cause spalling to the precast element when prestress was applied.

Sometimes a small rectangular groove is formed on the waterface of the joint line and a sealant applied.

C4.2.3
Movement joints
A movement joint is a specially formed joint intended to accommodate relative movement between adjoining parts of a structure, special provision being made for maintaining the watertightness of the joint. Movement joints may be of the following types:

4.2.3
Movement joints
Movement joints shall be provided as necessary to ensure that design assumptions are realised. Provision shall be made for displacement and rotation without the loss of water.

(a) Contraction joint. This is a movement joint which has a deliberate discontinuity but no initial gap between the concrete on both sides of the joint. The joint is intended to permit contraction of the concrete.

A distinction should be made between a complete contraction joint, in which both the concrete and reinforcement are interrupted, and a partial contraction joint, in which only the concrete is interrupted while the reinforcement is continued through the joint.

A water stop and/or sealing compound should be provided at contraction joints.

(b) Expansion joint. This is a movement joint which has complete discontinuity in both reforcement and concrete and is intended to accommodate either expansion or contraction of the structure. Water stops, a joint sealing compound and joint filler are essential at expansion joints.

(c) Sliding joint. This is a movement joint which has complete discontinuity in both reinforcment and concrete. Special provision is made to facilitate relative movement in the plane of the joint.

(d) Sliding layer. A sliding layer is a sp.cial category of sliding joint intended to permit sliding over a considerable area, for example, between a floor and a subfloor or blinding layer.

The effectiveness of movement joints in controlling cracking depends on their correct location, which may be characterized as the place where cracks would otherwise develop, for example, at changes of section. Movement joints in walls should preferably align with joints in the floor or wall footing. All movement joints should be designed and constructed so that the watertightness will be maintained during the subsequent movement of the joint. The location of all movement joints should be decided by the engineer and be indicated on the drawings.

Joints in floors. The floor of a reservoir may be designed to permit shrinkage and thermal contraction by minimizing restraints to movement. A separating layer should be provided between the floor slab and blinding concrete, for example, a layer of thick polyethylene sheeting. Panels may be cast consecutively or with a gap between adjacent panels. This gap should not exceed 1 m. Joints should be complete contraction joints except where allowance is made for expansion and an expansion joint provided.

Alternatively, the floor may be designed as fully restrained against shrinkage and thermal contraction. It should be cast direct on to the blinding concrete. Joints to allow for contraction should not be necessary but, if provided, should be of the partial contraction type.

Joints on walls. Careful consideration should be given to the probable contraction behaviour of the walls. Design and construction practice for restrained and unrestrained walls is described in BS 5337.

Pipes through walls and floors. When it is necessary for a pipe to pass through a wall or floor it is preferable to cast the pipes into the panel when it is concreted. If this is not practicable it will be neccessary to box out. In either case it is desirable that the position of the pipe should not coincide with a joint. When an opening has been boxed out the sides of the opening should be treated as construction joints.

Jointing material and water stops. The performance of joints is very dependent on the performance of the jointing materials used and care should be taken in selecting appropriate materials for particular conditions. See reference (4.1).

Appendix D of BS 5337 discusses the problem and solution.

Recent experience has shown that polysulphide rubber compounds are attacked by chlorine and are not suitable for use in reservoirs containing treated water. An initial filling with a higher than usual chlorine content for sterilisation hastens the attack.

C4.3.1
Unbonded tendons
Unbonded tendons are not allowed because sufficiently reliable protective systems for the environment have not been developed for reservoirs in New Zealand.

C4.3.2
Wound tanks
Tanks constructed with wound tendons which are stressed by drawing the tendon through a die are subject to variable prestress due to variations in wire diameter as supplied and to wear of the die. Such a method will not consistently meet this requirement (See ACI 344R-70).

The protection of the prestressing wires is crucial to the satisfactory long term performance of the tank. Poorly placed shotcrete coatings can have voids along the wires due to a shadow effect. Such voids can collect and transmit water leading to corrosion and early failure of the tank.

Good construction practice for the application of shorcrete is described in ACI 344R-70.

REFERENCES

4.1

4.3
Pretressing, Stressing and grouting.

4.3.1
Unbonded tendons
Unbonded tendons shall not be used in water retaining structures.

4.3.2
Wound tanks
Cylindrical tanks constructed using wound pretressing tendons shall be constructed using equipment capable of stressing and measuring the applied tendon force to within ±7.5 % of the nominal force required.

Systems not consistently meeting this requirement may be used if a possible variation of initial prestress force of ±20 % is treated as an additional load case in the design.

Protection of the tendons shall include the proper application and curing of a shotcrete cover of not less than 25 mm.

SECTION 5
COMMENTARY

C5.1.3
BS 5337 has severe requirements for water proofing of roofs but in New Zealand the atmosphere is cleaner and the worst potential contaminants are probably bird droppings. Reasonable protection should be provided by any reinforced concrete roof designed in accordance with this Code.

C5.2
Stiffness of cracked section
Although curvatures caused by strain induced loads (temperature and shrinkage) are insignificant as a proportion of the ultimate deformation, they may be large compared to elastic loadings at design load level. Hence, although temperature or shrinkage will not significantly effect ultimate capacity of the tank, they may severely bear on the servicability of the structure.

DZ 3106
(CPT KMcI 6)

SECTION 5
DESIGN OF REINFORCED CONCRETE ELEMENTS

5.1
General

5.1.1
This Section applies to the design of all unlined reinforced concrete elements of liquid retaining structures covered by this Standard that are in close proximity to the contained liquid, such as reservoir walls, floors and roofs.

5.1.2
The design of reinforced concrete elements of liquid retaining structures shall comply with the requirements of NZS 3101 Appendix B except as modified by the requirements of this Section.

5.1.3
For structures containing drinking water, roofs shall be waterproof and shall be graded so that they do not pond water.

5.2
Stiffness of cracked section

5.2.1
Except as provided by 5.2.2 strain induced forces (such as thermal and shrinkage) in the walls of circular reinforced concrete reservoirs may be calculated on the basis of an uncracked section and reduced by a factor F_t representing the local reduction in stiffness resulting from cracking in each direction.

5.2.2
A more rigorous approach to that given in 5.2.1 including an ana-

Increasing temperature or shrinkage subject the concrete to increasing stresses until the cracking strength of the section is reached. Further increase in stress is accompanied by a decrease in section rigidity as the crack propagates. A point is eventually reached where the crack propagation stops because the section reaches a rigidity capable of resisting the stress without further deformation. This stress is somewhat less than that calculated assuming an uncracked section and can be assessed simply, and with sufficient accuracy, by factoring the uncracked section moments and axial forces by a reduction coefficient F_t representing the reduction in stiffness with cracking. This reduction factor is given by the ratio of the cracked moment of inertia I_{cr} to the uncracked moment of inertia I_g adjusted for tension stiffening in the concrete. Values of F_t are plotted in figure C5.1 for a range of wall thickness and steel ratios.

The I_{cr} values used in tabulating F_t ignore the presence of axial forces. This omission is necessary to maintain simplicity, however the resulting errors are expected to be small and on the conservative side. Specifically, axial tension would further reduce section stiffness with a corresponding decrease in thermal and/or shrinkage stresses. Axial compression on the other hand increases section stiffness with a corresponding increase in thermal/shrinkage stresses. However, unless allowable compressive stresses are exceeded in the concrete, this load case is unlikely to result in an adverse service condition.

The I_{cr}/I_g ratio depends on wall thickness and reinforcement content. Because these parameters, the latter in particular, may vary with wall height, it may be necessary to calculate F_t values for each critical section of the wall, that is, where there is a change of wall thickness or steel content.

Tension stiffening significantly increases the stiffness above that calculated at a crack. Limited experimental and theoretical analysis of the cracked section stiffness everywhere in the wall may be used.

evidence for slabs indicates that the stiffening effect decreases with increasing reinforcement content p, and with increasing moment level (after cracking). Approximate maximum figures are 100 % increase at p = 0.005 and 30 % increase at p = 0.02. Because of its significance tension stiffening has been included in the derivation of the F_t values.

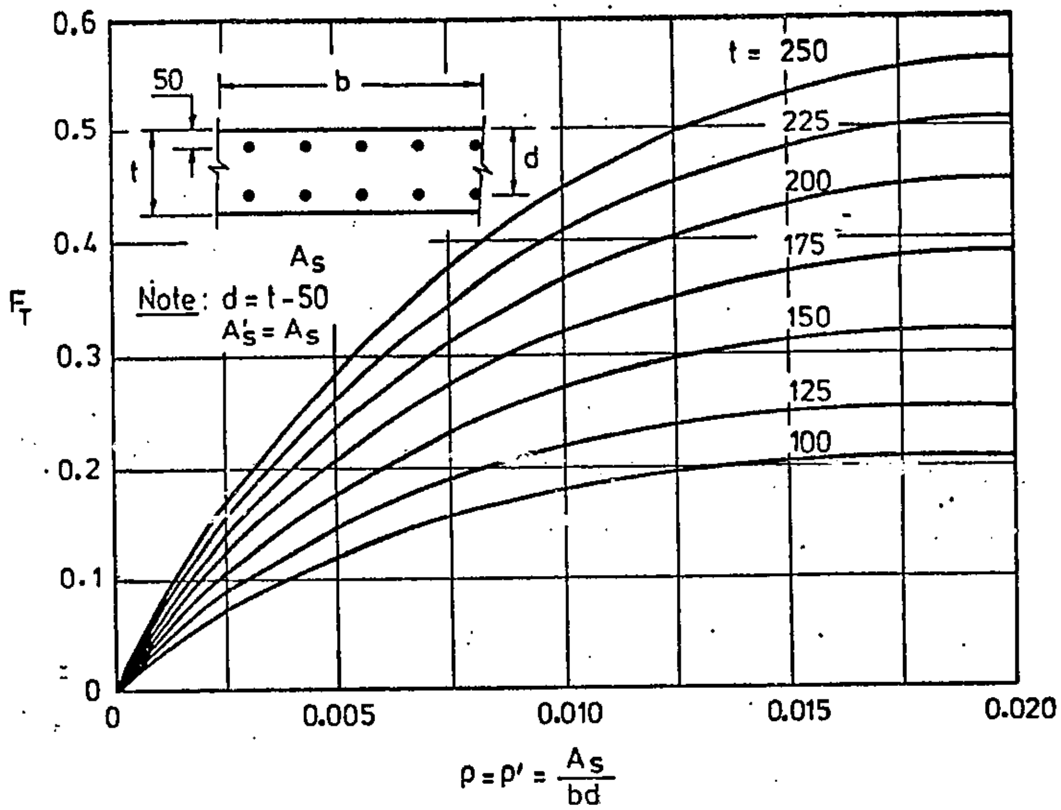

Fig. C5.1 REDUCTION OF STIFFNESS OF DOUBLY REINFORCED WALL ON CRACKING (Including tension stiffening effect)

5.3
Maximum permitted concrete stresses

5.3.1
The allowable stresses in concrete for calculating the bending strength of the elements of the structure shall be as given in table 5.1

Table 5.1
MAXIMUM PERMITTED STRESSES IN REINFORCED CONCRETE

	Stresses in MPa
Load combinations	Compression
Group A combinations	.45 f'c
Group B combinations	.60 f'c

C5.3.2

Note to Commentators - In the previous Code (NZS 3106P Part 1:1978) the design philosophy adopted was that cracking was to be prevented by limiting steel and concrete tensile stresses. Thus it was necessary to calculate concrete tensile stresses based on a transformed section.

In this Code the design philosophy adopted is to recognise that cracks will inevitable occur and to limit their width by limiting steel stresses and more rigorously calculating stresses due to transient loads and secondary effects. Thus it is not necessary to calculate concrete tensile stresses and limits are not given in table 5.1 as cracking is assumed to occur.

C5.4.1

The permissible steel stresses given in table 5.2 are varied to suit the degree of exposure. For walls less than 225 mm thick the exposure on one face is deemed to affect the reinforcement on both faces. This is consistent with the provisions of BS 5337:1976. "The structural use of concrete for retaining aqueous liquids". However it is not as conservative as the provisions of ACI 350R - 77 "Concrete Sanitary Engineering Structures", which is intended for use with more aggressive liquids.

5.3.2

In stress calculations the tensile strength of concrete shall be ignored, except that, in respect of provisions of shear in flexural memebers, the concrete may be assumed to take diagonal tension for the purposes of Appendix B of NZS 3101.

5.4

Reinforcing steel stresses

5.4.1

Maximum permissible stresses for plain bars, deformed bars, and hard drawn steel mesh conforming to NZS 3422 shall be as given in table 5.2.

5.5

Minimum reinforcement

5.5.1

The minimum reinforcement in each of two directions at right angles, in those parts of the structure not restrained from movement, shall have an area of not less than 0.3 percent of the gross concrete section normal to that direction for plain round bars, 0.25 percent for deformed bars, and 0.2 percent for welded hard drawn steel mesh.

C5.5.2

The most common situtation where partial restraint of shrinkage will occur is for a floor slab with an integral slab/foundation beam construction. Consider the case of a circular reservoir, where, as is generally the case, the foundation ring beam supporting the wall is cast first, with the slab cast later. Radial shrinkage of the slab is restrained by hoop compression in the foundation beam. The degree of restraint to slab shrinkage will depend on the relative stiffness of the slab in radial tension, and the ring beam in hoop compression.

A simple compatibility analysis will show that the shrinkage stress induced will be given by

$$f_s = \frac{1}{1 + K_R} \cdot Ecs \cdot \varepsilon_{sh} \quad \text{............ (Eq. C5-1)}$$

where ε_{sh} is the creep-compensated shrinkage strain, Ecs is the slab Modulus of Elasticity and

$$K_R = \frac{E t_s R}{E_{cb}\, Ab} \quad \text{............ (Eq. C5-2)}$$

is the ratio of slab to beam radial stiffness (t_s = slab thickness, R = radius to the slab/beam interface, E_{cb} = beam modulus of elasticity and A_b = cross-sectional area of beam).

The shrinkage strain sh in equation C5-1 may be taken from 2.3 for a slab of effective thickness $2t_s$, since moisture loss will occur only from the top surface.

Stresses resulting from frictional restraint of the slab sliding on its sub-base must be added to the stress computed in Eqn. C5-2. Note that since shrinkage occurs with the tank empty, the weight causing frictional restraint is the weight of the slab alone.

5.5.2

For those parts of the structure partially restrained against shrinkage, the minimum ratio of reinforcement area to gross concrete area shall be based on rational analysis of the degree of restraint to shrinkage, with reinforcement stresses not exceeding those given in table 5.2, but shall not be less than that required by 5.5.1.

Table 5.2
MAXIMUM PERMISSIBLE STRESSES IN REINFORCING STEEL
FOR STRENGTH CALCULATION

			Steel stresses (MPa)			
Type of reinforcement			Deformed bars		Hard drawn steel mesh	
Load Combinations			A	B	A	B
Members in direct tension			110	150	120	165
Members in bending	On liquid retaining face		110	150	120	165
	On face remote from liquid	Members less than 225 mm thick	110	150	120	165
		Members 225 mm or more thick	138	190	185	250
In shear reinforcement			110	150	120	165

For a design separating the floor slab from the ring beam by a movement joint, the only restraint to shrinkage will be from frictional restraint to sliding.

Similar restraint of slab shrinkage will occur in rectangular tanks where floor slab and walls are monolithic. To avoid problems, it is adviseable for the floor slab to be cast at the same time, or before, the walls are cast. Occasionally corner pours in floors and restrained roof slabs are delayed to minimise shrinkage.

C5.5.3
All structures are subject to the effects of temperature change, creep and shrinkage. The contraction which results from shrinkage, from the dissipation of the heat of hydration of the cement, and from seasonal variations in temperature can produce excessive tension in lightly reinforced restrained members. Thin sections such as walls and slabs for example, are particularly susceptible to drying shrinkage, and must therefore contain certain minimum quantity of steel in any direction in which they are restrained if they are not to be rendered unserviceable by the formation of wide cracks.

All concrete expands after placing, due to the heat of hydration of the cement. This expansion is followed by a contraction as the concrete cools to the temperature of its surroundings. This contraction can produce tension in a restrained plain concrete member which exceeds the tensile strength of the concrete and thus produces a single wide crack. Such cracking can be controlled by the provision of a relatively small amount of reinforcement in the form of small-diameter bars or fabric.

5.5.3
When hard drawn steel mesh is used, welded intersections shall not be further apart than 200 mm.

5.5.4
In concrete sections of thickness 200 mm or greater, the steel shall be distributed between two layers, one near each face of the section.

5.5.5
Where two layers of reinforcement are required by 5.5.4, no one layer shall contain less than one third of the total required area in that direction.

5.6
Bond and anchorage

5.6.1
Bond and anchorage requirements shall be as set out in Section 5 of NZS 3101.

C5.6.1
Reinforcement, especially at anchorage lengths and lapped splices, should be detailed in a manner that will minimise congestion which might inhibit the achievement of dense, void-free concrete. For this reason hooks and contact splices should not usually be used. Spliced bars which are separated by less than the greater of 1 1/2 times the bar diameter or 40 mm should be considered to be contact splices requiring compliance with Section 5 of NZS 3101. No more than one third of the bars at any cross section should be spliced within a length of 40 bar diameters unless special precautions are taken in accordance with Section 5 of NZS 3101.

5.7
Minimum cover to reinforcement

5.7.1
The minimum cover to any reinforcement shall be as specified in Section 5 of NZS 3101 with additional requirements for faces in contact with liquid as follows:

(a) For surfaces in contact with liquid the minimum cover shall be 40 mm

(b) The underside of a roof shall also be designed as a liquid retaining face except that for a shell roof the cover may be reduced to a minimum of 25 mm in both faces

C5.7.1
The minimum covers specified are minimum requirements and should be increased for aggressive liquids or abrasive conditions.

(c) For structures containing or surrounded by sewage, sewage sludge or seawater the minimum cover to any reinforcement shall be 50 mm

(d) For other liquids having a detrimental effect on the structure, approved procautions or protective coating shall be applied

(e) Where epoxy coated reinforcement is used covers may be reduced but shall be the subject of a special study.

C5.8
Floor slabs

Leaks in floor slabs can be undetected for a considerable period and in certain ground conditions even small flows can lead to severe damage. When designing floor systems the special conditions of the site should be considered. If risks and consequences warrant, then consideration should be given to providing a secondary collection system under floor slabs to give early warning of leakage. Drainage layers on a flexible membrane might provide such a collection system at a low cost.

C5.8.1
Site concrete shall not be considered as part of the structural thickness.

C5.8.2
Slabs should not be considered as unrestrained unless they are on a low friction surface and are not connected into perimeter beams or other footings. Restrained slabs should be provided with additional reinforcement to carry the retraining forces as set out in 5.5.

5.8
Floor slabs

5.8.1
The minimum thickness of floor slabs shall be 125 mm which shall be placed in one layer.

5.8.2
Minimum reinforcement for unrestrained slabs shall be as set out in 5.5 and such reinforcement shall pass through all joints not specifically designed for movement.

5.9
Roofs

5.9.1
Reinforcement concrete roofs shall be designed in accordance with the provisions of this Section as though they were in contact with the retained liquid notwithstanding any air space below the roof soffit.

5.9.2
Design loads shall be as set out in Section 2 of this Code.

5.9.3
The thickness of a dome roof shall be not less than

$$t = r\sqrt{\frac{20w}{E_c}} \quad \text{(Eq. 5-1)}$$

C5.9
Roofs

C5.9.1
The air space below roof soffits is often at 100 % humidity and the slab is exposed to severly corrosive conditions. Thus the roof slab should be designed with the same crack control philosophy as for the rest of the structure.

C5.9.2
Some of the transient loading cases for roofs are different from those in walls. For example differential temperature effects are thought to be less severe than for the case of liquid in contact with the inside face and that is recognised in Section 2.

Roofs constructed monolithically with the walls should be designed for the additional stresses due to differential shrinkage between the wall and the roof.

On large rectangular tanks with monolithic roofs the walls at the corners provide rigid constraint to shrinkage thereby inducing a severe loading condition.

It is sometimes considered worthwhile delaying the placing of the corner sections of rectangular roofs until some of the initial shrinkage has taken place.

5.9.4
The ring beam for a dome roof shall be prestressed or reinforced with sufficient tension capacity to carry the full horizontal component of the dome reaction without a contribution from hoop steel in the walls or roof.

C5.9.4
Equation 5-1 is based on the ACI 344 recommendations (ref) to avoid the possibility of buckling of the shell. It is valid for domes with rise to span ratios between 1:6 and 1:10 and r/t less than 800. A factor of safety against buckling of 5 has been adopted in formulating Eqn. 5-1.

SECTION 6
COMMENTARY

C6.1
General
Design of prestressed concrete elements of liquid retaining structures will generally be governed by those clauses of NZS 3101 Section 13 that refer to behaviour under service load conditions. In particular, reference should be made to NZS 3101 for requirements for estimating prestress losses, for anchorage design and for general servicability requirements.

C6.2
Materials
Materials and construction methods used for prestressed elements of liquid retaining structures must be of the highest quality to avoid the possibility of unexpected cracking and possible corrosion of prestressing steel. For this reason, hand-placed or mechanically placed mortar is not permitted, and until more conclusive test data are available establishing the long-term corrosion resistance of unbonded tendons, all prestressing tendons must be fully bonded.

DZ 3106
(CPT KMcI 6)

SECTION 6
DESIGN OF PRESTRESSED CONCRETE ELEMENTS

6.1
General
Design of prestressed concrete elements of liquid-retaining structures shall comply with the requirements of NZS 3101, Section 13 except as modified by the requirements of this Section.

6.2
Materials

6.2.1
Materials for prestressed concrete elements of liquid-retaining structures shall comply with the requirements of Section 3.

6.2.2
Hand-placed or mechanically-placed mortar shall not be used for prestressed elements of liquid-retaining structures.

6.2.3
Unbonded prestressing tendons shall not be used in the construction of liquid-retaining structures.

C6.3
Allowable stresses

Table 6.1 requires residual compression under long duration loads, but allows siginficant tension stresses under strain-induced load combinations or seismic loading. No distinction is made between inside and outside surfaces, as moisture levels in the concrete will be similar, except for very thick walls. Shear stresses are unlikely to govern design. However, shear associated with bending in the vertical direction should be checked by calculating the principal tension stress existing under the combined effects of vertical prestress, and shear through the wall thickness.

Figure C6.1 illustrates the procedure for a typical tank wall subjected to axial compression force P, moment M_v, and shear V, per unit length of wall. The shear force V may be found from the slope of the vertical bending moment diagram, and will be a maximum at the wall base. The axial load and bending moment combine to give a linear distribution of direct stress, (fig. C6.1b) given by:

$$f = \frac{P}{t} + \frac{12M_v.y}{t^3} \qquad \text{.......... (Eq.C6-1)}$$

The distribution of shear stress will be parabolic, (fig. C6.1c) and may be expressed as:

$$v = \frac{1.5V}{t}\left[1-\left(\frac{2y}{t}\right)^2\right] \qquad \text{.........(Eq.C6-2)}$$

6.3
Allowable stresses

6.3.1
Prestressed reinforcement
Temporary and working stresses in prestressing steel shall comply with the requirements of NZS 3101, Section 13.

TABLE 6.1 – PERMISSIBLE CONCRETE STRESSES

	Stresses in MPa			
Load Combinations	Compression	Tension in monolithic concrete	Tension across construction joints	Principal tension stress resulting from shear
Group A combinations	$0.4\ f'_c$	-0.7 MPa*	-0.7 MPa	$0.3\sqrt{f'_c}$
Group B combinations	$0.55\ f'_c$	$0.5\sqrt{f'_c}$	0**	$0.5\sqrt{f'_c}$

* i.e. residual compression of 0.7 MPa required
** refer 6.3.3.

6.3.2
Concrete and pneumatically placed mortar
Maximum stresses in prestressed concrete and prestressed pneumatically placed concrete shall not exceed the limits specified in table 6.1, except as allowed by 6.6.

From Mohrs circle for stress, the principal tensile stress will be given by:

$$f_t = \frac{f}{2} - \sqrt{\frac{f^2}{4} + v^2} \qquad \text{..........(Eq.C6-3)}$$

The distribution of f_t will be unsymmetrical, with the maximum occurring close to the centre, but offset to the side of reduced flexural compression stress, as shown in fig. C6.1(d). For most cases it will be sufficient to check f_t at the wall centre line ($y = 0$). Thus

$$f_{to} = \frac{P}{t} - \sqrt{\left(\frac{P}{t}\right)^2 + \left(\frac{1.5V}{t}\right)^2} \qquad \text{..........(Eq.C6-4)}$$

In Eqns C6-3 and C6-4 the sign convention used is compression positive.

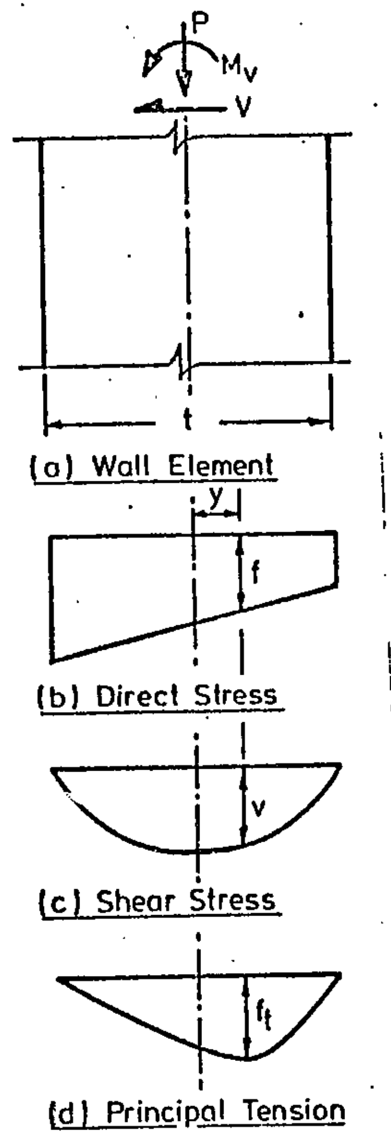

Fig. C6.1

PRINCIPAL TENSION STRESS DUE TO SHEAR

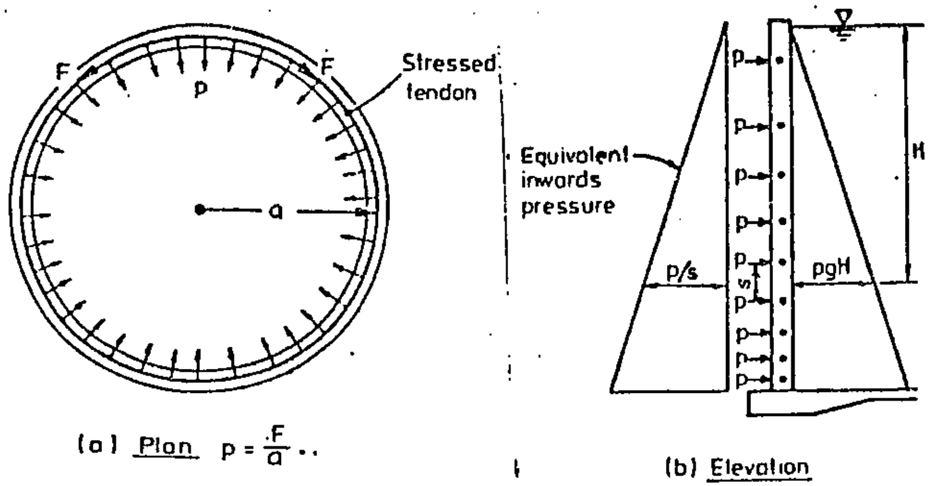

FIG. C6.2 - SIMULATION OF PRESTRESS AS RADIALLY INWARD PRESSURE

C6.4

Secondary prestress stresses

This clause refers in particular to the distribution of stress induced in walls of circular prestressed storage tanks by circumferential prestress. Vertical bending moments will be induced by circumferential prestress with the magnitude and distribution depending on the degree of base restraint, as well as the level of distribution of applied circumferential prestress force. The instantaneous vertical bending moments (secondary moments) induced by circumferential prestress can be modelled by considering the prestress to be an equivalent radially inward pressure, as shown in fig. C6.2. For tendon forces of F_p, spaced s apart, the equivalent pressure per unit height of wall will be:

$$f_p = F_p/s \qquad \text{(Eq.C6-5)}$$

6.3.3

Group B load configurations

Cracking at construction joints under Group B load combinations is permitted provided:

(a) non-prestressed reinforcement is provided across the construction joint in the tension zone to carry the entire calculated tension force, based on uncracked-section analysis, with tension stress in the reinforcement satisfying the limits of table 5.2 or

(b) the construction joint is protected against moisture ingress by an approved joint sealant, and compression stresses at the joint are calculated on the basis of a cracked-section analysis, assuming zero concrete tension capacity.

6.4

Secondary prestress stresses

6.4.1

Liquid retaining structures shall be designed for the full effects of prestressing, including secondary stresses, and time-dependent creep redistribution of prestress.

The equivalent radially inwards pressure is thus

$$p_p = f_p/a = F_p/(a.s) \text{..................(Eq.C6-6)}$$

The application of circular prestress forces to walls with pinned or moment resisting connections to the base will result in no circumferential stresses being induced at the level of the wall base because the rigidity of the base prevents development of radial displacement, and hence circumferential strain. Consequently it is common to apply some or all of the prestress with the wall initially free to slide radially. This enables compression stresses to be developed at the base of the wall.

If the base is pinned or fixed after the application of prestress, radially inwards creep displacements are restrained at the base, but may still develop at levels higher up the wall, resulting in an in-time radial displacement of the form shown in fig. C6.3. From the curvature of the wall it is apparent that vertical bending moments have been developed, whereas the initial linear deflection indicates no vertical bending.

The effect of the structural modification provided by pinning or fixing the base after prestress application, is to produce circumferential and vertical stresses that are the same as those that would result from a fraction of the prestress, p_i, being applied with the base free to slide, and the remainder, P_f (= 1 - pi) of the prestress being applied with the base pinned (or fixed). A rate-of-creep method of analysis results in the following expression for p_i.

$$p_i = e^{-Ct} \text{..............................(Eq.C6-7)}$$

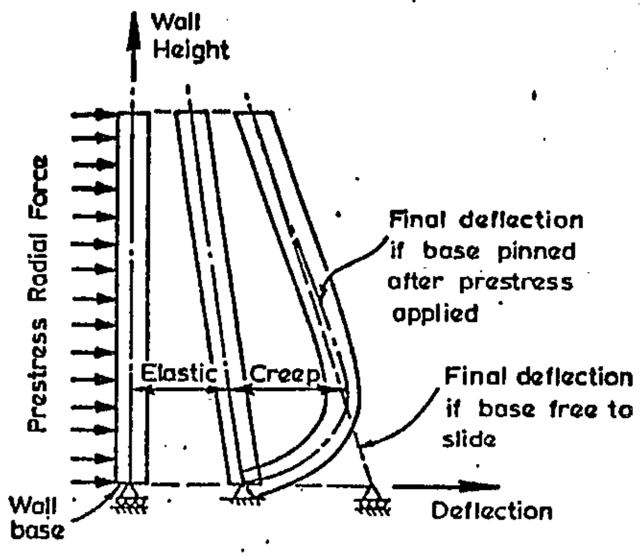

FIG. C6.3 - RADIAL DEFLECTION OF WALL UNDER PRESTRESS where C_t is the part of the creep function relative to time of applying circumferential prestress, still remaining at time of pinning the base. A typical value of C_t for a tank stressed at age = 3 months and pinned one week after stressing is $C_t = 0.9$. This results in an estimate that 40 % of the prestress may be considered to act in the initial (sliding) condition, and 60 % of the prestress may be considered to act in the final (pinned, or fixed) condition.

C6.6
Partial prestressing
This clause permits the use of partial prestressing as an alternative to full prestressing for the design of concrete elements, providing stresses in non-prestressed reinforcement, and crack widths satisfy limiting values. Concrete compression and shear stresses must still satisfy table 6.1.

In adopting a partially prestressed approach however, considerable care is needed in calculating crack widths because of the effects of creep and shrinkage. Prior to cracking, the non-

6.5
Non-tensioned reinforcement
Non-tensioned reinforcement shall be provided in prestressed elements in:

(a) end anchorage zones, as shear and bursting reinforcement

(b) between end anchorages, where prestress is calculated to be inadequate to sustain applied forces

(c) in other regions, such as construction joints, shown by analysis to require crack control.

6.6
Partial prestressing
A partially prestressed design approach, permitting cracking of concrete may be used, provided non-prestressed reinforcement is provided in the tension zone, and stresses in this reinforcment, taking full account of creep and shrinkage effects, satisfy the limits of table 5.2.

prestressed reinforcement in a partially prestressed section is subject to an initial compression stress which gradually increases due to creep of the concrete under the prestress force and also due to shrinkage. On the application of a load sufficient to reduce concrete stresses at the level of the reinforcement to zero the strain in the reinforcement is reduced by an amount equal to the initial elastic strain resulting from prestress. Thus though the surrounding concrete is at zero stress, the reinforcement is still subject to compression stress, which may be of considerable magnitude. As the load is increased to a level where cracking results, and concrete tension force is transferred to the reinforcement, the final reinforcement tension stress is effectively dictated by requirements of equilibrium of forces. The result is that the stress change in the reinforcement associated with cracking (from a compression stress at zero concrete tension, to a tension stress after crack initiation) is larger than if the effects of creep on the initial stress distribution has been ignored. Consequently the crack width will be proportionally larger. It is this change in reinforcement stress which must not exceed the allowable stress levels given in table 5.1. The residual compression stress in the reinforcement at zero concrete stress may be calculated from the expression

$$f_{sr} = E_s C_t \frac{f_c}{E_c} \quad \text{..........................(Eq.C6-8)}$$

where f_c is the average compression stress in the concrete immediately adjacent to the reinforcement, prior to decompression, and C_t is the appropriate creep factor. The influence of shrinkage is not included in Eqn. C7-8, since the normal operating conditon for the tank will be with the tank full, and thus swelling will compensate for previous shrinkage.

It should be noted that cracking of a partially prestressed tank will cause reduction of stiffness and hence a reduction of strain-induced forces, such as those resulting from thermal load. However, the reduction factor given in fig. C5.1 applies only to non-prestressed elements, and a rational analysis, taking tension stiffening effects into account must be adopted for partially prestressed elements. At the time of drafting the code, specific information on the tension stiffening characteristics of partially prestressed walls was not available, but it is felt that tension stiffening would be considerable. In this absence of such specific information the reduction factor F_t should be taken as 1.0.

SECTION 7
COMMENTARY

C7.1
General
Tanks constructed of reinforced cement mortar have been in service in New Zealand and other countries for many years.

They differ from convential reinforced concrete tanks in detailing and method of construction.

Typically they are reinforced with many layers of very small diameter reinforcing separated by separately applied layers of high strength mortar. The high specific surface of the reinforcement produces a reduced characteristic bond length resulting in diminished crack widths. Added to this is the corrosion inhibiting properties of the dense fine grained mortar with its cement rich mix providing an alkaline environment.

It therefore follows that because of the inherent properties of cement mortar some of the code provisions for reinforced concrete elements can be relaxed for cement mortar elements.

C7.2
Construction
The Code allows high stresses and smaller covers for cement mortar and consequently tolerances in construction should be closer. The manufacture and application of mortar is a highly skilled trade and it is important that high standards of construction are maintained. It is for these reasons that the Code requires that construction is only carried out by those able to demonstrate the necessary skills.

DZ 3106
(CPT KMcI 6)

SECTION 7
CEMENT MORTAR ELEMENTS

7.1
General

7.1.1
This Section applies to the elements of circular non-prestressed tanks which are constructed in multiple layers of hand placed, mechanically placed or pneumatically placed mortar.

7.1.2
Except where otherwise specified in this Section the relevant requirements of Sections 1, 2 and 3 should apply to cement mortar elements.

7.2
Construction

7.2.1
Cement mortar elements of tanks shall be constructed by specialist firms able to demonstrate satisfactory experience.

7.2.2
Continuous supervision shall be provided to the approval of the Engineer.

C7.3
Pneumatically-placed mortar

It is difficult to codity good practice in this area. An account of American practice is described in Ref. No. (ACI 344).

7.3
Pneumatically-placed mortar

7.3.1
The aggregate and cement shall be mixed and applied using special purpose equipment operated by skilled operators.

7.3.2
The velocity of the material leaving the nozzle shall be maintained uniform and such as to produce minimum rebound of sand.

7.3.3
Care shall be taken to attain uniform application behind reinforcing and other obstructions.

7.3.4
Immediately after the pneumatic mortar has been placed it shall be protected against premature drying by shading from strong sunshine and shielding from the wind.

7.3.5
As soon as pneumatic mortar has hardened just sufficiently to avoid damage, it should be thoroughly wetted and thereafter kept continuously wet for at least seven days, or alternatively protected by an approved curing compound.

7.3.6
Adequate protection by shading and shielding shall be given against fluctuations in temperature.

7.4
Hand-placed mortar

7.4.1
Every layer of mortar shall be brushed or otherwise treated after initial set to provide adequate bond for the succeeding layer.

7.4.2
The thickness of any layer of mortar shall be such that no slumping occurs.

7.4.3
Hand-placed mortar shall be cured as described in 7.3.4, 7.3.5 and 7.3.6.

7.5
Mechanically-placed mortar

7.5.1
The provisions of 7.4 shall apply also to mechanically-placed mortar.

7.6
Minimum wall thickness for watertightness

7.6.1
Where any wall is constructed entirely with pneumatically-placed mortar, the minimum thickness shall be:

(a) For factory made portable tanks up to 25 m3: Three mortar coats aggregating not less than 33 mm

(b) For other tanks not exceeding 40 m3: Four mortar coats aggregating not less than 44 mm

C7.6.1
Note to commentators. <u>Specific comment is invited on the minimum thicknesses of mortar coats set out in 7.6.1</u>.

(c) For tanks exceeding 40 m3 or with walls exceeding 3 m in height: 75 mm.

7.7
Reinforcement

7.7.1
Plain or galvanized netting, loosely linked steel wire mesh (chain netting), hard drawn steel wire and mesh, and plain and deformed bar may be used as reinforcement of mortar tanks.

7.7.2
Where netting is used the working strength shall be limited to 0.1 of the breaking strength of the effective area of the netting resolved in the direction of stress.

7.7.3
For loosely linked steel wire mesh and hard drawn steel wire and mesh the working strength shall be:

200 MPa for Group A loading combinations, and
300 MPa for Group B loading combinations

but in no case more than 0.55 f_y.

7.7.4
For plain and deformed reinforcing bar 6 mm or more in diameter the permissible stresses given in table 5.2 shall apply.

C7.7.1
An essential characteristic of cement mortar elements is a high density of closely spaced small diameter reinforcement.

C7.7.2
Convential wire netting is not regarded as a good engineering material and therefore its working strength has been downgraded in the Code to 0.1 of f_y.

C7.7.3
Other small diameter fabric such as woven or welded mesh and chain netting are considered to be satisfactory and the allowable working stress for them is given as 0.55 of f_y.

Small diameter wires and bars (less than 6 mm) are also useful for increasing the percentage of reinforcement in one direction and the allowable working stress for them is the same as for fabric.

C7.7.4
Bars of 6 mm or greater are considered to behave as reinforcement in conventional reinforced concrete and corresponding limitations on working levels are appropriate.

Plain bars of 6 mm and greater should not be used in water retaining structures where the improved bond characteristics of deformed bars will provide better control of cracking.

C7.8
Minimum cover to reinforcement
The code requirements for cover to reinforcement in cement mortar elements are reduced from those required for convential reinforced concrete because of the better corrosion resistance of cement mortar as described in C7.1.

7.8
Minimum cover to reinforcement

7.8.1
The minimum cover to reinforcement shall be as specified in 5.7, except that where mortar is hand-placed by trowel, or pneumatically-placed mortar is used, cover may be reduced to a minimum of 15 mm or 25 mm respectively.

7.8.2
Where the reinforcing layer nearest the face is galvanized the cover may be further reduced to 10 mm where the member is hand placed by trowel and to 15 mm when the member is pneumatically placed.

7.8.3
For factory made portable tanks up to 25 m3 the minimum cover on the inside face shall be not less than:

(a) 10 mm where the internal coat is applied last, or

(b) 12 mm where the internal coat is applied first.

7.8.4
Where steel-trowelled hand-placed mortar is used the outer surface shall be applied with a minimum of two layers.

7.8.5
For structures containing or surrounded by sewage, sewage sludge, or sea water the minimum cover to any reinforcement shall be 25 mm using at least two layers of hand-placed mortar, or 40 mm using pneumatically-placed mortar.

7.8.6

For other liquids having a detrimental effect on the structure, approved precautions shall be taken or a protective coating shall be applied.

7.9

Cement mortar roofs

7.9.1

Tanks may be covered with conical or domed shell roofs of cement mortar designed in accordance with established principals of structural design and adequately tied into the top of the wall.

7.9.2

The minimum thickness of a cement mortar sheel roof shall be the greater of 40 mm and the figure given by Eqn. 5-1 subject to the provisions of minimum steel coverages, as specified in 7.8, except that the thickness at the springing of the shell shall be not less than the thickness of the wall at that point.

For factory made portable tanks up to 40 m3 with galanized reinforcement the minimum thickness of a shell roof shall be 33 mm, subject to the provisions of minimum steel cover.

7.9.3

A cement mortar roof may be constructed in one layer provided that:

(a) the minimum thickness and the cover to reinforcment shall be as required for a roof construction in several layers

(b) special care shall be taken to ensure the accurate location and secure fixing of the reinforcment and

(c) special care shall be taken to prevent premature drying and to ensure adequate curing conditions.