CI/Sfb | Xt6

CINTEC

Designed Anchor Systems

SPECIFIC PROJECT TEST DATA

SECURING THE PAST FOR THE FUTURE

Independent Testing

77 Howard Street, Toronto, Canada
Testing by Halsall and Associates,
Toronto Corbel Anchors (May 1992)

Toronto Hydro Building, Canada
Testing by Halsall and Associates,
Toronto

Corbel Anchors and RAC wall tie
(September 1992)

Newark Court and Jail, Newark, NJ
Axial Pullout Tests. Testing by Testwell
Craig Materials Consultant (July 1994)

Empire State Building, New York.
Testing by LZA Technology, New York
Pull out Tests M10/3/8" dia anchor
(December, 1994)

Botanical Gardens Montreal Canada
Testing of RAC Type Anchors by L M
Sauvé (1994)

Union County Court House, Elizabeth,
NJ
Load Tests on wall anchors for shear
loads masonry and terra cotta. Testing
carried out by Testwell Laboratories and
Simpson Gumpertz and Heger Inc.
(July 1999)

PS230K and PS238K for New York
Schools Construction Authority
Cornice stabilization.
Testing carried out by Versatile
Consulting and Testing Services (July
2001)

CINTEC ANCHOR TESTING

77 Howard Street, Toronto, Canada

TYPE:

TESTING OF CORBELL ANCHORS

BY:

HALSALL AND ASSOCIATES TORONTO

(MAY 1992)

CASE HISTORY

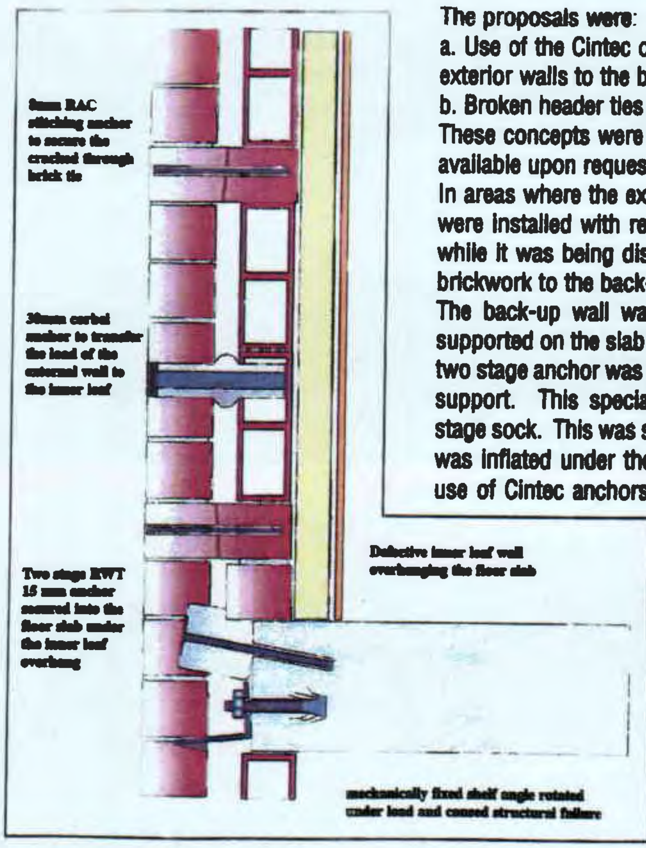
77 HOWARD STREET, TORONTO, ONTARIO, CANADA

Exterior wall restoration

This 24-storey apartment block's exterior wall consists of two wythes tied together by courses of header bricks. The exterior wythe is a glazed clay brick and is supported by a painted steel shelf angle connected at each floor into the floor slab. The inner wythe consisted of a 4" hollow concrete block back-up wall. Deterioration is due to vertical loads imposed by shortening of the structural frame. Lack of soft joints below the shelf angles to accommodate movement has resulted in;

1. Bowing of walls.
2. Crushing of over stressed units.
3. Shear failure of the header courses.
4. Rotation of shelf angles

Corrosion deterioration has also occurred in the shelf angles and connecting bolts. Due to occupation of the dwellings, complete replacement of the walls was impractical. Thus Halsall Associates in conjunction with CLS Cintec Canada participated in the development of a stabilization strategy.



The proposals were:

a. Use of the Cintec corbel anchor to transfer vertical loads from the exterior walls to the back-up walls

b. Broken header ties to be restored using Cintec stitching anchors.

These concepts were proven with full laboratory load tests. Results available upon request.

In areas where the exterior walls were beyond repair, Cintec anchors were installed with retaining plates to prevent collapse of the panel, while it was being dismantled. The anchor was used to tie the new brickwork to the back-up wall.

The back-up wall was found, during construction, to be not fully supported on the slab edge at some locations. A special RWT, 15mm two stage anchor was designed and supplied to provide the necessary support. This special two-stage anchor had an oversized second stage sock. This was secured into the floor slab, and the second stage was inflated under the inner leaf overhang to provide support. The use of Cintec anchors thus provided stabilization and repair on this project, without disturbance or relocation of the tenants.

Conclusions of the Test Report:

The test assembly failed by crushing of the concrete block interior (back-up) wythe at the corbel anchors. The observed failure load of 10.3 Kn (2295 lb) exceeded the design (service) load of 2.85 Kn (636 lb) by a factor of 3.6.

Engineers

Halsall Associates, Toronto, Ontario, Canada

Contractors

Maxim Group General Contractors, Concord, Ontario, Canada

CINTEC

I T Howard S'

LOAD TEST DATA

ROBERT HALSALL AND ASSOCIATES LIMITED CONSULTING ENGINEERS

TEST PERFORMED BY:	ROBERT HALSALL AND ASSOCIATES LTD.
PROJECT NO.:	92x722C
LOCATION:	BURLINGTON, ONTARIO, CANADA
DATE:	21 MAY, 1992
CLIENT:	CINTEC CANADA
TEST COMPONENT SUPPLIER:	CINTEC CANADA

CINTEC CANADA LTD.

COMPONENT DESCRIPTION:

Cintec Harke Cementitious Corbel and Stitching Grout Anchors.

OBJECT:

To determine the load carrying capacity in vertical shear of a masonry exterior wall system using CINTEC injection anchors to tie the two wythes together and to transfer gravity load of the exterior wythe to the interior (back-up) wythe.

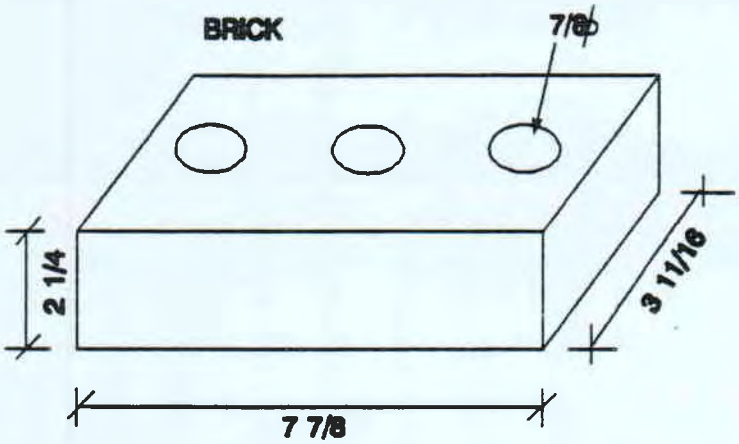
CONCLUSIONS:

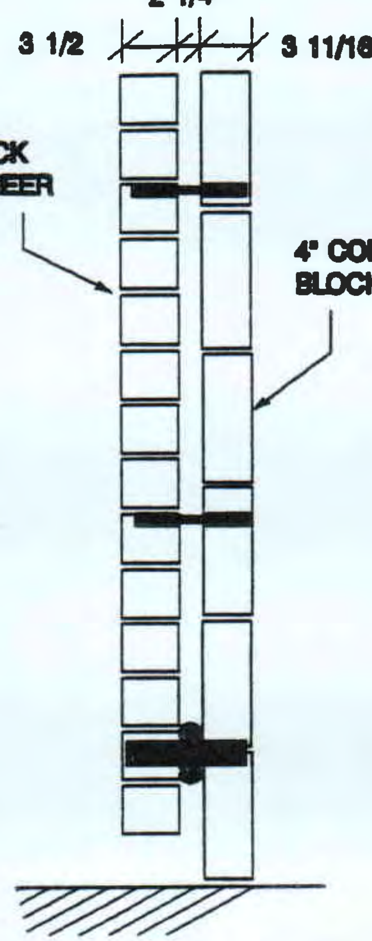
The test assembly failed by crushing of the concrete block interior (back-up) wythe at the corbel anchors. The observed failure load of 10.3 Kn (2295 lb) exceeded the design (service) load of 2.85 Kn (636 lb) by a factor of 3.6.

Test witnessed by: Eric P. Jokinen, P.Eng.

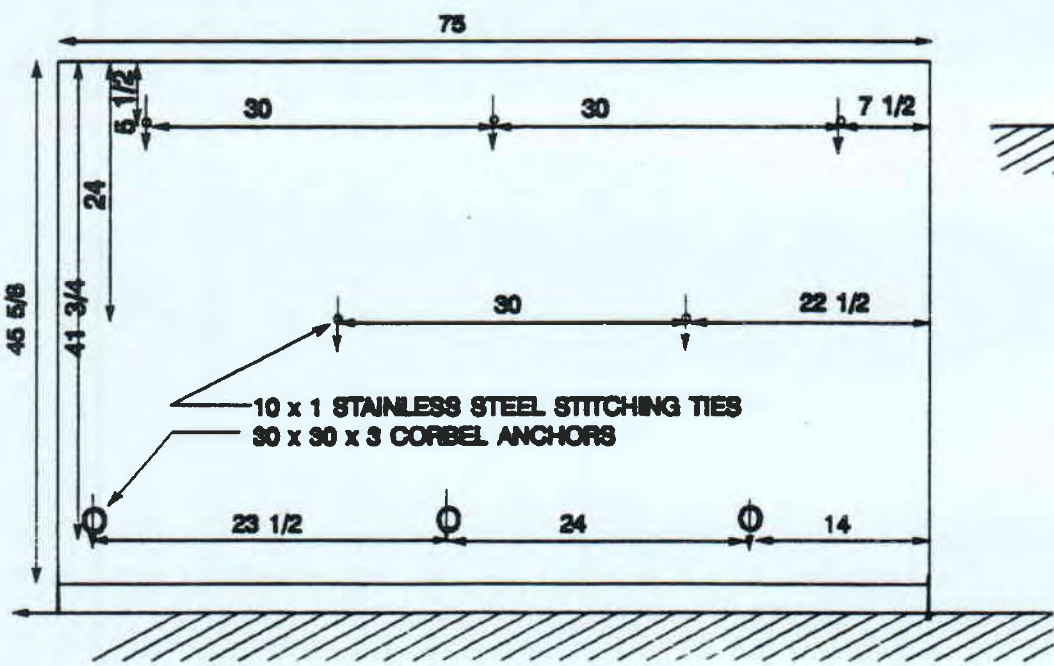
Report prepared by: Eric P. Jokinen, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Apr 12/93
ERIC P. JOKINEN
PROVINCE OF ONTARIO



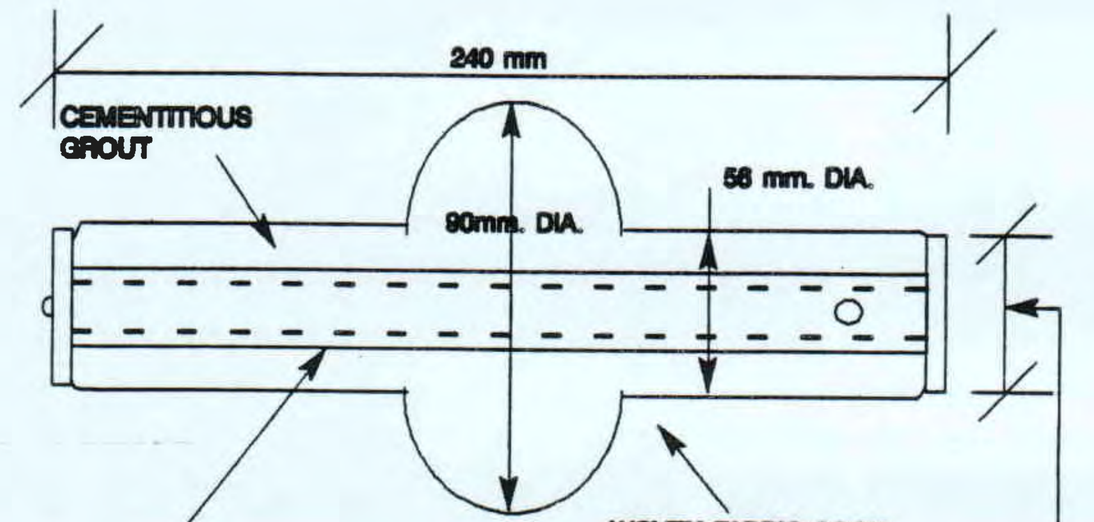


TEST WALL SECTION

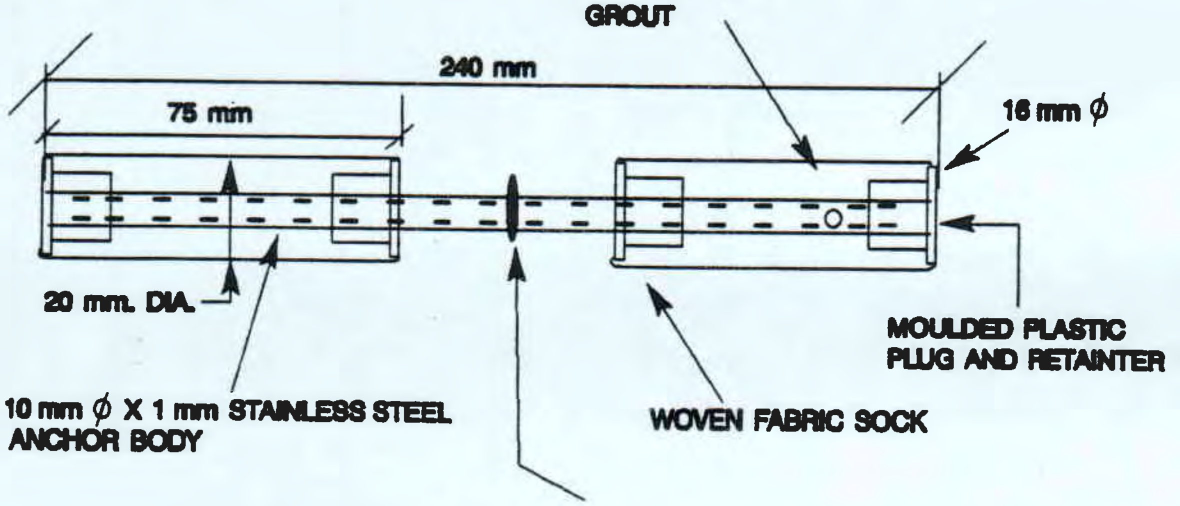


ANCHOR LAYOUT

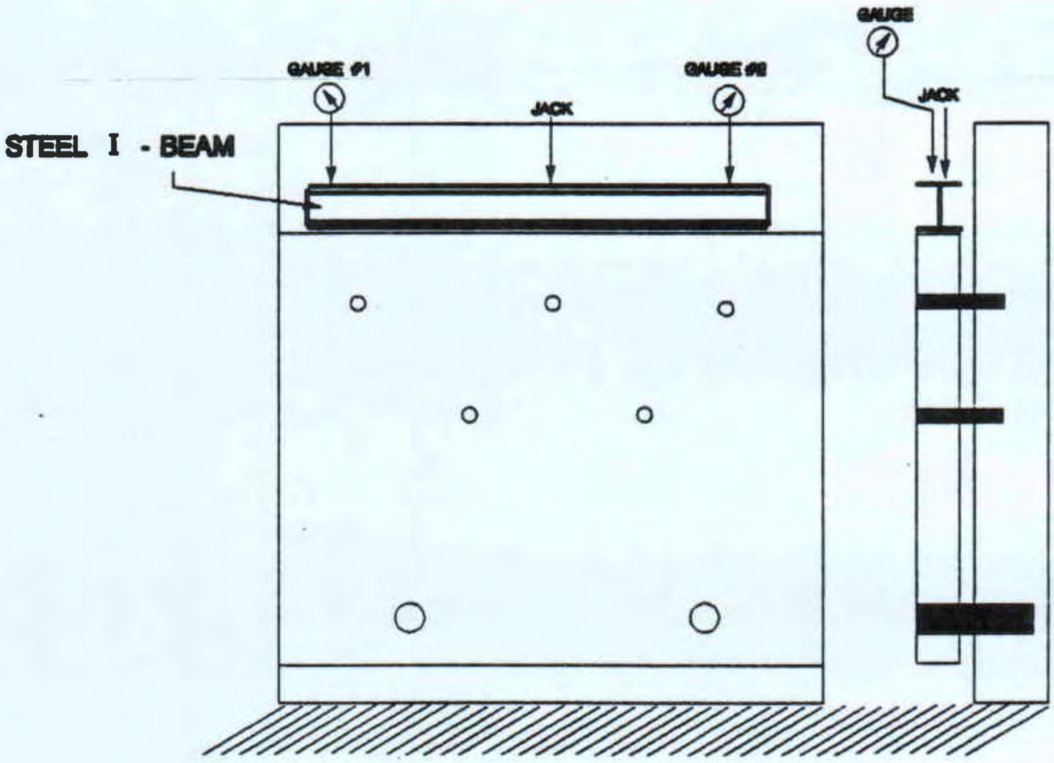
ALL MEASUREMENTS IN INCHES



CORBEL ANCHOR

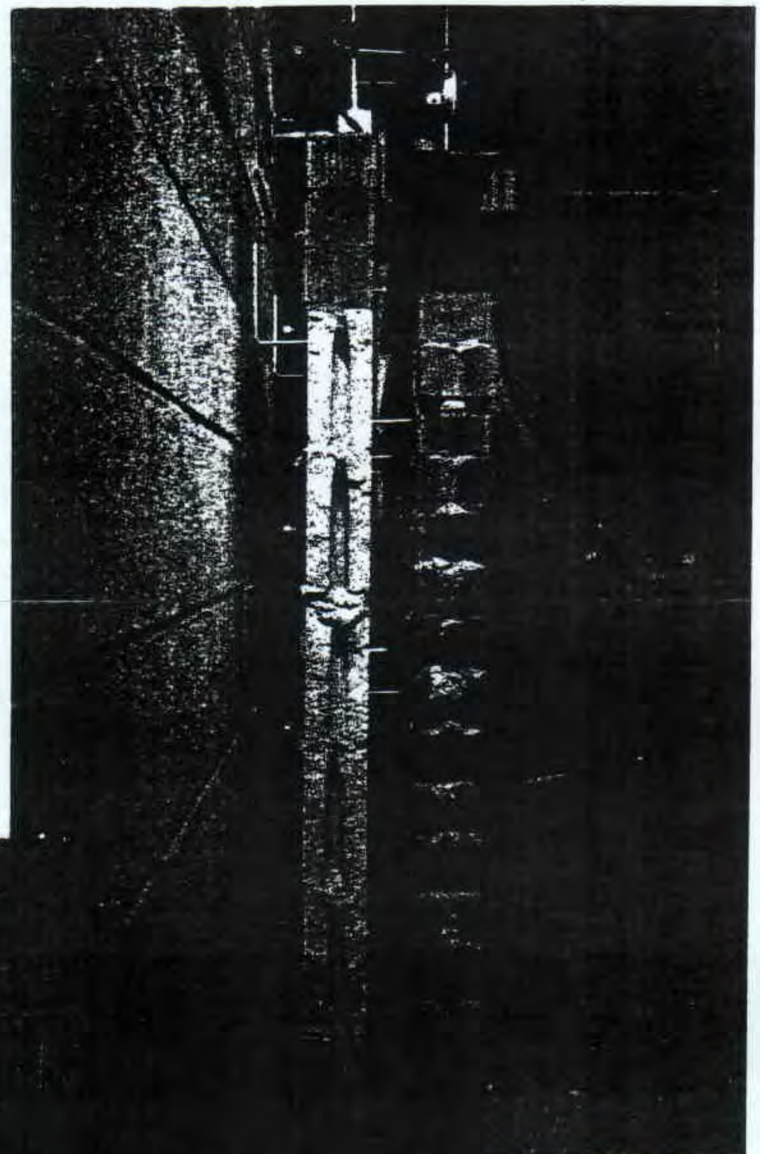


STITCHING ANCHOR



TEST SET-UP

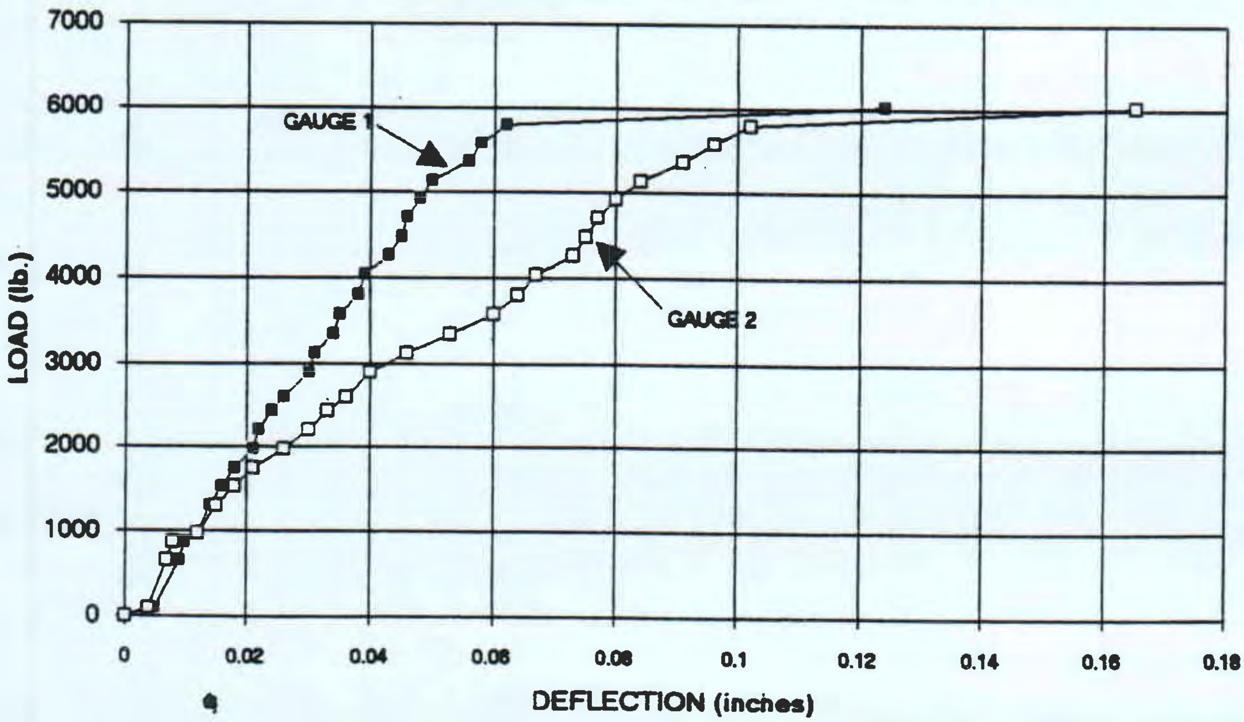
TEST SET UP PHOTOGRAPHS

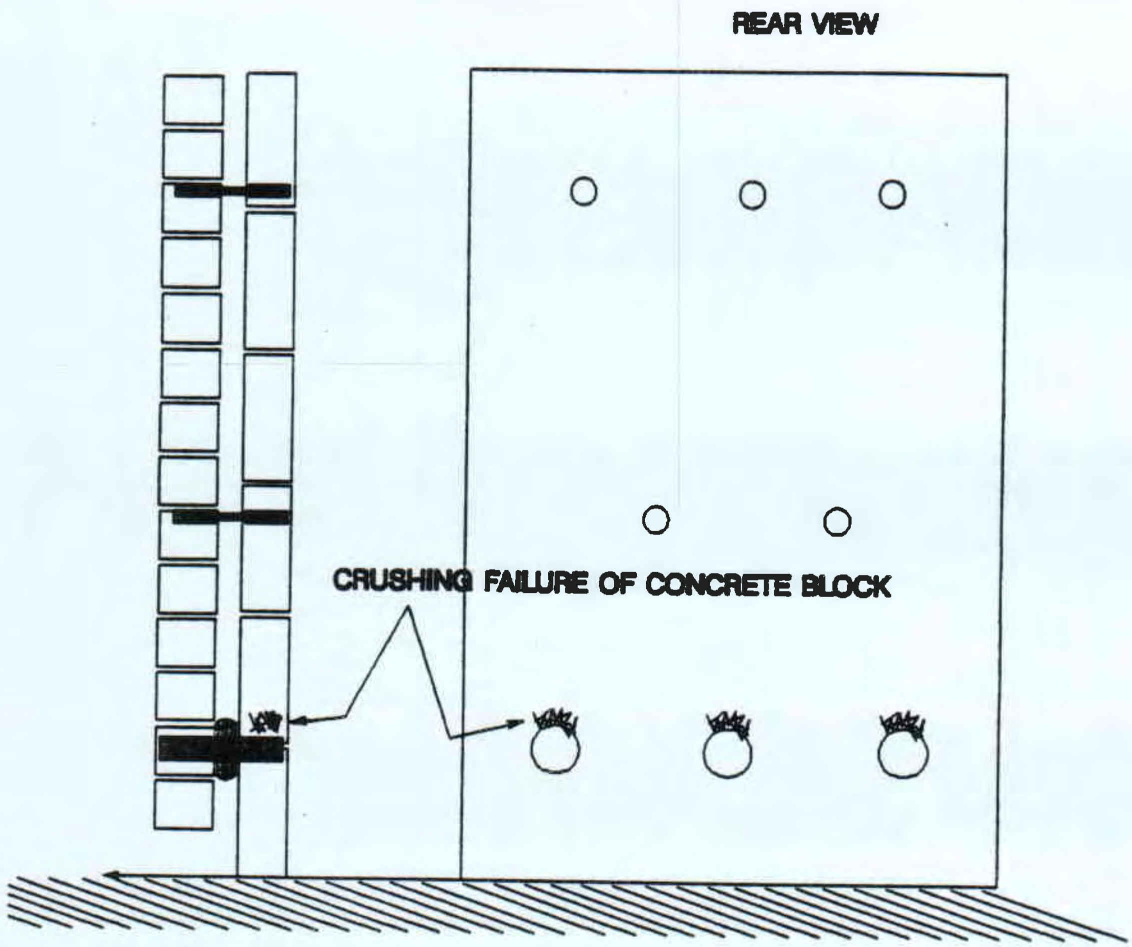


TEST BY: ROBERT HALSALL AND ASSOCIATES LIMITED
FOR: CINTEC CANADA
COMPONENT SUPPLIER: CINTEC CANADA

PROJECT NO.: 92x722C
DATE: 21 MAY, 1992

P (lb.)	DEFLECTION GAUGE 1 (in)	DEFLECTION GAUGE 2 (in)
0	1.600	1.600
109	1.595	1.596
656	1.591	1.593
875	1.590	1.592
984	1.588	1.588
1313	1.586	1.585
1531	1.584	1.582
1750	1.582	1.579
1979	1.579	1.574
2208	1.578	1.570
2437	1.576	1.567
2606	1.574	1.564
2895	1.570	1.560
3125	1.569	1.554
3353	1.566	1.547
3582	1.565	1.540
3811	1.562	1.536
4040	1.561	1.533
4260	1.557	1.527
4480	1.555	1.525
4700	1.554	1.523
4920	1.552	1.520
5140	1.550	1.516
5360	1.544	1.509
5580	1.542	1.504
5800	1.538	1.498
6020	1.476	1.435





FAILURE MODE

LOAD PER ANCHOR AT FAILURE = 8.93 Kn (2007#)
DESIGN (SERVICE) LOAD PER ANCHOR FOR THIS TEST ASSEMBLY = 2.85 Kn (636#)

TEST BY: ROBERT HALSALL AND ASSOCIATES LIMITED
FOR: CINTEC CANADA
COMPONENT SUPPLIER: CINTEC CANADA

PROJECT NO.: 92x722C
DATE: 21 MAY, 1992

MORTAR CUBE COMPRESSIVE STRENGTH

SPECIMEN		P(N)	DIMENSIONS (mm)	COMPRESSIVE STRENGTH (MPa)	AGE AT TEST
BLOCK WALL MORTAR	A1	12900	50.4 x 49.3	5.2	
	A2	13800	49.8 x 50.0	5.5	
	A3	12800	51.2 x48.95	5.1	
			AVERAGE:	5.3 MPa = 768 psi	@ 15 DAYS
BRICK WALL MORTAR	B1	20700	49.7 x 50.0	8.3	
	B2	25700	49.2 x 50.2	10.4	
	B3	24100	50.2 x 49.6	9.7	
			AVERAGE:	9.5 MPa = 1,380 psi	@ 14 DAYS
CINTEC ANCHORS GROUT	C1	111750	50.0 x 50.5	44.3	
	C2	107250	50.0 x 50.8	42.2	
	C3	110500	50.0 x 50.4	43.8	
			AVERAGE:	43.4 MPa = 6,295 psi	@ 5 DAYS

MORTAR CUBES

CUBES A1, A2, A3
CAST: 5 MAY, 1992 4:20PM TESTED: 20 MAY, 1992
MIX 1 PART TYPE 'S' MASONRY CEMENT (LAKE ONTARIO CEMENT)
3 PARTS CLEAR BRICK SAND
CLEAN TAP WATER
MIX 1/2 HOUR OLD AT TIME OF CASTING, MORTAR USED FOR BOTTOM 5 COURSES OF CONCRETE BLOCK WALL, D PALMOLIVE DISH DETERGENT RELEASE AGENT IN FORMS, CUBES REMOVED FROM FORM AT 11:00PM, 5 MAY, 1992, AND LEFT ON FLOOR OF LAB TO CURE

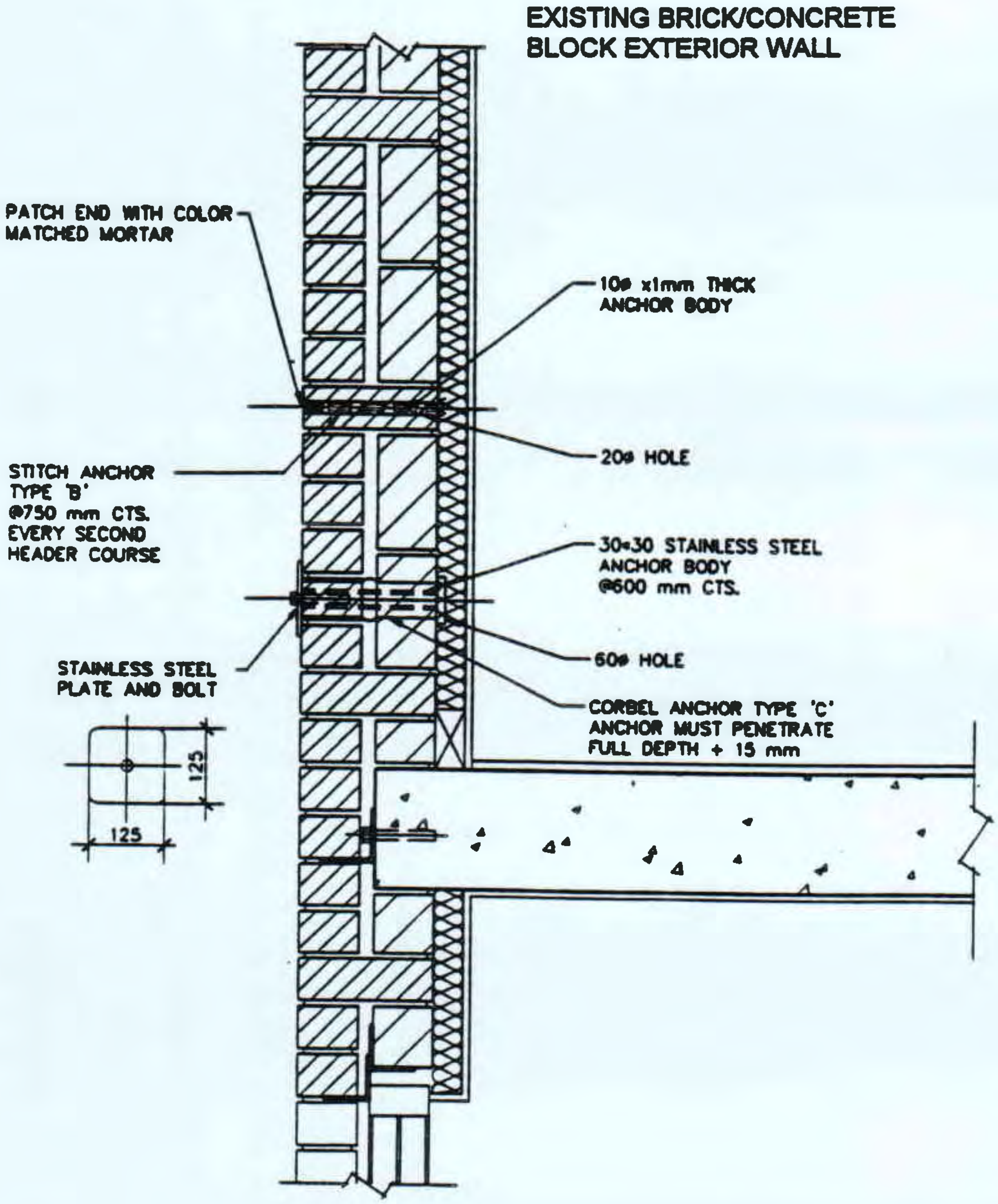
CUBES B1, B2, B3
CAST: 6 MAY, 1992 1:30PM TESTED: 20 MAY, 1992
MIX: SAME AS 1st SET
MORTAR NEWLY MADE; USED FOR BOTTOM 5 COURSES OF CONCRETE BLOCK WALL
CUBES REMOVED FROM FORM AT 7:00PM

CUBES C1, C2, C3
CAST: 15 MAY, 1992 11:00AM TESTED: 20 MAY, 1992
GROUT FOR CINTEC ANCHOR

BRICK VENEER DENSITY MEASUREMENT

1. REPRESENTATIVE SECTION OF BRICK VENEER
Represents 0.082 sq.m. of wall = 0.880 s.f.
Weight = 14.70 kg (32.93#)
Calculated Weight / Surface Area of Wall:
179.27 kg/sq.m or 37.42 p.s.f.

2. INDIVIDUAL BRICK USED IN TEST INSTALLATION
Represents 0.013 sq.m of wall = 0.138 s.f
Weight = 2.0 kg (4.48#)
Calculated Weight / Surface Area of Wall:
153.85 kg/sq.m or 36.46 p.s.f.



PROPOSED ANCHOR APPLICATION

CINTEC ANCHOR TESTING

Toronto Hydro Building Canada

TYPE:

TESTING OF CORBELL ANCHORS FOR SHEAR LOADS

BY:

HALSALL AND ASSOCIATES (SEPTEMBER 1992)

Toronto Hydro Building, Toronto

LOAD TEST DATA

ROBERT HALSALL AND ASSOCIATES LIMITED CONSULTING ENGINEERS

TEST PERFORMED BY:	ROBERT HALSALL AND ASSOCIATES LTD.
PROJECT NO.:	92x713C
LOCATION:	BURLINGTON, ONTARIO, CANADA
DATE:	9 SEPTEMBER, 1992
CLIENT:	TORONTO HYDRO
TEST COMPONENT SUPPLIER:	CINTEC CANADA

toronto hydro

CINTEC CANADA LTD.

COMPONENT DESCRIPTION:

Cintec Harke Cementitious Corbel and Stitching Grout Anchors.

OBJECT:

To determine the load carrying capacity in vertical shear of a masonry exterior wall system using CINTEC injection anchors to tie the two wythes together and to transfer gravity load of the exterior wythe to the interior (back-up) wythe.

CONCLUSIONS:

The test assembly failed by a flexural bond failure in the interior (back-up) wythe. The observed failure load of 40.6 Kn (9130 lb) exceeded the design (service) load of 5.67 Kn (1274 lb) by a factor of 7.2.

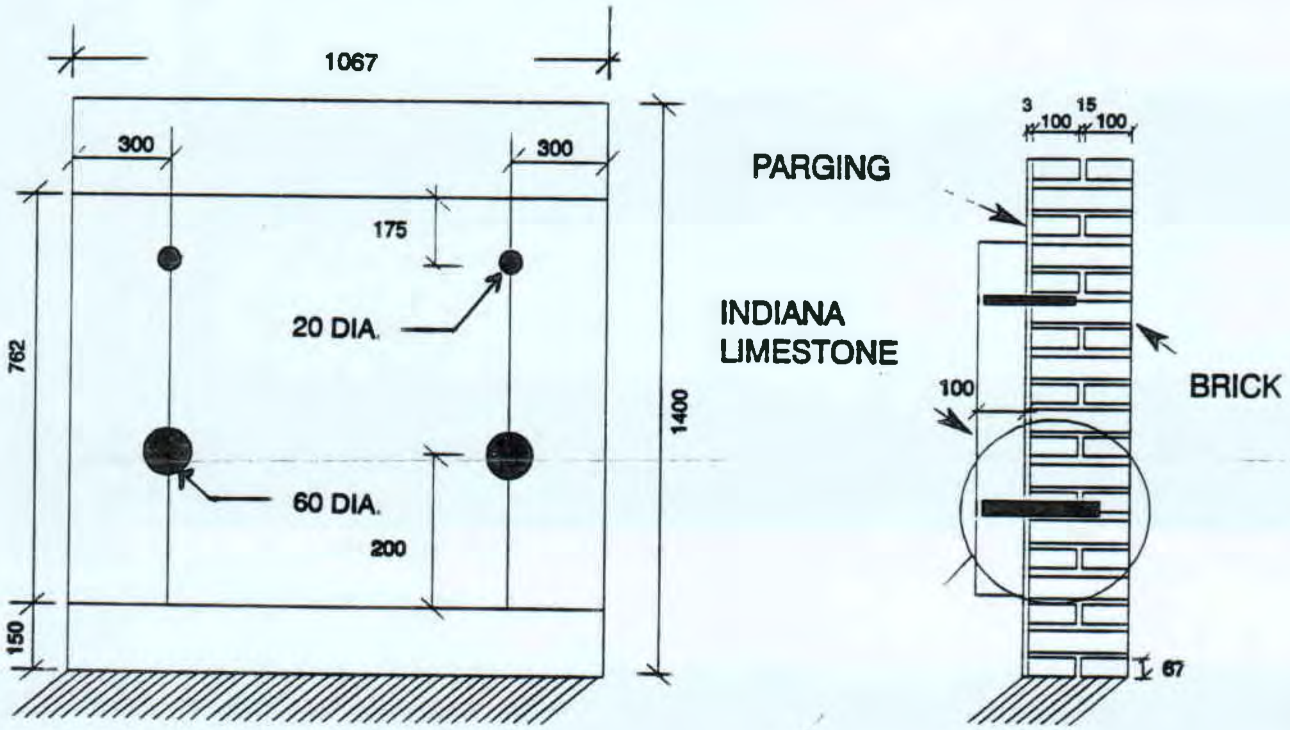
The ultimate shear capacity of the anchors could not be determined as the wall assembly failed prior to reaching the failure load of the anchors.

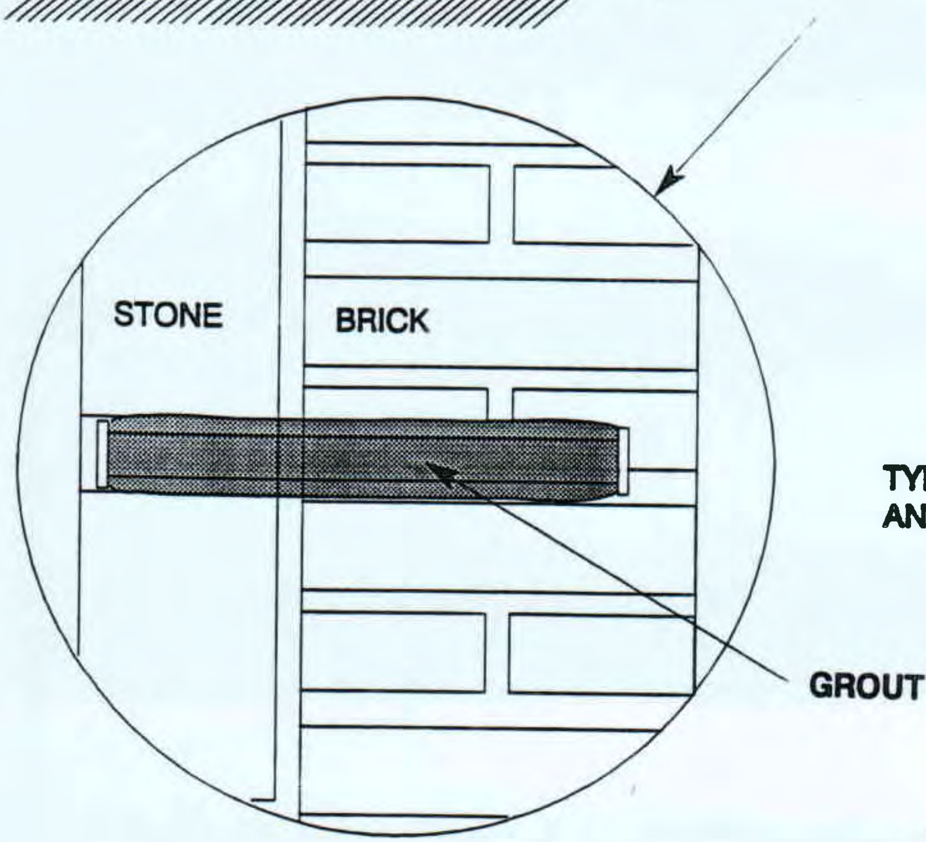
Test witnessed by: Eric P. Jokinen, P.Eng.

Report prepared by: Eric P. Jokinen, P.Eng.

LICENSED PROFESSIONAL ENGINEER
ERIC P. JOKINEN
PROVINCE OF ONTARIO

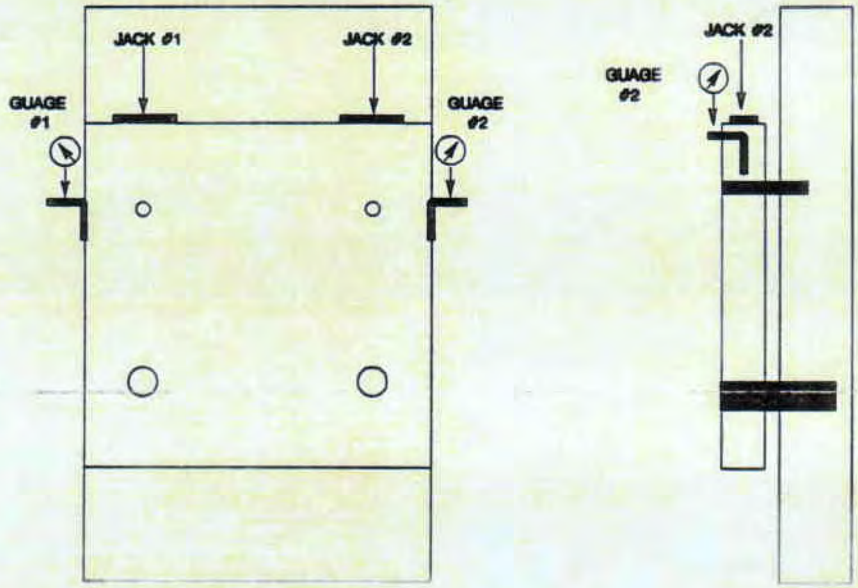
TEST ASSEMBLY DRAWINGS (all measurments in mm):



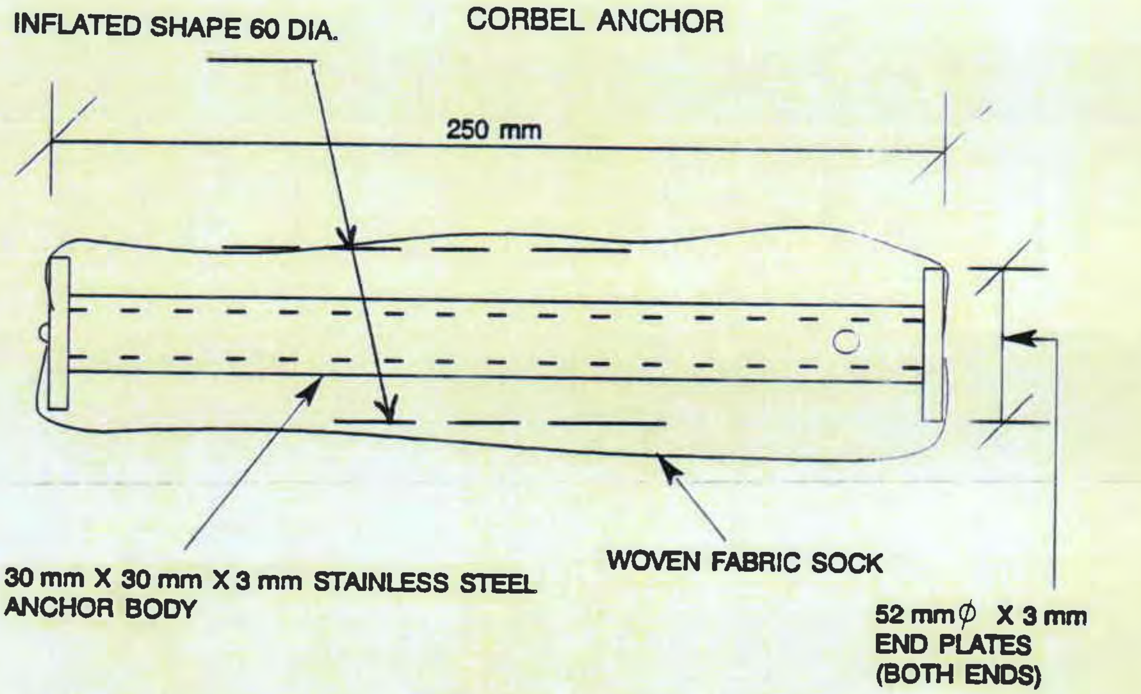


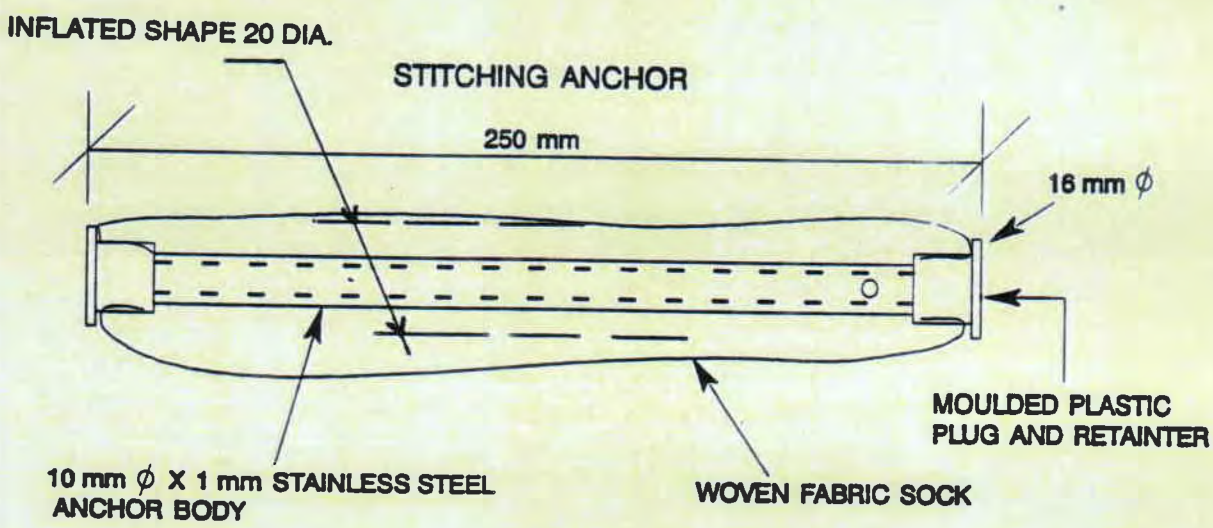
TYPICAL INSTALLED ANCHOR DETAIL

TEST SET-UP



TEST SET-UP PHOTO


CORBEL ANCHOR
INFLATED SHAPE 60 DIA.
250 mm
30 mm X 30 mm X 3 mm STAINLESS STEEL ANCHOR BODY
WOVEN FABRIC SOCK
52 mm ∅ X 3 mm END PLATES (BOTH ENDS)

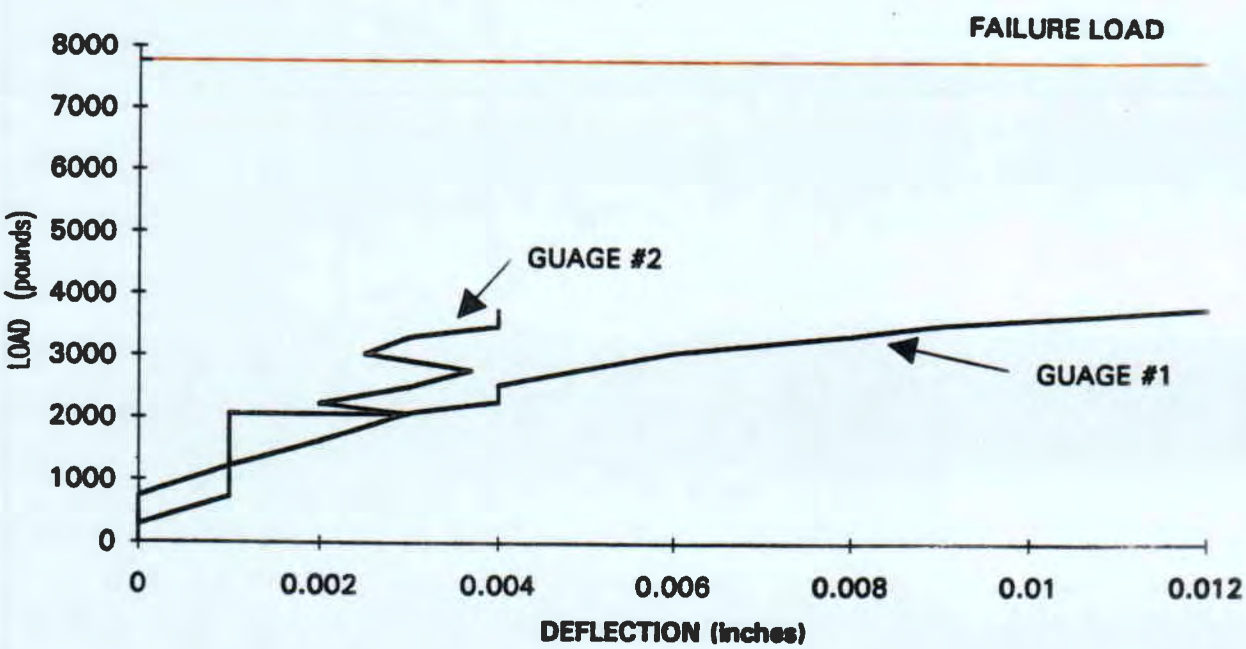

STITCHING ANCHOR
INFLATED SHAPE 20 DIA.
250 mm
16 mm ∅
10 mm ∅ X 1 mm STAINLESS STEEL ANCHOR BODY
WOVEN FABRIC SOCK
MOULDED PLASTIC PLUG AND RETAINTER

TEST BY:	ROBERT HALSALL AND ASSOCIATES LIMITED		
FOR:	TORONTO HYDRO	PROJECT NO:	92x713C/a
COMPONENT SUPPLIER:	CINTEC CANADA	DATE:	9 SEPTEMBER, 1992

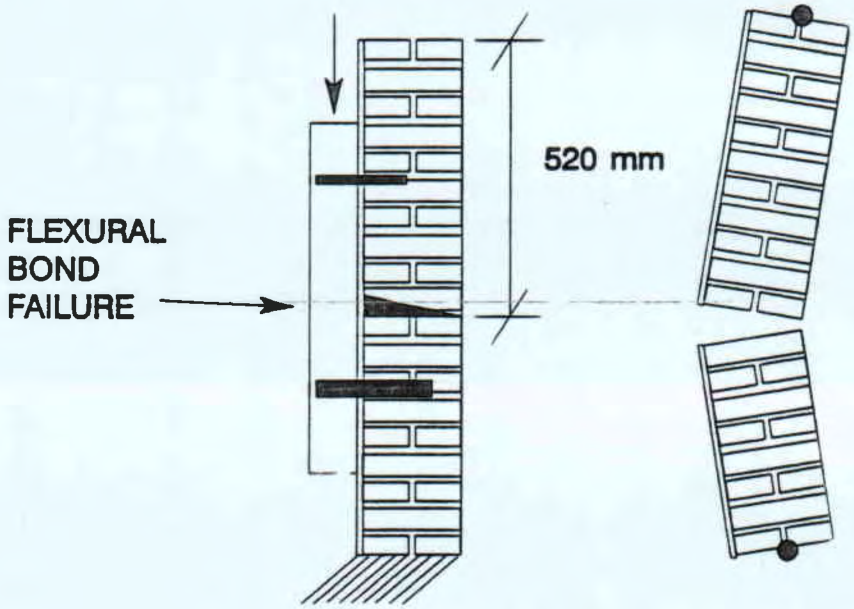
LOAD TEST

LOAD (pounds)		DEFLECTION (in.)	
JACK #1	JACK #2	GUAGE #1	GUAGE #2
0	0	0	0
279	275	0	0
729	720	0	0.001
1215	1200	0.001	0.001
1620	1600	0.002	0.001
2070	2044	0.003	0.001
2070	2044	0.003	0.003
2250	2222	0.004	0.002
2520	2489	0.004	0.003
2790	2755	0.005	0.0037
3060	3022	0.006	0.0025
3330	3289	0.008	0.003
3510	3467	0.009	0.004
3780	3733	0.012	0.004

FAILURE LOAD 15,518 POUNDS TOTAL ON 2 JACK'S



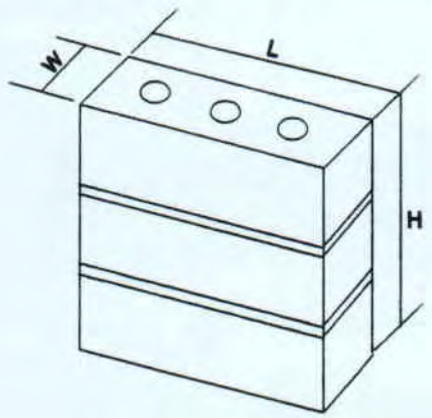
FAILURE MODE:



LOAD PER ANCHOR
AT FAILURE = 35.5 Kn
(7989#)

DESIGN (SERVICE)
LOAD PER ANCHOR
FOR THIS TEST
ASSEMBLY = 4.95 Kn
(1115#)

TEST BY: ROBERT HALSALL AND ASSOCIATES LIMITED
FOR: TORONTO HYDRO PROJECT NO. 92x713C/a
COMPONENT SUPPLIER: CINTEC CANADA DATE: 9 SEPTEMBER, 1992



TYPICAL PRISM

- PRISMS WERE CUT FROM TEST WALL
- PRISM ENDS WERE CAPPED WITH HYDROSTONE CEMENT

PRISM #	L	H	W	LOAD	COMPRESSIVE STRESS	
					(psi)	(MPa)
1	8.5"	8.25"	3.875"	116,500	3537	24.4
2	8.375"	8.25"	3.875"	103,500	3189	22.0
3	6.5"	8.25"	4"	89,000	3423	23.6
				AVG	3383	23.3

TESTS PERFORMED BY McMASTER UNIVERSITY, HAMILTON, ONTARIO

PRISMS MADE: 20 AUGUST, 1992
TESTED: 22 OCTOBER, 1992
AGE: 63 DAYS

CINTEC GROUT CUBE COMPRESSION TEST:

CUBE	COMPRESSIVE STRENGTH
A	35.5 MPa
B	34.0 MPa
C	37.1 MPa
AVG	35.5 MPa = 5148 PSI @ 11 DAYS

50 mm CUBES CAST 28 AUGUST, 1992; TESTED 8 SEPTEMBER, 1992 BY J.T. DONALD CONSULTANTS LTD.

STONE DENSITY MEASUREMENT

SAMPLE: 12" x 12" x 2.25" INDIANA GREY LIMESTONE
MEASURED WEIGHT: 11.8 Kg = 26 lbs
DENSITY: 2354 Kg/cm = 147 PCF

ROBERT HALSALL AND ASSOCIATES LIMITED
CONSULTING ENGINEERS

CINTEC ANCHOR TESTING

Essex County Court House and Jail
Newark New Jersey USA

TYPE:

AXIAL PULL-OUT TESTS

BY:

TESTWELL CRAIG MATERIALS CONSULTANT
(JULY 1994)

TCMC **TESTWELL CRAIG MATERIALS CONSULTANTS**

a Division of Testwell Craig Laboratories, Inc.

ENGINEERING REPORT

OF AN AXIAL PULL TEST ON INFLATED ANCHORS

FOR

CINTEC AMERICA INC.

(NEWARK COURTHOUSE & JAIL)

TESTWELL CRAIG MATERIALS CONSULTANTS

CONSTRUCTION MATERIALS SCIENTISTS & ENGINEERS

OSSINING, NEW YORK

TABLE 2: TEST RESULTS OF CINTEC ANCHOR PIN B

Gage Pressure (PSI)	Load (lbs.)	Holding Time (min.)	Observation
120	220	1	None
220	410	1	None
320	600	1	None
430	800	1	None
640	1280	1	None
730	1500	1	None
880	1870	1	None
1000	2170	1	None
1120	2400	1	None
1220	2590	1	None
1320	2790	1	None
1420	2980	1	None
1520	3190	5	None

Note: No visible damage to the anchor was observed at all increments of the load.

HYDRAULIC EQUIPMENT AND TEST SETUP

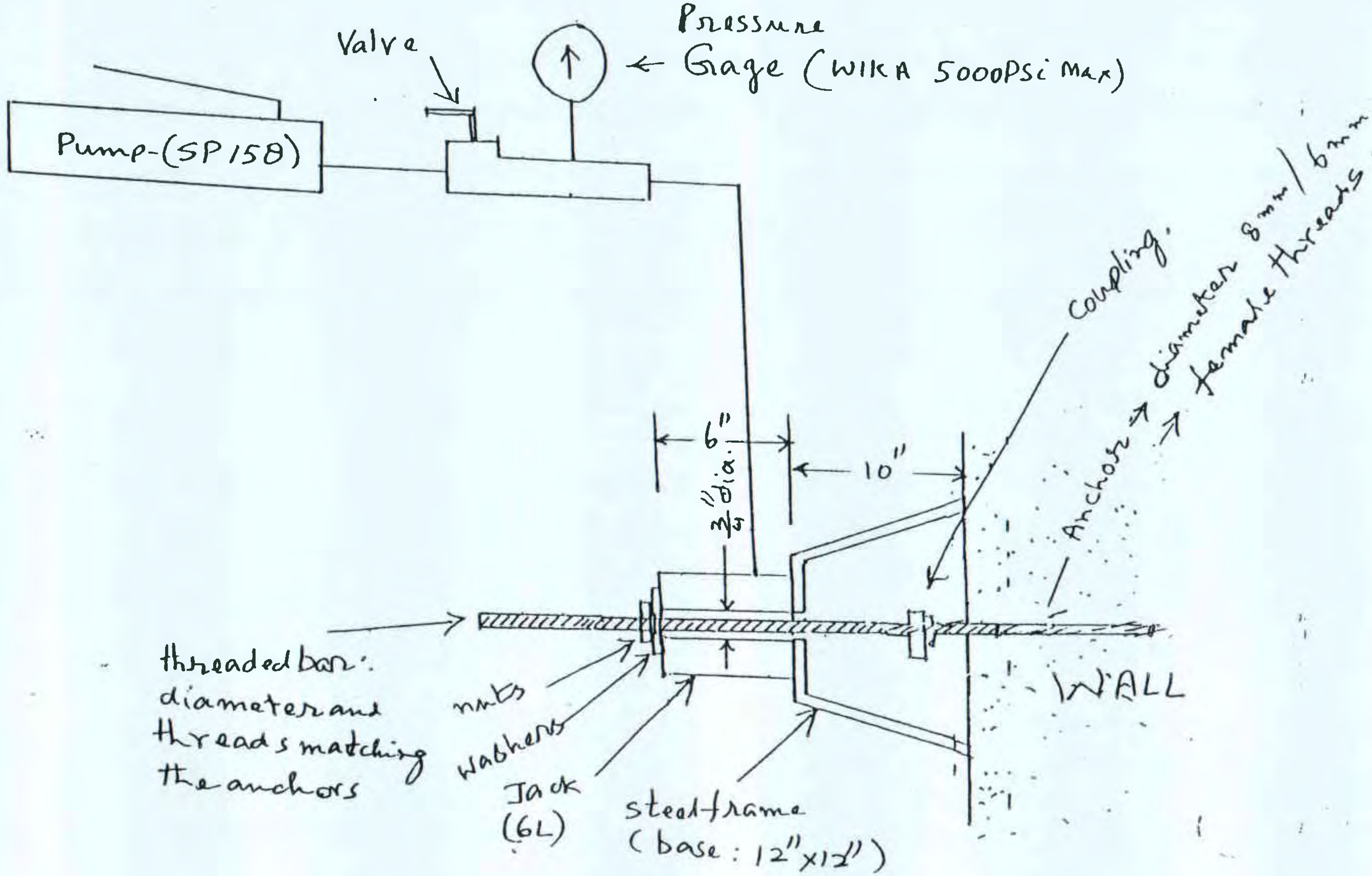
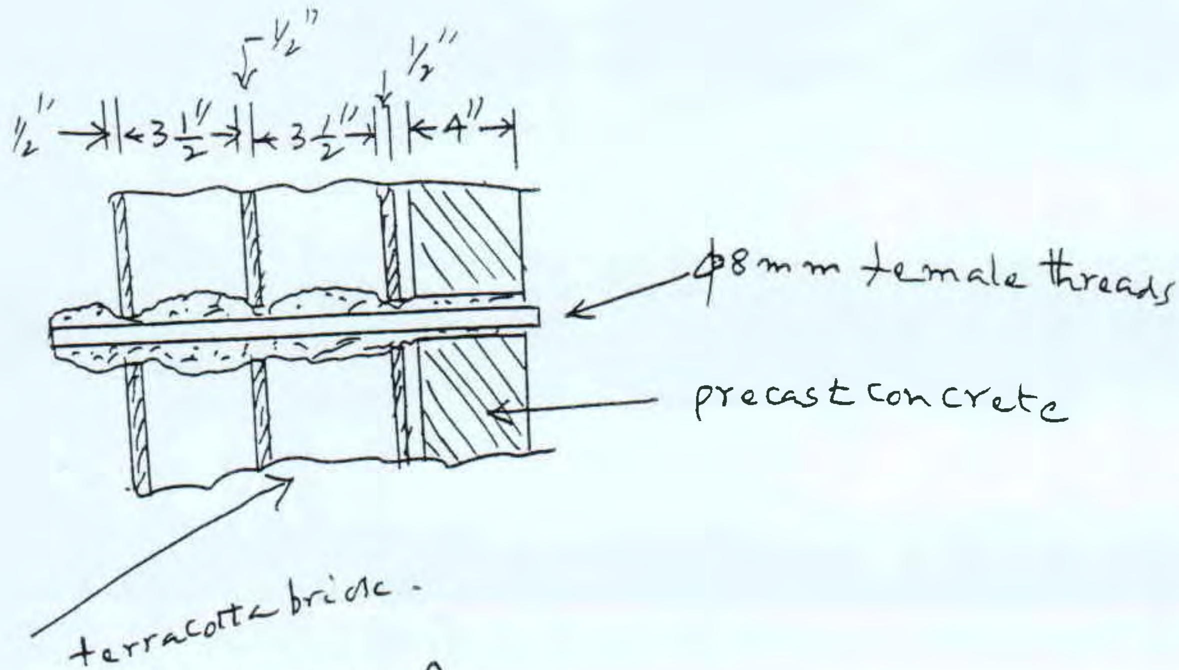
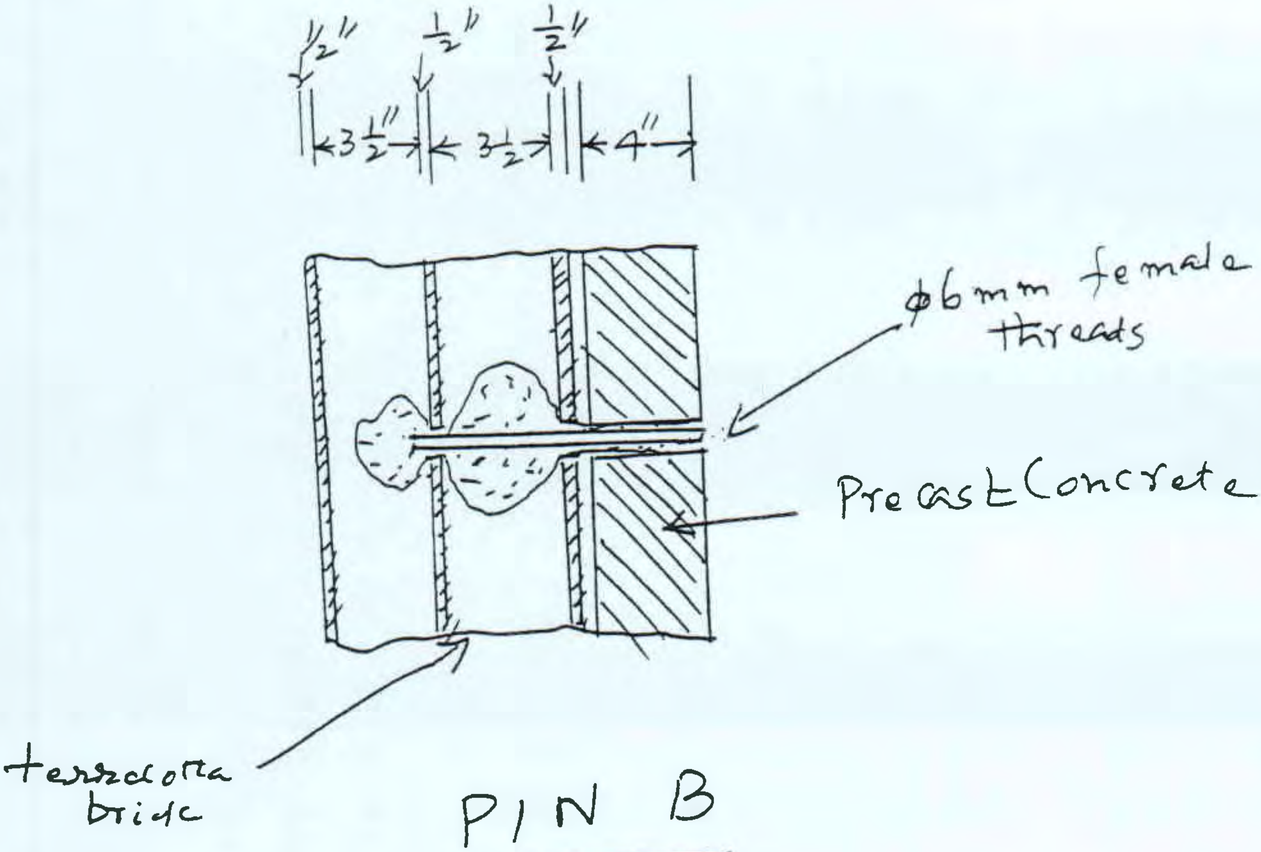


FIGURE-2



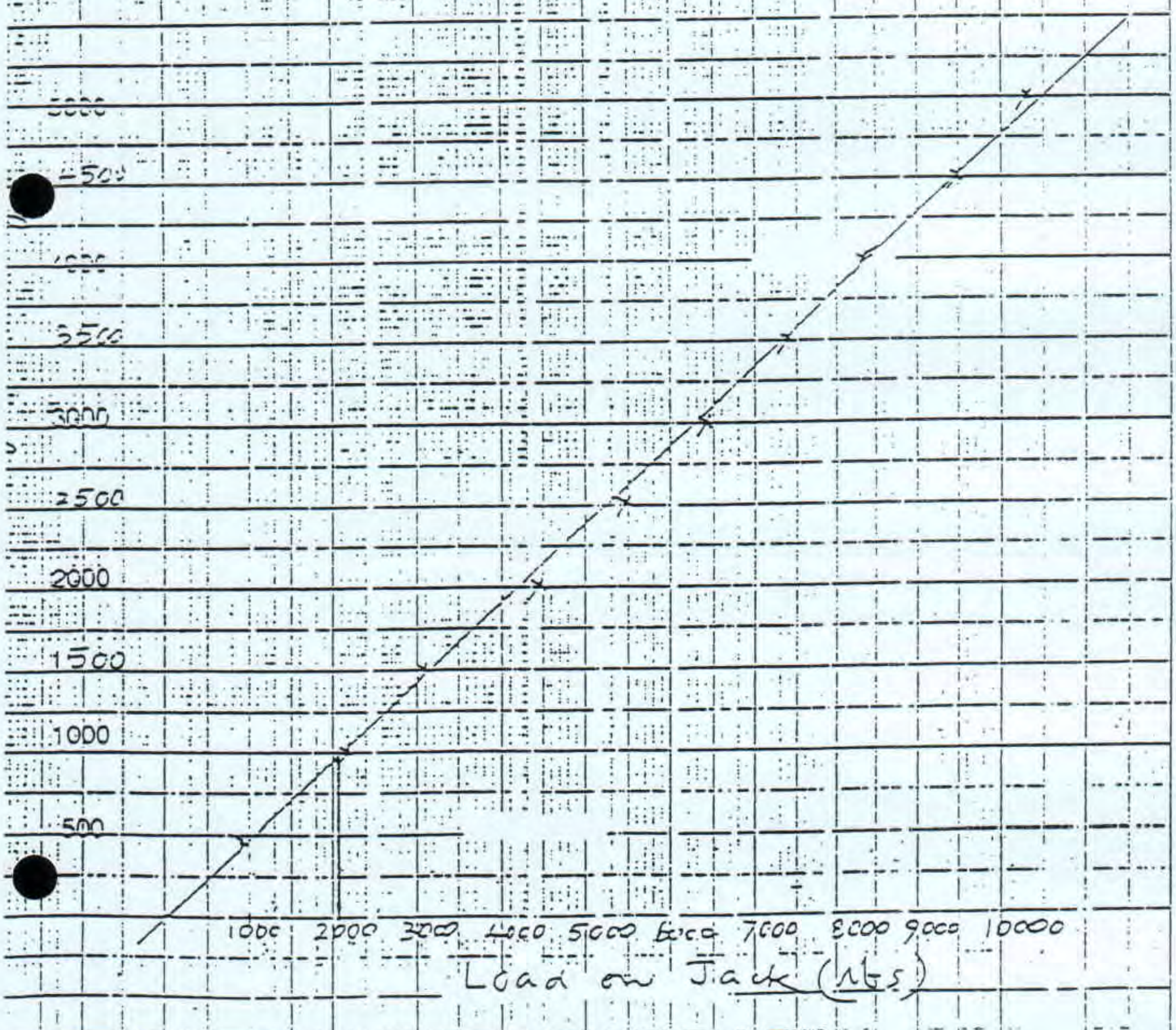
PIN A



PIN B

FIGURE-1

Calibration of Pull Test Equipment
Pump: SPL58
Gauge: WIKA 56246 5000psi
Jack: Blue Jack (L1)
Date of Calibration: 7/27/94



Annexure - 1

CALIBRATION OF PULL TEST EQUIPMENT

(Gage: WIKA 5000 psi, Jack: 6L, Pump: SP158)

Date of Calibration: 7/27/94

psi	lbs	psi	lbs	psi	lbs	psi	lbs	psi	lbs
20	37	1020	2206	2020	4507	3020	6472	4020	8381
40	75	1040	2244	2040	4547	3040	6510	4040	8429
60	112	1060	2283	2060	4587	3060	6549	4060	8477
80	149	1080	2322	2080	4627	3080	6588	4080	8525
100	187	1100	2360	2100	4667	3100	6626	4100	8573
120	224	1120	2399	2120	4707	3120	6665	4120	8621
140	261	1140	2437	2140	4747	3140	6704	4140	8669
160	299	1160	2476	2160	4787	3160	6742	4160	8717
180	336	1180	2515	2180	4827	3180	6781	4180	8765
200	373	1200	2553	2200	4867	3200	6820	4200	8813
220	411	1220	2592	2220	4907	3220	6858	4220	8861
240	448	1240	2631	2240	4947	3240	6897	4240	8909
260	485	1260	2669	2260	4987	3260	6936	4260	8957
280	522	1280	2708	2280	5027	3280	6975	4280	9005
300	560	1300	2747	2300	5067	3300	7013	4300	9053
320	597	1320	2785	2320	5107	3320	7052	4320	9101
340	634	1340	2824	2340	5147	3340	7091	4340	9149
360	672	1360	2863	2360	5187	3360	7129	4360	9197
380	709	1380	2901	2380	5227	3380	7168	4380	9245
400	746	1400	2940	2400	5267	3400	7207	4400	9293
420	784	1420	2978	2420	5307	3420	7245	4420	9341
440	821	1440	3017	2440	5347	3440	7284	4440	9389
460	858	1460	3056	2460	5387	3460	7323	4460	9437
480	896	1480	3094	2480	5427	3480	7361	4480	9485
500	933	1500	3133	2500	5467	3500	7400	4500	9533
520	982	1520	3186	2520	5506	3520	7437	4520	9565
540	1032	1540	3240	2540	5544	3540	7475	4540	9597
560	1081	1560	3293	2560	5583	3560	7512	4560	9629
580	1130	1580	3346	2580	5622	3580	7549	4580	9661
600	1180	1600	3400	2600	5660	3600	7587	4600	9693
620	1229	1620	3453	2620	5699	3620	7624	4620	9725
640	1279	1640	3507	2640	5737	3640	7661	4640	9757
660	1328	1660	3560	2660	5776	3660	7699	4660	9789
680	1377	1680	3613	2680	5815	3680	7736	4680	9821
700	1427	1700	3667	2700	5853	3700	7773	4700	9853
720	1476	1720	3720	2720	5892	3720	7811	4720	9885
740	1525	1740	3773	2740	5931	3740	7848	4740	9917
760	1575	1760	3827	2760	5969	3760	7885	4760	9949
780	1624	1780	3880	2780	6008	3780	7922	4780	9981
800	1673	1800	3933	2800	6047	3800	7960	4800	10013
820	1723	1820	3987	2820	6085	3820	7997	4820	10045
840	1772	1840	4040	2840	6124	3840	8034	4840	10077
860	1821	1860	4093	2860	6163	3860	8072	4860	10109
880	1871	1880	4147	2880	6201	3880	8109	4880	10141
900	1920	1900	4200	2900	6240	3900	8146	4900	10173
920	1970	1920	4254	2920	6278	3920	8184	4920	10205
940	2019	1940	4307	2940	6317	3940	8221	4940	10237
960	2068	1960	4360	2960	6356	3960	8258	4960	10269
980	2118	1980	4414	2980	6394	3980	8296	4980	10301
1000	2167	2000	4467	3000	6433	4000	8333	5000	10333

(Kaspal R Thumma)
Laboratory Manager

CINTEC

Pull Out Tests for Cintec Anchors carried out on the EMPIRE STATE BUILDING, NEW YORK

The Cintec Anchors were installed on December 8th, 1994 at the 6th floor elevation by David Aston from CLS Cintec America in the presence of Mr. R. Wagner LZA Technology and Mr. A. Destefano of Empire National. ½"Ø RAC anchors were used being of different lengths.

Type 1 overall length 3 ¾" socked 2 ¾" for testing in 4" thick limestone panels.

Type 2 overall length 7 ¾" socked 6 ¾" for testing in 8" thick limestone panels.

Type 3 overall length 11 ¾" socked from the rear for testing the brick back up wall.

Testing was carried out 8 days after installation by David Aston, in the presence of Mr. R. Wagner using a Mark IV Hilti Test Meter Serial No. 01632.

The loads achieved are set out on the test date result sheet overleaf. It should be noted at the achieved loads there were no failures or any visible damage to the areas surrounding the test anchors.

Cintec Ltd.
Factory Road, Newport, South Wales NP9 5FA
Tel. ++44 (0)1633 246614 Fax. ++44 (0)1633 246110
Web Site: www.cintec.com
E-mail cintec@aol.com

CLS Cintec Canada Ltd.
38 Auriga Drive, Suite 200,
Nepean, Ontario, Canada, K2E 8A5
Tel. (613) 225 3381 Fax. (613) 224 9042
E-mail cintec@ca.icomos.org

CLS Cintec America Inc.
307 West Pensylvania Ave. Towson,
Maryland 21204, U.S.A.
Tel. 1800 3636066 Fax. 1800 4611862
E-mail cintec@ca.icomos.org

CLS Cintec Australasia Pty Ltd.
40 Tyrrell Street, P.O. Box 141,
Newcastle, NSW 2300, Australia.
Tel. +61 (0)49 294841 Fax. +61 (0)49 297933
E-mail cintec@hunterlink.net.au

COMPANY ADDRESS
LZA TECHNOLOGY 641 AVENUE OF THE AMERICAS NEW YORK NY 10011

TEST ANCHORS & RESULTS

Date: 16/12/94

SITE ADDRESS
EMPIRE STATE BUILDING 5^{TH} AVENUE NEW YORK

TEST REQUIRED	TEST NO	INSTALL TIME	TYPE OF ANCHOR	LOAD REQUIRED	LOAD ACHEIVED	BASE MATERIAL	STOCK IMBEDMENT	STOCK DIAMETER	PULLOUT TIME
1) To Test Brick Facade 2) To Secure Brick Facade on 6th Floor	1	20 mm	¾" Solid Bar	1000lbs	3200lbs	STONE	2 ¾"	1"	10 MINS
	2	20 mm	¾" Solid Bar	1000lbs	2800lbs	STONE	2 ¾"	1"	10 MINS
	3	20 mm	¾" Solid Bar	1000lbs	3200lbs	STONE	6 ¾"	1"	15 MINS
	4	20 mm	¾" Solid Bar	1000lbs	3150lbs	BRICK	6"	1"	10 MINS
	5	20 mm	¾" Solid Bar	1000lbs	3000lbs	BRICK	6"	1"	10 MINS

PERSONS PRESENT ON TEST / DEMONSTRATION

PRINT NAME	COMPANY	POSITION	PHONE NUMBER
MR ROBERT WAGNER	LZA TECHNOLOGY	SENIOR PROJECT DIRECTOR	212 741 1300

COMMENTS

It should be noted that at the achieved loads no failure of the anchors was observed, in addition there was no visible damage to the areas surrounding the test anchors.

SIGNED
FOR & ON BEHALF OF CAVITY LOCK SYSTEMS LTD
POSITION:- *CLS NORTH AMERICA*

CLS CINTEC CANADA LTD.
38 Auriga Drive, Suite 200
Nepean, Ont. K2E 8A5
613-225-3381

SIGNED
FOR & ON BEHALF OF LZA TECHNOLOGY
POSITION:- *SENIOR PROJECT DIRECTOR*

CINTEC ANCHOR TESTING

Botanical Gardens
Botanical Gardens Montreal Canada

TYPE:
RAC (10 MM X 1 CHS – [3/8" DIA CHS])

TESTING ENGINEERS:
JASMIN TRUDEL ING. LM SAUVÉ (1994)

L.M. SAUVÉ (1964) Limitée

St-Léonard, May 4th, 1994

Monsieur François Robert
Gestionnaire de projet - Ville de Montréal
Services de l'Approvisionnement et des immeubles
Module des services professionnels immobiliers
Division programmes et projets
385, rue Sherbrooke est
Montréal (Québec) - H2X 1E3

Subject: **Montreal's Botanical Garden**

Sir,

You will find, included, a copy of the results obtained after some tests were done on the anchor's traction CINTEC/HARKE.

We would like to point out that the weakest results obtained are shown on tests Nos. 5, 6 and 7. Please take note that these tests were performed on the roof level where the masonry joints are totally damaged. We have to take into consideration the weakness of the terra-cota in that zone.

Hoping you will find everything to your satisfaction, we remain,

Yours truly,

L.M. SAUVÉ (1964) LIMITÉE
A Sauvé Group Company

Jasmin Trudel

Jasmin Trudel, Ing. Jr.
Project manager

Encl.

JT/gl

COMPANY ADDRESS
L.M. SAUVE 8305 LAFRENAIE 329-3399

TEST ANCHORS & RESULTS

DATE 94 5 14.

SITE ADDRESS
JARDIN BOTANIQUE DE MONTRÉAL

TEST REQUIRED	TEST NO	INSTALL TIME	TYPE OF ANCHOR	LOAD REQUIRED	LOAD ACHEIVED	BASE MATERIAL	SOCK IMBEDMENT	SOCK DIAMETER	PULLOUT TIME
JARDIN	1	27/4/94			11 KN	2400 LBS	10.5 POUCES		29/4/94
BOTANIQUE	2	"			9.5 KN	2000 LBS	"		2/5/94
	3	"			13.1 KN	2900 LBS	"		"
	4	"			11 KN	2400 LBS	"		"
	5	"			7.6 KN	1650 LBS	"		"
	6	"			5.5 KN	1200 LBS	"		"
	7	"			5.0 KN	1100 LBS	"		"

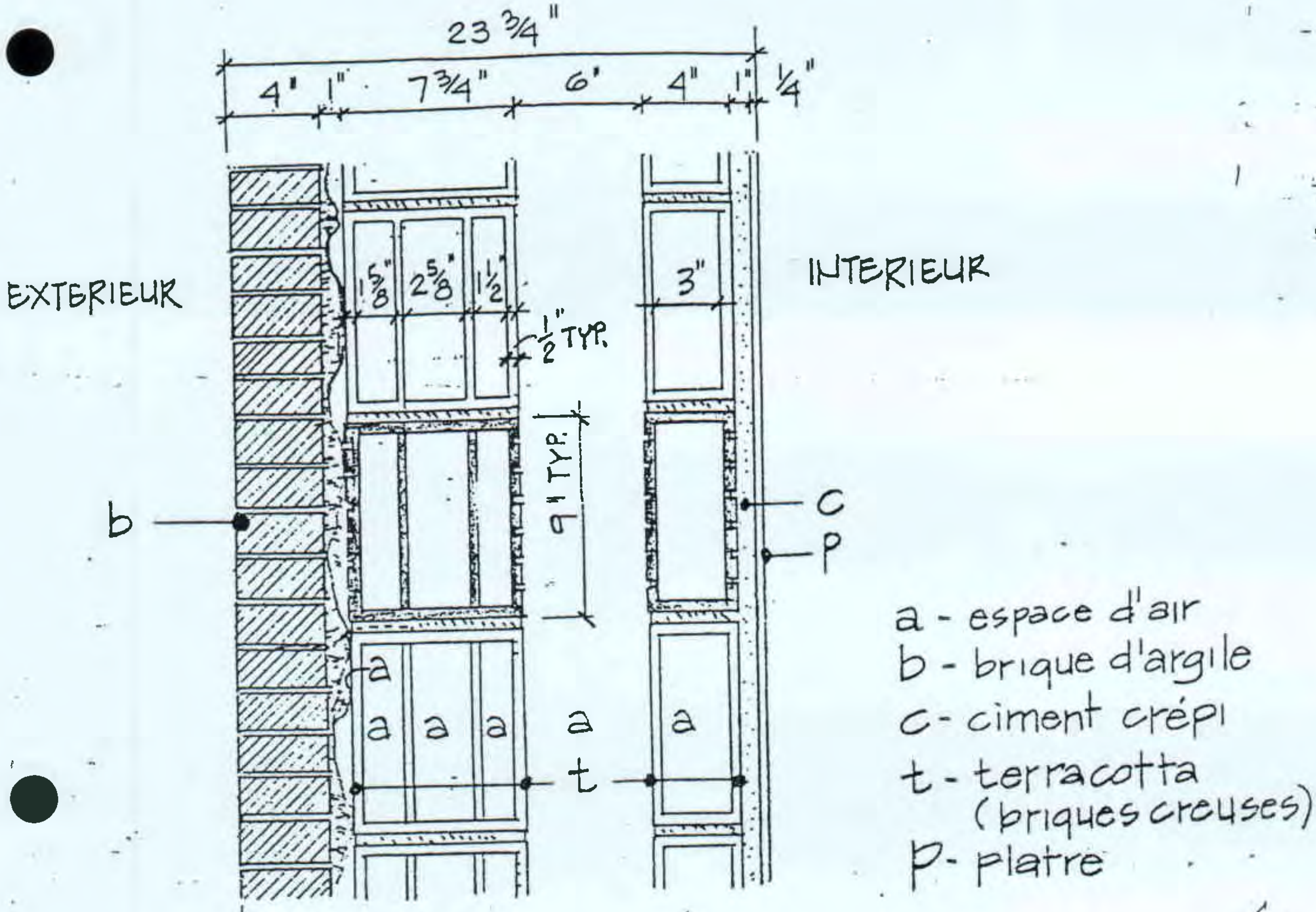
PERSONS PRESENT ON TEST/DEMONSTRATION

PRINT NAME	COMPANY	POSITION	PHONE NUMBER
JASMIN TRUDEL	L.M. SAUVE.	DIRECTEUR DE PROJET	329-3399

COMMENTS

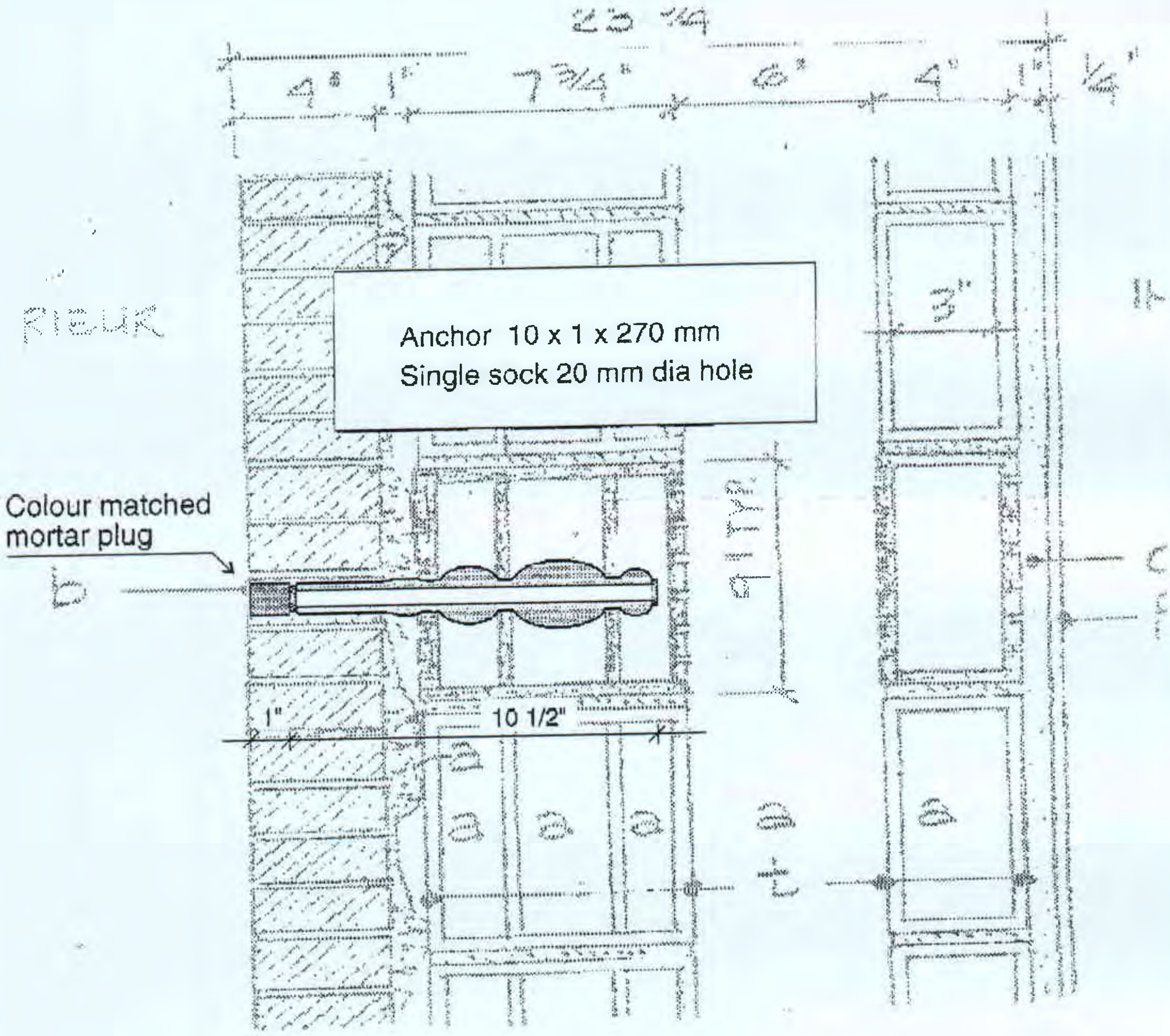
SIGNED Jasmin Trudel ing. jr.
FOR & ON BEHALF OF CAVITY LOCK SYSTEMS LTD.
POSITION :-

SIGNED Christian [illegible]
FOR & ON BEHALF OF Ville de Montréal
POSITION :-



MUR TYPE I (TYMPAN)

PAVILLON ADMINISTRATIF DU JARDIN BOTANIQUE
VILLE DE MONTREAL

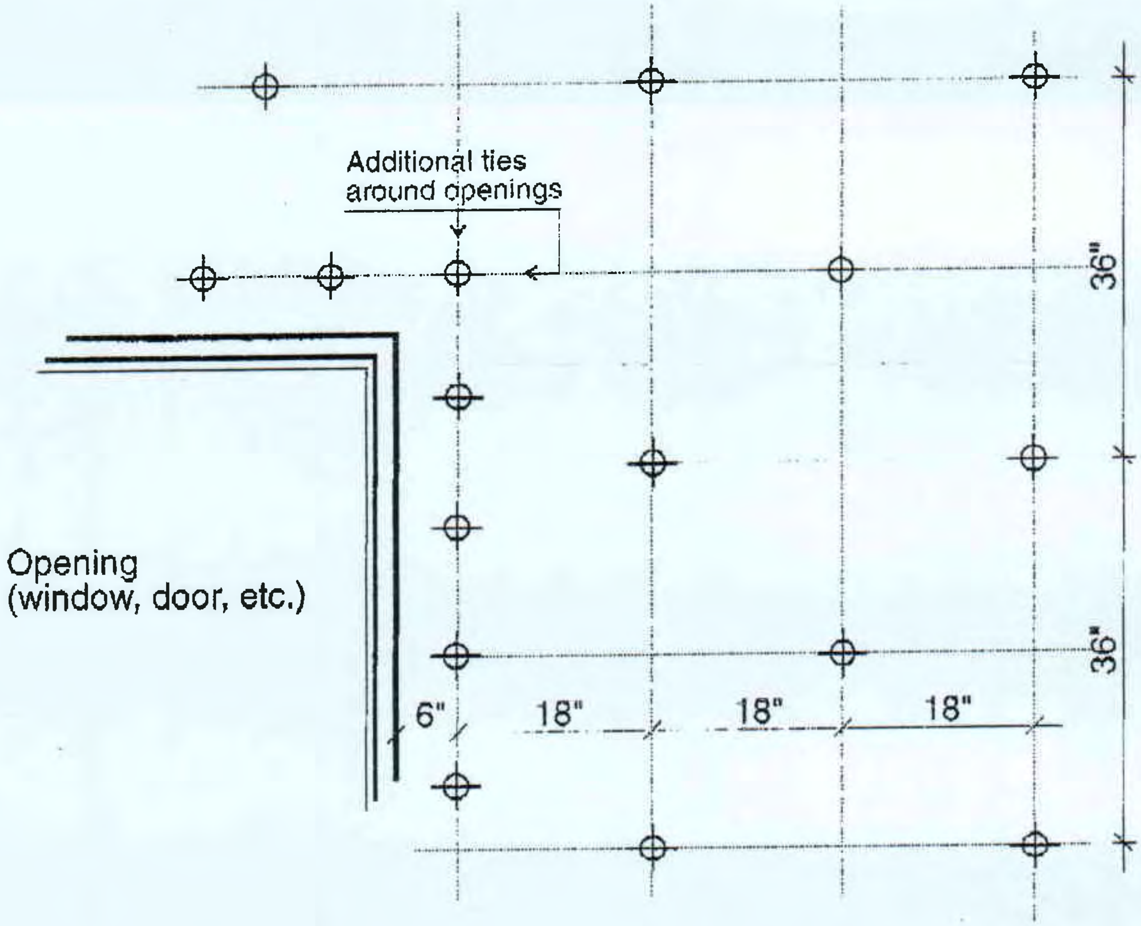


23 1/4
4"
1"
7 3/4"
6"
4"
1"
1/4"
3"
RIEUR
Anchor 10 x 1 x 270 mm
Single sock 20 mm dia hole
Colour matched
mortar plug
b
1"
10 1/2"
a
a
a
a
a
a
b

CINTEC HARKE

Anchor Layout

18 x 36system



CINTEC ANCHOR TESTING

Union County Court House
Elizabeth New Jersey USA

TYPE:

SHEAR LOAD TESTS FOR MASONRY AND TERRA-COTTA

BY:

TESTWELL LABORATORIES
AND
SIMPSON GUMBERTZ AND HEGER
(JULY 1999)

TESTWELL
LABORATORIES INC

Corporate Headquarters
47 Hudson Street
Ossining, N.Y 10562

Tel (914) 762-9000
Fax (914) 762-9638
Environmental Fax: (914) 941-4154

ENGINEER'S REPORT

Client: CINTEC AMERICA INC.
Project: Union Country Courthouse
2 Broad St. Elizabeth, NJ
Engineer: K. R. Thumma, P.E.
Technicians: G. Ramkaran, A. Uzhca,
C. Suarez, C. Clark

Lab #: MIA-001
Report#: MC-01
Date Of Testing: 7/21/99

Re: Load Tests on Wall Anchors.

INTRODUCTION

This report presents results of load tests wall anchors performed by Testwell Laboratories, Inc. on 7/21/99 at the above referred project site. Eight anchors, six of type HSS 15mm X 15mm and two of type RACCHS 10mm, 3/8" dia., were tested. These anchors were installed by the client in a brick masonry wall prior to arrival of personnel of Testwell Laboratories, Inc., at the job site Mr Robert Lloyd-Rees of CLS CINTEC AMERICA INC. and Mr. Mark Berman (part of the tests) of Michael Zemsky Architects witnessed the tests. The locations of anchors tested are shown in Figure 1.

TEST PROCEDURE

Two types of tests were performed. (1) Sheer Load Test and (2) Pull out load tests.

Sheer load tests were performed by hanging 50 lb. weights as shown in Figure 2 and Photos 1 and 2. Each weight of 50 lbs. was added slowly one by one to the trolley hung from the anchor with a steel strand. Shear tests were performed on anchors A, B and C.

Pull out load tests on anchors D, E and F were performed by using a hydraulic jack setup shown in Figure 3 and Photos 3 and 4. This setup includes a hydraulic pump, a hydraulic jack, a pressure gauge, a valve, hoses and a threaded rod. The jack is supported on steel frame resting on the brick wall and around the anchor. The anchor is connected to the jack with a box coupling and threaded bar. When the pump is operated, the pressure of hydraulic fluid increases and load is applied on the piston of the jack. The pressure of the hydraulic fluid is measured by the pressure gauge. The gauge pressure indication in psi is calibrated to the load applied on jack and consequently on the anchor.

Pullout tests on the anchors G and H were performed by using a HILTI Mark V tester shown in Photo 5. This equipment was furnished by the client.

TEST RESULTS

Table 1 shows all the results of load tests performed on the anchors indicating type of anchor, test apparatus used and the load applied along with observations on the behavior of anchors as the load is applied to different levels. The locations of the anchors on the brick wall are shown in Figure 1.

TESTWELL LABORATORIES, INC.

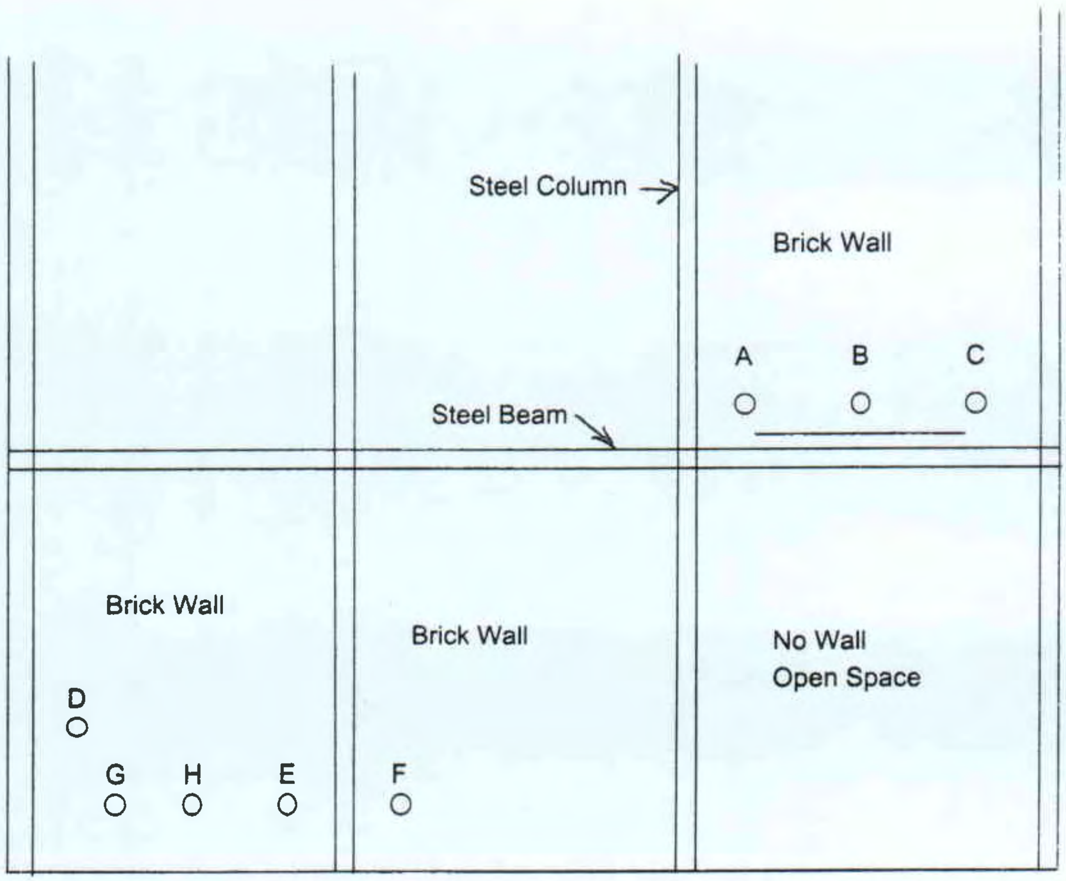
STATE OF NEW YORK
KASPAL REDDY THUMMA
074294
LICENSED PROFESSIONAL ENGINEER

Kaspal R. Thumma, Eng. Sc. D, P.E.
Vice President
KRT/SK

Table 1: Results of Load Tests Performed on CINTEC Anchors

Test No.	Type of Test	Anchor Tested	Method of Testing	Load Applied (lbs)	Observations
1	Shear Load at 2" away from wall	C	Hanging Weights	900	Loading tackle slipped bending lever arm
2	Shear Load at 2" away from wall	B	Hanging weights	1000 1150	Started deflection of lever arm Deflection increased. Test stopped
3	Shear Load at 2" away from wall	A & B together	Hanging weights	2050	No signs of failure or bending. Test stopped
4	Shear Load at 2" away from wall	A	Hanging weights	1700	Started bending Test stopped.
5	Shear Load at 2" away from wall	B	Hanging weights	1500 2050	Started bending. Bent by 1" at load line. Test stopped.
6	Pull out Test	E	Hydraulic Jack	1900 2500	Anchor started slipping out of wall Anchor slipped out of wall chipping brick around it. Maximum load.

Test No.	Type of Test	Anchor Tested	Method of Testing	Load Applied (lbs)	Observations
7	Pull out Test	F	Hydraulic Jack	2200 2800	Anchor started slipping out of wall Anchor slipped out of wall. Maximum load.
8	Pull out Tests	D	Hydraulic Jack	1900 2200	Anchor Started slipping out of wall Anchor slipped out of wall. Maximum load.
9	Pull out Tests	G	HILTI MARK V	2900 3300	Anchor started slipping out of wall Anchor slipped out of wall. Maximum load.
10	Pull out Tests	H	HILTI MARK V	3200 4000	Anchor started slipping out of wall. Maximum load Anchor slipped out of wall. Maximum load.



Anchor Types: A, B, C, D, E, F - HSS 15mmX15mm, 6" embedment
G, H - RAC CHS 10mm, 3/8" Dia, 7" embedment

Figure 1: Locations of Test Anchors

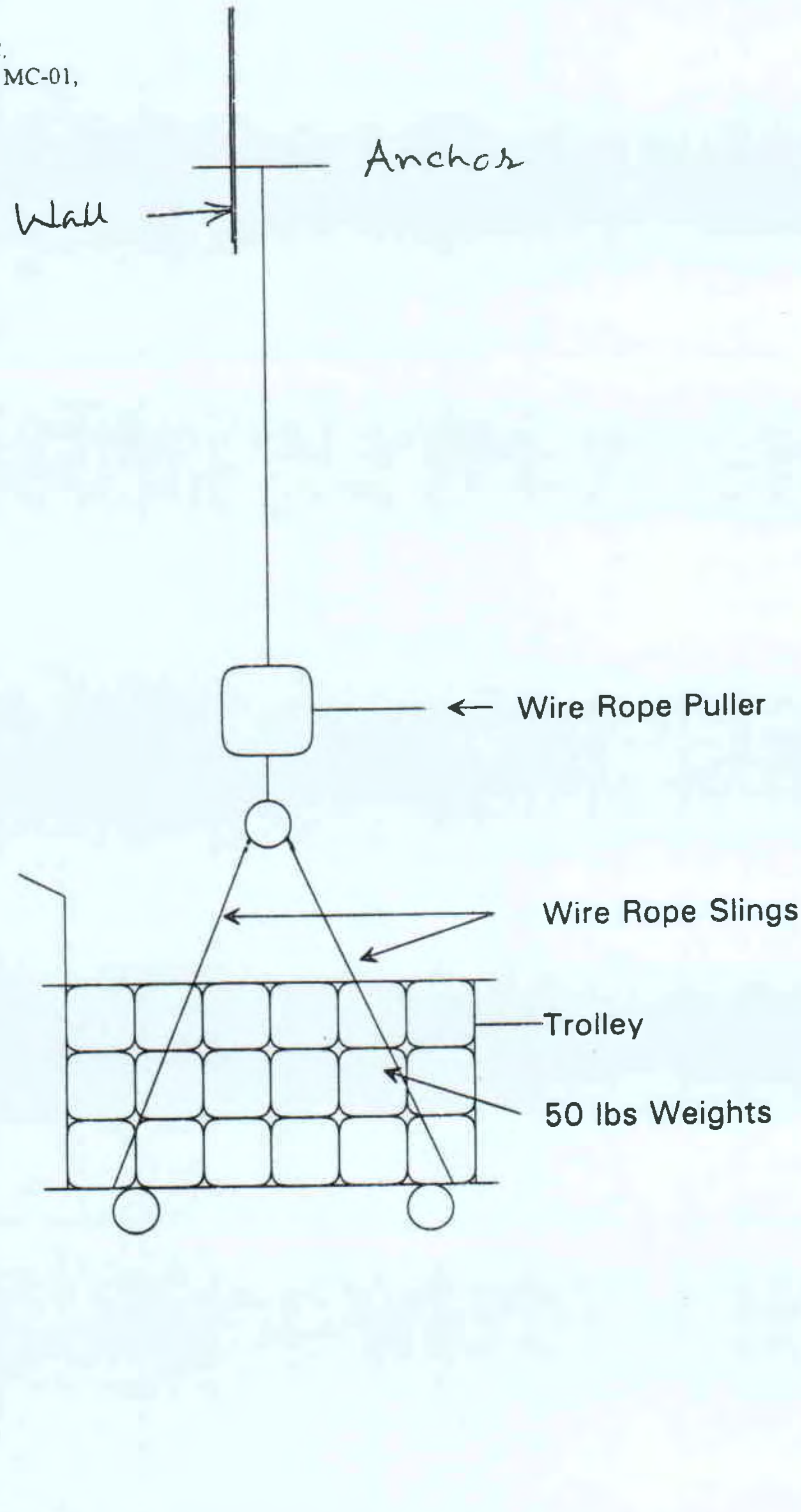


Figure 2: Setup For Shear Load Tests

HYDRAULIC EQUIPMENT SETUP

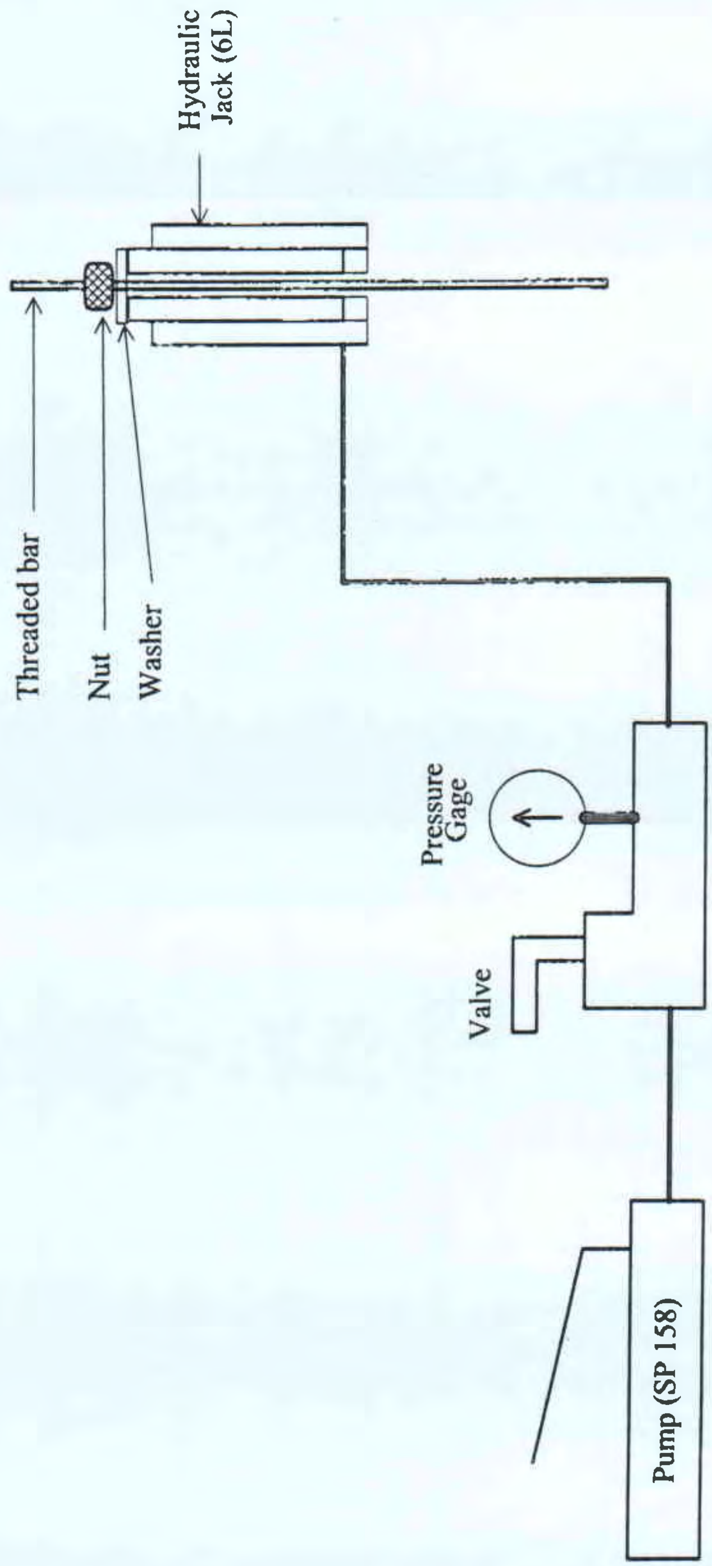


Figure 3

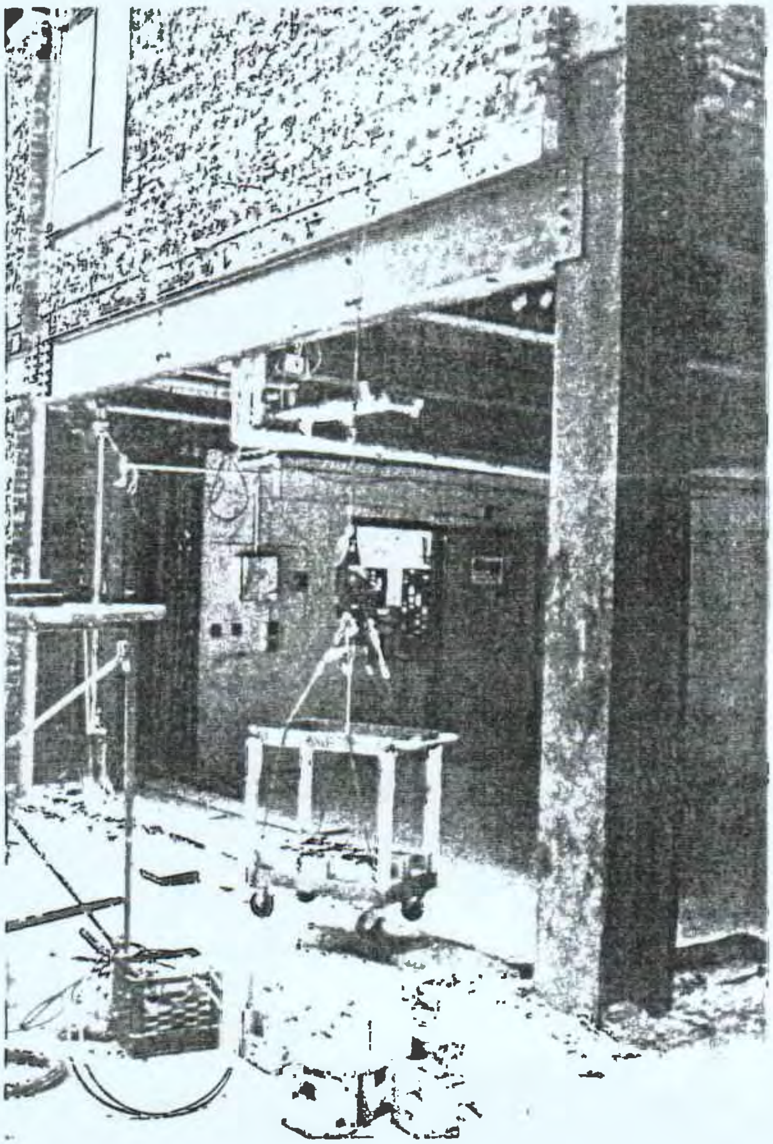


PHOTO 1

Setup for Shear Load Testing a Single Anchor

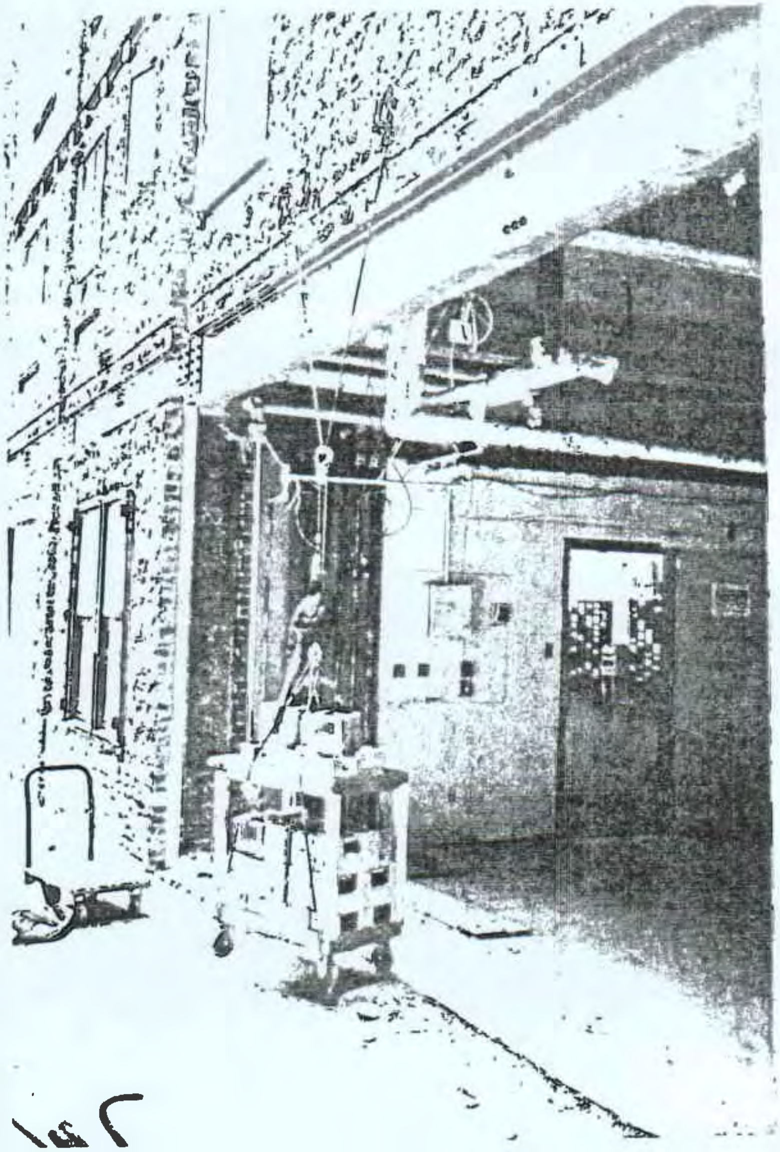


PHOTO 2

Setup for Shear Load Testing of Two Anchors Together

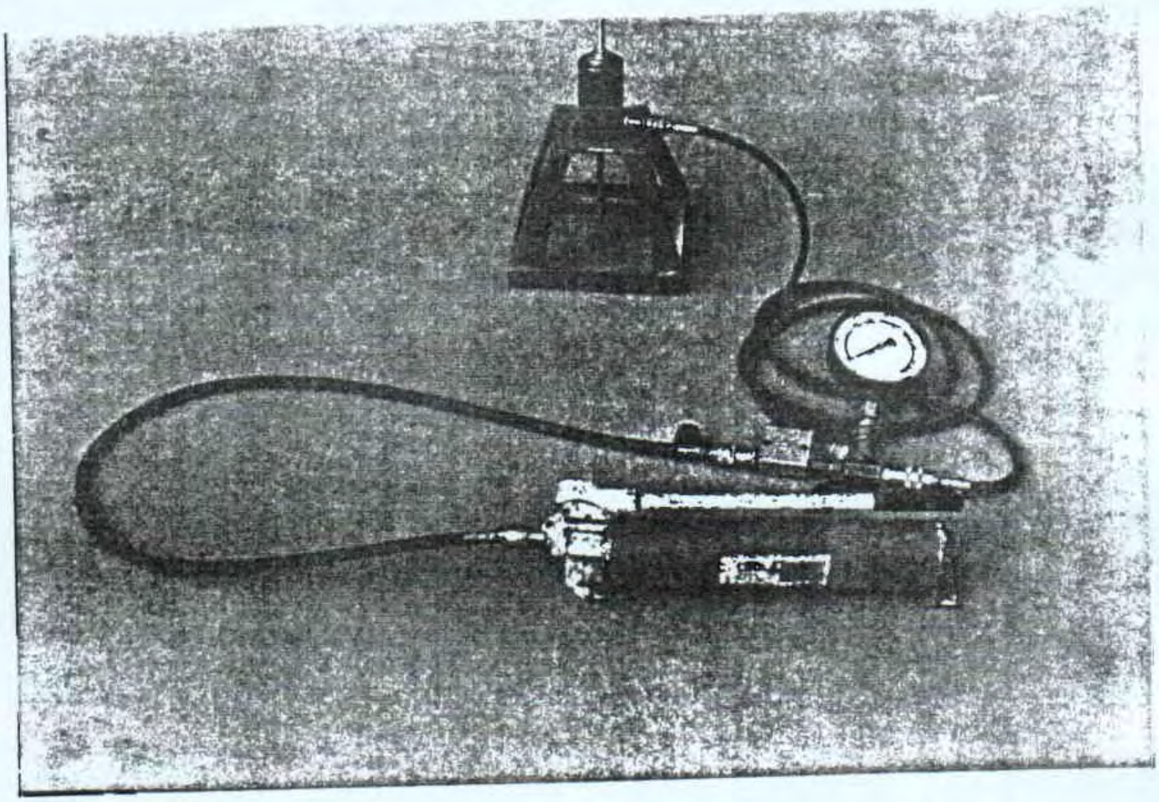


PHOTO 3

Hydraulic Equipment for Pullout Test Showing Pump, Valve, Gauge and Jack

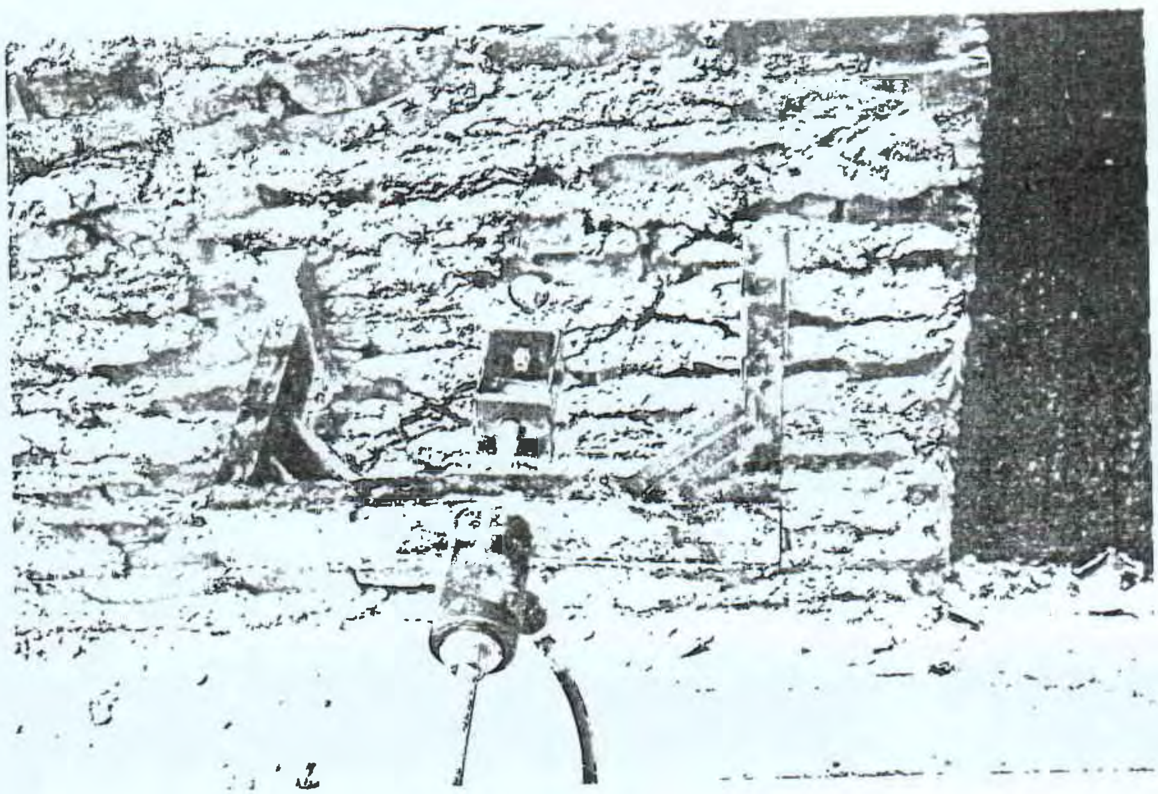
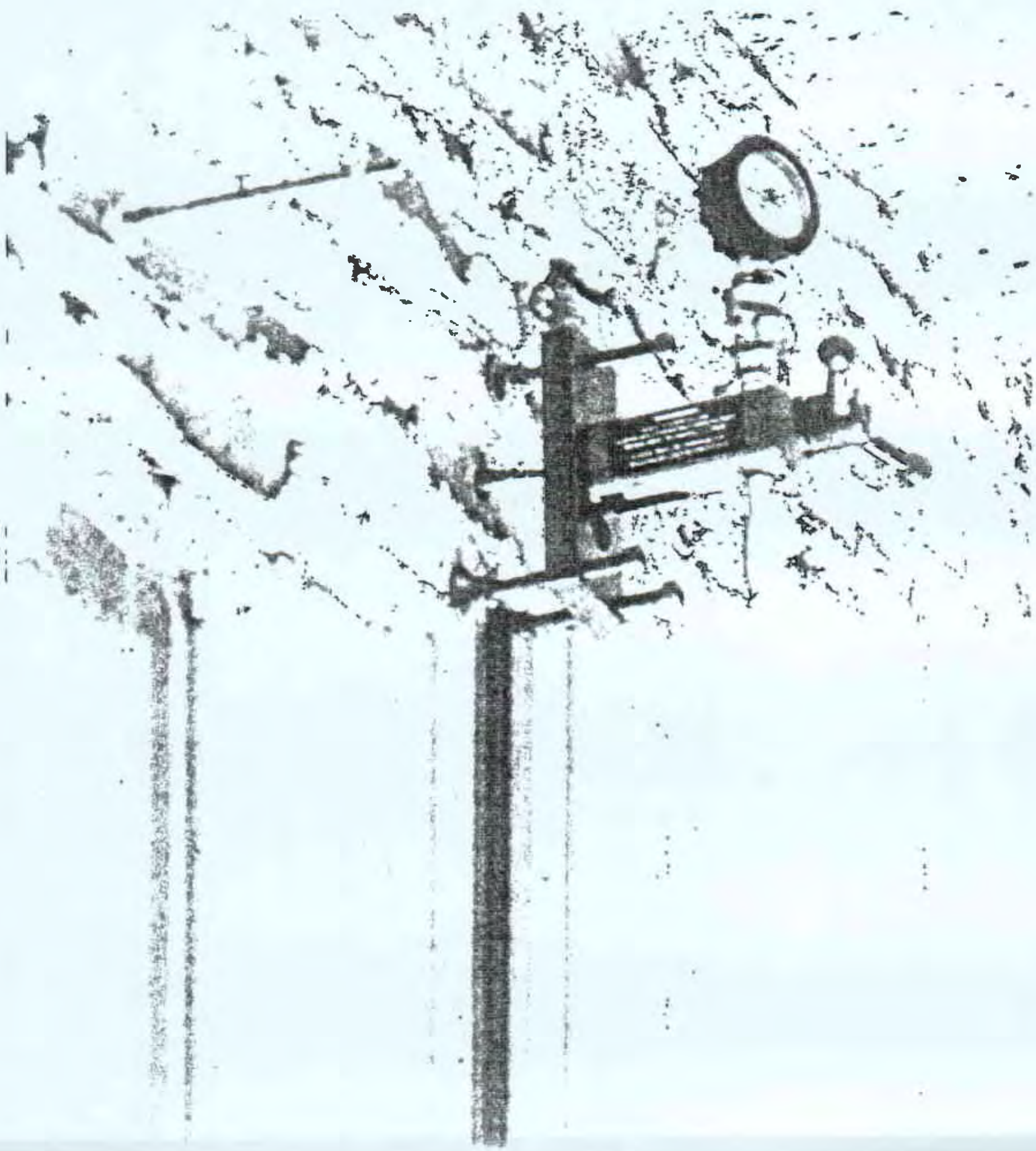


PHOTO 4

Steel Frame and Coupling for Connecting Jack to the Anchor for Pullout Test



Simpson Gumpertz & Heger Inc.
Consulting Engineers

Arlington MA / San Francisco ,A

9 August 1999

Mr. Robert Lloyd-Rees
Cintec
38 Auriga Drive, Suite 200
Nepean, Ontario K2E8A5
CANADA

Comm. 99331 - Testing of Anchors in Terra Cotta, Union County Courthouse, NJ

Dear Mr. Lloyd-Rees:

This letter report summarizes the results of our tests of the Cintec anchors in two terra-cotta blocks you submitted for the Union County Courthouse

1. Pull-out Tests

We tested four anchors in pull-out. We loaded the anchors with a pull-out testing machine with supports located outside of the expected failure cone of the anchor The following are the results of the pull-out tests.

Test 1

Nearly simultaneous cone failure and flexural failure of the block at a load of 4047 pounds. Deflection was not measured.

Test 2

Flexural failure of the block at a load of 3035 pounds and a deflection of 0.0646 inches.

Test 3

Cone failure of the block at a load of 4384 pounds and a deflection of 0.0787 inches.

Test 4

Pull-out failure of the anchor in the grout bulb in the block at a load of 3878 pounds and a deflection of 0.0882 inches.

29⁻ Broadway Arlington MA 02474-5310 ■ Telephone 781 643-200[illegible] ■ Facsimile 781-643-2009 ■ [illegible]h.com

2. Shear Test

We tested one set of anchors in shear. We set the two anchors in the bottom cell of the block into grout filled core holes in a concrete substrate. The block was separated from the concrete substrate by a 1 inch cavity. We loaded the top of the block with a hydraulic jack through a wide flange beam to spread the jack load across the entire width of the block. We restrained the top of the block against rotation with a 3/16 in. x 1 ½ in. x 1 ½ in. angle spanning across the exterior face of the block. We stopped the test at a load of 6144 pounds and deflection of 0.116 inches at the bottom of the block because the top of the block had rotated approximately ½ in.

Sincerely yours,

Michael L. Brainerd

Michael L. Brainerd, Principal
MLB69-99.ras

CINTEC ANCHOR TESTING

New York Schools Construction Authority

TESTING AT:
PS230K
1 ALBERMARLE ROAD, BROOKLYN

AND

PS238K
1633 EAST 8TH, BROOKLYN, NEW YORK

TESTING ENGINEERS:
VERSATILE CONSULTING AND TESTING SERVICES (JULY 2001)

VERSATILE CONSULTING & TESTING SERVICES, INC.

Integrity Is What This Firm Is Built On

240-02 68th Avenue
Douglaston, New York 11362 1925
Tel.: (718) 428-5025
Fax: (718) 428-1036
www.versatileconsulting.com

Contracts: PS 230 K and PS 238 K

Date: July 9, 2001

Client: Cintec North America

PROFESSIONAL ENGINEER SERVICE

Procedure: Anchors Installation

Location: Parapet Wall

I, Roman Sorokko, P.E., being duly sworn say: I am a Professional Engineer, (Lic. # 072800) assigned by Hill International, Inc. to conduct the controlled inspection for the subject contract. I have read all provisions of the Building Code of the City of New York, and I am thoroughly familiar with the plans, specifications and standards referred to herein.

As an Engineer of Record, and as directed by Hill International, Inc. and NYC DDC I will personally perform the controlled inspection of the Cintec anchors installation for this project.

I was also directed to generate an engineering calculation in order to confirm the adequacy of the anchors to the design purpose – to secure the terra cotta blocks attached to the exterior surface of the parapet wall (as per as per Item 04525 – Terra Cotta Restoration and Repair, Paragraph 2.2 Anchors).

I certify that I have carefully analyzed the proposed anchors' parameters using a conservative engineering approach (see attachment No. 1) to the best of my knowledge, and I have found that their application will be adequate to the design purpose, and it will be incompliance with the Specifications of the subject contract.

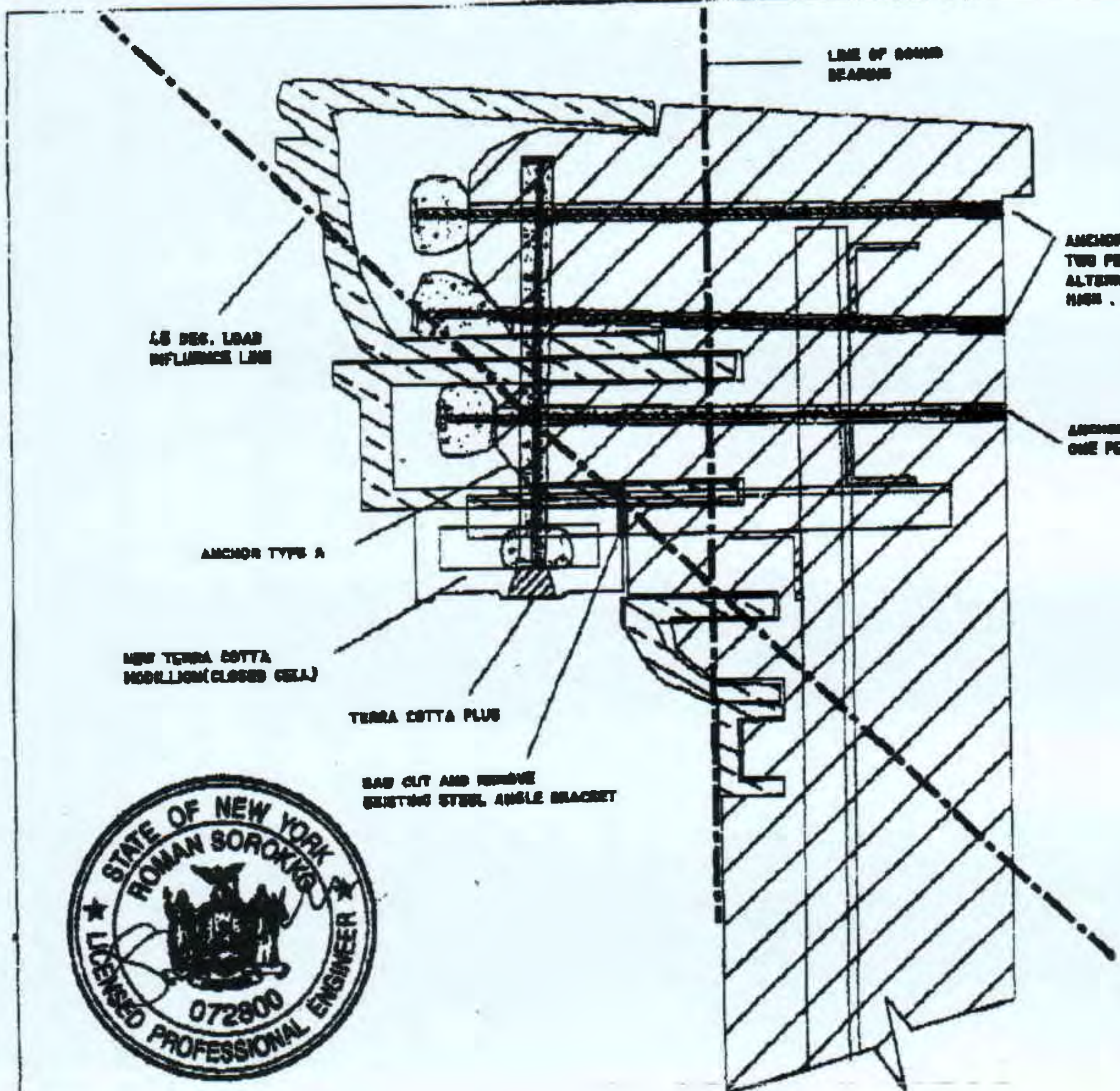
I executed the full scale pull out tests (see Attachment No. 2) for these anchors, and I have found that the achieved results are significantly exceeded the design criteria.

Therefore, I recommend these anchors to be used for the above mentioned contract.

Prep. by Roman Sorokko, P.E.

STATE OF NEW YORK ROMAN SOROKKO 072800 LICENSED PROFESSIONAL ENGINEER

FROM : VERSATILE CONSULTING & TESTING FAX NO. : 718 428 1036 Aug. 05 2001 09:58PM P4



STATE OF NEW YORK
ROMAN SOROKKO
072800
LICENSED PROFESSIONAL ENGINEER

Project Name 230 & PS 238
Location BROOKLYN, NEW YORK
Consultant: TAMS CONSULTANTS, INC.

CORNICE STABILIZATION
MODILLION REPLACEMENT ONLY

Engineered Grout Injection Anchors by: **CINTEC AMERICA INC.** Tel 613 225 3381 Fax 613 224 9042 E-mail: [illegible]	Project #: By: AZ Date: Dec 14/2000
Product Engineering By: JOKINEN ENGINEERING SERVICES Tel 905 333 1079 Fax 905 333 3659 E-mail: eric.jokinen@[illegible].com — JES C129	SK 1 DRWG #: — 1 REV. #

PAGE 1 OF 10

ANCHOR DESCRIPTIONS FOR CORNICE STABILIZATION
AND MODILLION REATTACHMENT

ANCHOR TYPE A
MODILLION REATTACHMENT

1/2" DIA SOLID THREADED SS CINTEC ANCHOR-
PLAIN ENDS- IN 1 1/4" DIA HOLE APPROX
24" LONG SOCKED FULL LENGTH. SOCK
OVERSIZED TO EXPAND INTO CELL OF NEW
T/C UNIT

ANCHOR DESIGN - TENSION

ANCHOR TYPE B
CORNICE STABILIZATION

1 1/2" x 1 1/2" x 1/8" HSS SS CINTEC
ANCHOR-PLAIN ENDS -IN 3" DIA HOLE APPROX 30" LONG
SOCKED FULL LENGTH. SOCK OVERSIZED
TO EXPAND INTO VOID AT FRONT
OF EXISTING T/C UNIT.

ALTERNATE DESIGN - EXTEND ANCHOR TO
INSIDE FACE OF PARAPET AND PROVIDE
SS NUT, WASHER AND BEARING PLARTE

ANCHOR DESIGN - COMBINED BENDING AND SHEAR

STATE OF NEW YORK
ROMAN SOROKKO
072800
LICENSED PROFESSIONAL ENGINEER

OPTION 1
CANTILEVERED DESIGN
(CONSERVATIVE)

Project Name PS 230 & PS238
Location BROOKLYN New York
Consultant: TAMS CONSULTANTS

Engineered Grout Injection Anchors by: CINTEC AMERICA INC. Tel 613 225 3381 Fax 613 224 9042 E-mail: rlr@cintec.com	Project #: By: AZ Date: JANUARY 2001	
Product Engineering By: JOKINEN ENGINEERING SERVICES Tel 905 333 1079 Fax 905 333 3659 E-mail: eric.jokinen@sympatico.com	SK 2A DRWG #:	0 REV. #

ANCHOR DESCRIPTIONS FOR CORNICE STABILIZATION
AND MODILLION REATTACHMENT

ANCHOR TYPE A
MODILLION REATTACHMENT

1/2" DIA SOLID THREADED SS CINTEC ANCHOR-
PLAIN ENDS- IN 1 1/4" DIA HOLE APPROX
24" LONG SOCKED FULL LENGTH. SOCK
OVERSIZED TO EXPAND INTO CELL OF NEW
T/C UNIT

ANCHOR DESIGN - TENSION

ANCHOR TYPE B
CORNICE STABILIZATION

3/4" DIA SOLID THREADED SS CINTEC ANCHOR-
PLAIN ENDS -IN 2" DIA HOLE APPROX 30" LONG
SOCKED FULL LENGTH. SOCK OVERSIZED
TO EXPAND INTO VOID AT FRONT
OF EXISTING T/C UNIT.

ALTERNATE DESIGN - EXTEND ANCHOR TO
INSIDE FACE OF PARAPET AND PROVIDE
SS NUT, WASHER AND BEARING PLARTE

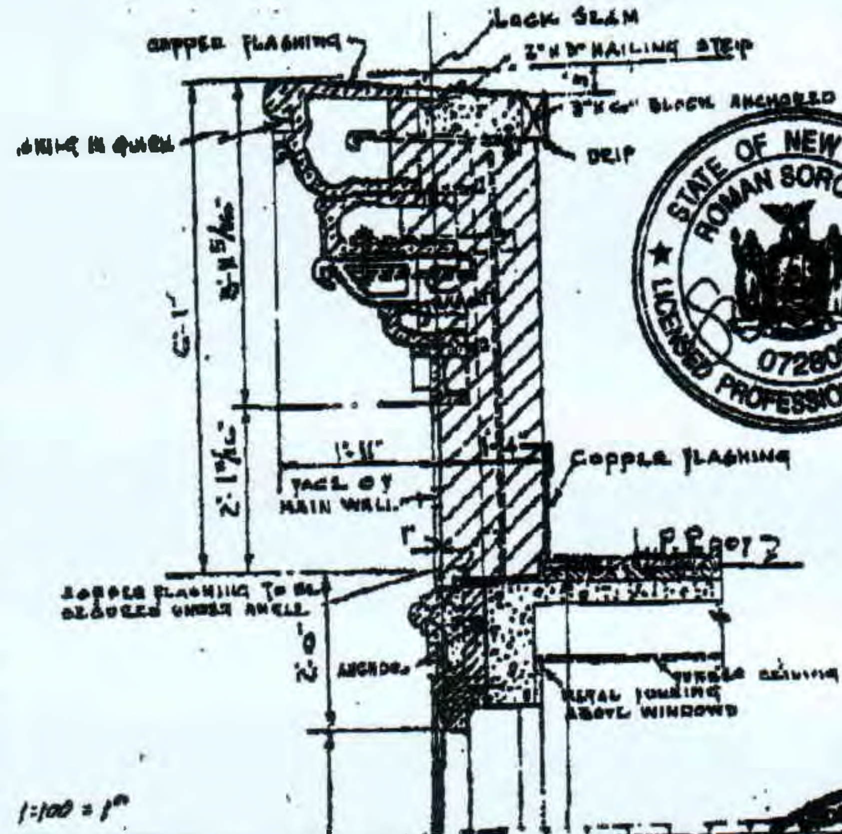
ANCHOR DESIGN - COMBINED PULL-OUT AND SHEAR

STATE OF NEW YORK
ROMAN SOROKKO
072800
LICENSED PROFESSIONAL ENGINEER

OPTION 2
CORBELLED DESIGN
(LESS CONSERVATIVE)

Project Name PS 230 & PS238
Location BROOKLYN NEW YORK
Consultant: TAMS CONSULTANTS

Engineered Grout Injection Anchors by: CINTEC AMERICA INC. Tel 613 225 3381 Fax 613 224 9042 E-mail: rlr@cintec.com	Project #:
	By: AZ
	Date: JANUARY 2001
Product Engineering By: JOKINEN ENGINEERING SERVICES Tel 905 333 1079 Fax 905 333 3650 E-mail: eric.jokinen@sympatico.com	SK 2B DRWG #: / 0 REV. #



STATE OF NEW YORK
ROMAN SOROKKO
072800
LICENSED PROFESSIONAL ENGINEER

EXISTING CONDITION

FEB 2 0 2001
ERIC F. JOKINEN

Project Name PS 230 & PS 238
Location BROOKLYN, NEW YORK
Consultant: TAMS CONSULTANTS, INC.

DESIGN CALCULATIONS
CORNICE / PARAPET

Engineered Grout Injection Anchors by:

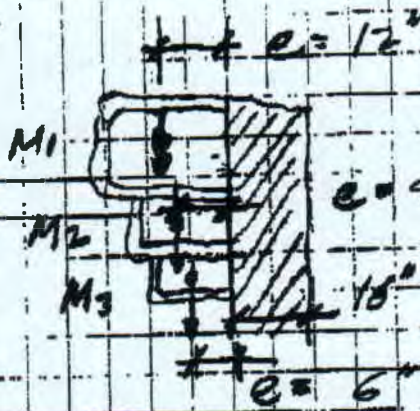
CINTEC AMERICA INC.
613 225 3381 Fax 613 224 9042 E-mail: ctr@cintec.com

Product Engineering By: JOKINEN ENGINEERING SERVICES
Tel 905 333 1079 Fax 905 333 3650 E-mail: eric.jokinen@sympatico.com

JES G129

Project #:
By: AZ
Date: Dec 14/2000

DE 1	0
DRWG #:	REV. #



STATE OF NEW YORK ROMAN SOROKKO LICENSED PROFESSIONAL ENGINEER 072800

(L)

M1: 2' x 1.5' x 2.5 @ 100PCF = 750#

M2: 1' x 1.5' x 3.0 @ 100PCF = 450#

M3: 1' x 0.67' x 2.0 @ 50PCF = 100#

MATL: AISI 304 SS } STEEL

F_y = 29,000 PSI

f'_c = 5000 PSI GROUT

f_b = 300 PSI BRICK SUBSTRATE

ERIC P. JOKINEN

Project Name: PS 230 & PS 238

Location: BROOKLYN, NEW YORK

Consultant: TAMS CONSULTANTS, INC.

DESIGN CALCULATIONS

CORNICE & PARAPET

Engineered Grout Injection Anchors by: CINTEC AMERICA INC. 613 225 3381 Fax 613 224 9042 Email: rin@cintec.com		Project #:
		By: AZ
		Date: Dec 14/2000
Product Engineering By: JOKINEN ENGINEERING SERVICES Tel 905 333 1079 Fax 905 333 3659 E-mail: eric.jokinen@sympatico.ca	JES C129	DE 2 / 0
		DRWG #: / REV. #

OPTION 1: CANTILEVER DESIGN (CONSERVATIVE)

BLOCK M1: 750#/2 ANCHORS

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072800
LICENSED PROFESSIONAL ENGINEER

2 Anchors / block

1½ x 1½ x ⅛ Anchor body $A_v = 0.531\ in^2$

#55 $S_b = 0.266\ in^3$

Shear

$V = \frac{750}{2} = 375\#$

$f_v = \frac{V \times 2.0}{0.531} = 1412\ PSI < 0.4 F_y$

$M = \frac{750}{2} \times 12" = 4500\ in\text{-}lb$

$f_b = M/S = \frac{4500}{0.266} = 16917 < 0.66 F_y$

LICENSED PROFESSIONAL ENGINEER
FEB 20 2001
ERIC P. JOKINEN
PROVINCE OF ONTARIO

Project Name PS 230 & PS 238
Location BROOKLYN, NEW YORK
Consultant TAMS CONSULTANTS, INC.

DESIGN CALCULATIONS

Engineered Grout Injection Anchors by: CINTEC AMERICA INC. 613 225 5381 Fax 613 224 3042 E-mail: rir@cintec.com		Project #:
		By: AZ
		Date: Dec 14/2000
Product Engineering By: JOKINEN ENGINEERING SERVICES Tel 905 333 1079 Fax 905 333 2659 E-mail: eric.jokinen@sympatico.com	JES C129	DE 3 DRWG #: / 0 REV. #

OPTION 1: CANTILEVER (CONSERVATIVE)

BLOCK M1 (CONT'D)

CHECK BRG ON BRICK

ASSUME 3" φ DIA HOLE

BRG ON ½ CIRCUMF.

EMBEDMENT = 1'-2" = 16" - 2" = 14"

$S_b = \frac{3 \times 3.14}{2} \times \frac{14^2}{6} = 154 \text{ in}^3$

$A_b = \frac{3 \times 3.14}{2} \times 14 = 66 \text{ in}^2$

Moment about centroid of brick brg area

$M = \frac{e}{12} + \frac{14}{2} = 19" \times 375^\# = 7125 \text{ in-lb}$

$f_{bend} = \frac{7125}{154} = 46 \text{ psi}$

$f_{brg} = \frac{375}{66} = 6 \text{ psi}$

$\Sigma = 52 \text{ psi} < 300 \text{ PSI ALLOWABLE}$

STATE OF NEW YORK ROMAN SOROKKO LICENSED PROFESSIONAL ENGINEER 072800

Project Name: PS 230 & PS 238

Location BROOKLYN, NEW YORK

Consultant: TAMS CONSULTANTS, INC.

DESIGN CALCULATIONS

ERIC P. JOKINEN

Engineered Grout Injection Anchors by:

CINTEC AMERICA INC.

613 225 3581 Fax 613 224 9042 E-mail: rlr@cintec.com

Product Engineering By: JOKINEN ENGINEERING SERVICES

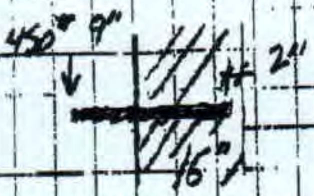
Tel 905 333 1079 Fax 905 333 3659 E-mail: eric.jokinen@sympatico.com

JES C129

Project #:	
By: AZ	
Date: Dec 14/2000	
DE 4	0
DRWG #:	REV. #

OPTION 1 - CANTILEVER

BLOCK M2:



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TRY 1 ANCHOR/BLOCK

$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{8}$ in 3" HOLE - EMBED 14"

$M_C = 450 \times 9 = 4050 < M_c$ (BLOCK 1) ∴ OK

$M_{BRG} = 450 \times (9+7) = 7200$ in-lb $\approx M_{brg}$ (BLOCK 1) ∴ OK

USE SAME DESIGN AS BLOCK 1 EXCEPT 1 ANCHOR/BLOCK

BLOCK M3 IN TENSION

$P = 100$#/blk ½"φ THRD ROD $A_{net} = 0.11$ in²

$P_{allow} = 0.11 \times 29000 \times 0.6 = 1900$# OK

Pull out 1¾"φ HOLE x 12" EMBED @ 65 PSI BOND

$P_u = 3060$# OK

END OPTION (1)

Project Name: PS 230 & PS 238

Location: BROOKLYN, NEW YORK

Consultant: TAMS CONSULTANTS, INC.

DESIGN CALCULATIONS

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ERIC JOKINEN
PROVINCE OF ONTARIO

Engineered Grout Injection Anchors by:

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613 225 3381 Fax 613 224 9042 E-mail: rle@cintec.com

Product Engineering By: JOKINEN ENGINEERING SERVICES

Tel 905 333 1079 FAX 905 333 3657 E-mail: eric.jokinen@sympatico.com

IES C129

Project #:

By: AZ

Date: Dec 14/2000

DE 5	0
DRWG #:	REV. #

OPTION 2: CORBELLED

$P = M1 + M2 + M3$
$= 750 + 450 + 100\#$
$= 1300\#$

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$M_c = 12 \times 1300 = 15600$ in-lb

$P = \frac{M}{d} = \frac{15600}{16} = 975\#$ PULLOUT ON 1 ANCHOR

TRY 3/4"φ in 1 1/2"φ HOLE - 14" EMBED

STEEL: $P_a = 0.25 \times 29000 \times 0.6 = 4350\# < 975\#$

BOND: $P_a = 1.5\pi \times 12" \times 65$ PSI $= 3673\# < 975\#$

∴ OK

BLOCK M3 HANGER (AS OPTION 1 DESIGN)

Project Name PS 230 & PS 238
Location BROOKLYN, NEW YORK
Consultant TAMS CONSULTANTS, INC.

DESIGN, CALCULATIONS

LICENSED PROFESSIONAL ENGINEER · FEB 20 2001 · ERIC JOKINEN · PROVINCE OF ONTARIO

Engineered Grout Injection Anchors by:
CINTEC AMERICA INC.
613 225 3381 Fax 613 224 9042 E-mail: [illegible]

Product Engineering By: JOKINEN ENGINEERING SERVICES
Tel 905 333 1079 Fax 905 333 3659 E-mail: eric.jokinen@sympatico.ca

JES C129

Project
By AZ
Date: Dec 14/2000
DE 6 | 0
DRWG #: | REV. #

OPTION 2: CORBELLED

CHECK SHEAR IN ANCHORS @ PARAPET ~~EXT.~~ FACE

3 - 3/4"ϕ THRD ROD DOWELS

$$V_{allow} = \frac{0.25 \times 29000 \times 0.4}{1.33} = 2180^{\#}/\text{anchor} > 1300^{\#} \text{ TOTAL REQD}$$

$\therefore$ OK

END OPTION 2

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FEB 20 2001
ERIC P. JOKINEN
PROVINCE OF ONTARIO

Project Name PS 230 & PS 238

Location BROOKLYN, NEW YORK

Consultant: TAMS CONSULTANTS, INC.

DESIGN CALCULATIONS

Engineered Grout Injection Anchors by:	Project #:
CINTEC AMERICA INC.	By: AZ
Tel 613 224 9981 Fax 613 224 9042 E-mail: cintec@cintec.com	Date: Dec 14/2000
Product Engineering By: JOKINEN ENGINEERING SERVICES — JES C129	DE 7 (DRWG #) — 0 (REV. #)
Tel 905 333 1079 Fax 905 335 3659 E-mail: eric.jokinen@sympatico.ca	

FROM : VERSATILE CONSULTING & TESTING FAX NO. : 718 428 1036 Aug. 05 2001 10:06PM P15

Report A1

FILE

Versatile Consulting and Testing Services, Inc.

ANCHORS TESTING PROGRAM

Project: PS 230K

Prepared for Hill International, Inc.

by Roman Sorokko, P.E.
/Lic. No. 072800/

STATE OF NEW YORK
ROMAN SOROKKO
072800
LICENSED PROFESSIONAL ENGINEER

JUNE 2001

THE CEMENTITIOUS INJECTED GROUT ANCHORS TESTING PROGRAM.

I, Roman Sorokko, P.E., being duly sworn say: I am a Professional Engineer, (Lic. # 072800) assigned by Hill International, Inc. to conduct the anchor testing for the Project E3000.

I have read all provisions of the Building Code of the City of New York, and the Project Specifications, and I am thoroughly familiar with the plans, and standards referred to herein, and I am thoroughly familiar with all responsibilities for the inspection of the subject item.

1. Introduction

As per NYC DDC request, and as directed by Hill International, Inc. we performed the pull out test of the steel anchors fabricated by Cintec America Inc. The purpose of the test is to verify the anchors' design parameters as per Item 04525 – Terra Cotta Restoration and Repair, Paragraph 2.2 Anchors.

2. Equipment

20 tons hydraulic jack with center hole cylinder, gage 1000 psi, hydraulic hand pump, loading valve.

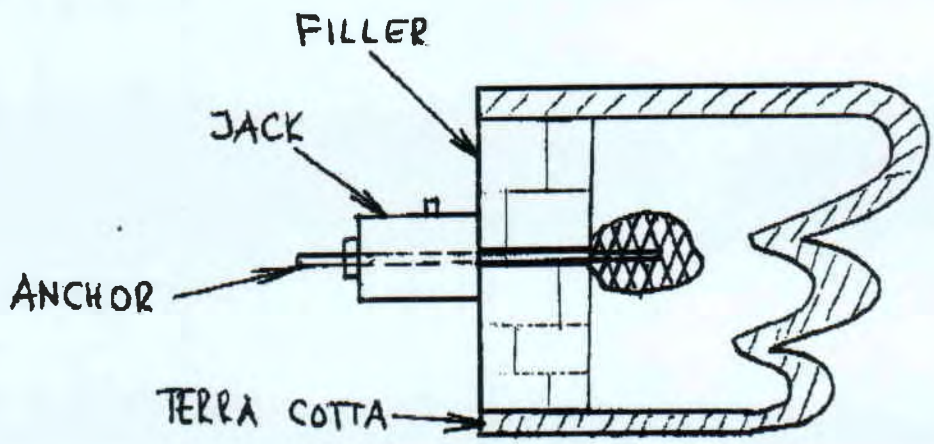
3. Procedure

One June 18, 2001 the anchors Type A and Type B were installed by the representatives of Cintec America Inc. This installation was also a part of the Contractor's training program. Two Type A anchors (1/2" in diameter) designated for the testing were placed into the pies of terra cotta which was previously removed from the building. One designated for testing Type B anchor 1.5" x 1.5" x 1/8" HSS shape was installed into the parapet wall.

4. Test Results

TEST No. 1

Type A anchors were tested under two different schemes. During the first setting the jack was placed directly on top of the terra cotta's masonry filler (see Photo No. 1 and Sketch below). Therefore, the forces developed by the hydraulic jack were transformed into the anchor's expanded part through the brick masonry filler. Hence, the terra cotta, as well as the joints between the terra cotta and the filler were not stressed.

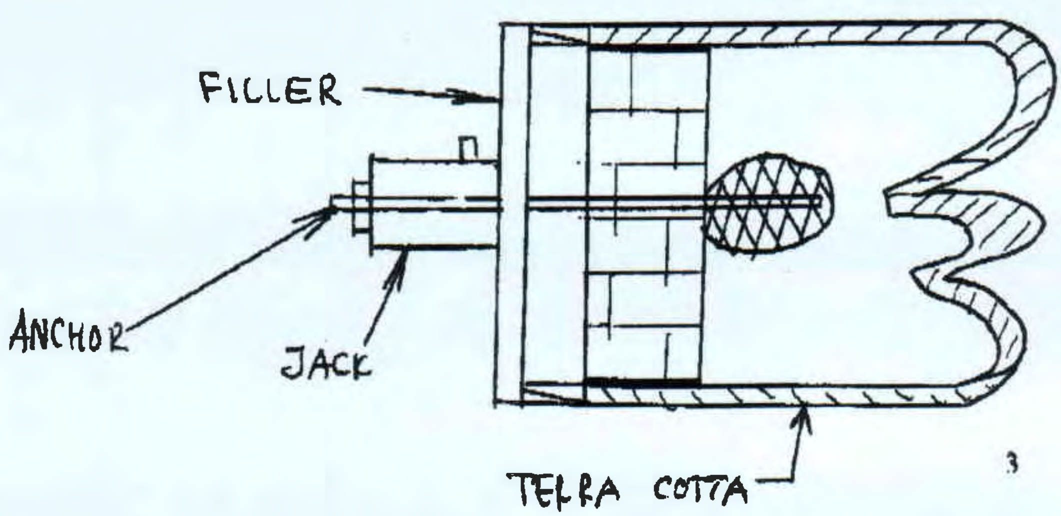


Under this format the Type A anchor was loaded up to 4,690 lbs or 2.3 tons (equivalent of 1,000 psi reading). The load was gradually applied in steps of 100 psi increment.

The tested anchor sustained the maximum load of 4,690 lbs or 2.3 tons during 15 minutes.

TEST No. 2

During the second test the same (previously tested) Type A anchor was loaded in such way that the terra cotta was under the compression stress from the hydraulic jack. Therefore, the pull out forces were developed along the joint between the terra cotta and the masonry filler (see Photos No. 2 and 3 and Sketch below).



The anchor was gradually loaded up to 2,680 or 1.3 tons which equivalent to 550 psi reading. At this point the joint between the terra cotta and the masonry filler failed (see Photos No 4, 5, 6 and 7).

TEST No. 3.

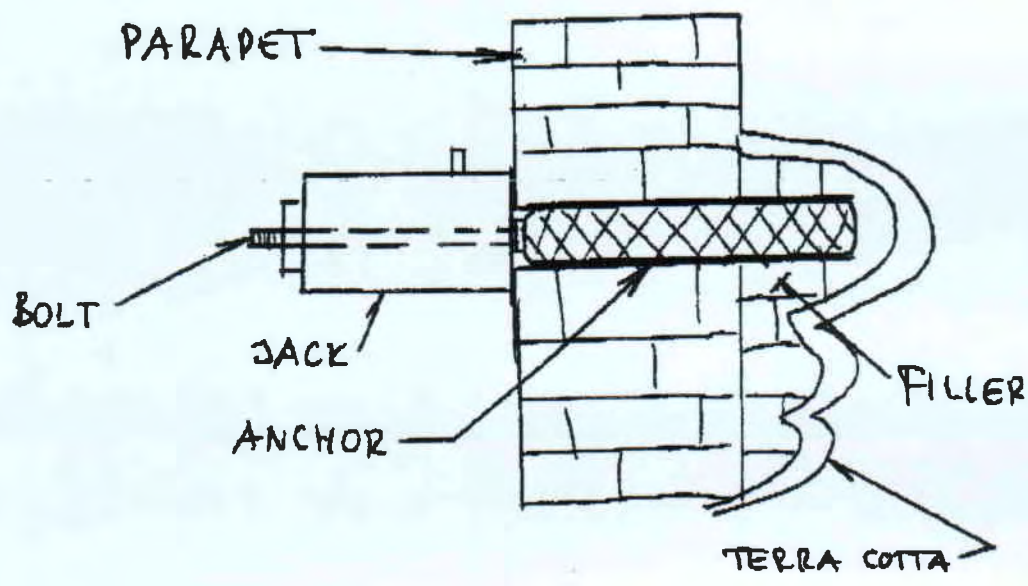
The second Type A anchor was loaded similar to the TEST No. 2 – pull out forces were applied to the joint between the terra cotta and the masonry filler.

The joint failed under the load of 3,987 lbs or 2.0 tons (equivalent to 850 psi reading).

TEST No. 4

The anchor Type B (installed into the brick masonry parapet wall and partly inserted into the terra cotta) was tested under the similar setting to the TEST No. 2 and 3 as shown on Sketch below.

For the testing purpose the steel bolt, ¾" in diameter, was welded to the anchor's cover plate.



The anchor was gradually loaded up to 2,814 lbs or 1.4 tons (equivalent to 600 psi reading). The anchor has sustained this load for 15 minutes.

Testing of Cintec Anchors for Masonry Arch Bridges, Tunnels Signal Gantry Support Anchors and Ground Anchors

Introduction to the Transport Research Laboratory, UK (TRL)

Load Test to failure on a ring separated arch (TRL) (January 1998)

Further test to failure on a strengthened brick arch using multi-bar anchors (TRL) (June, 2001)

Strengthening and Load Testing of Baber Bridge Hounslow, UK TRL (February, 2000)

Testing Kennet Bridge for British Rail. Testing by Ove Arup and Partners (February 1988)

Testing of Ground Anchors at Bridge 325 Abington. Testing by S Woodhouse, B.Eng. (Hons) C.Eng M.I.Struct. E (April 1993)

Testing of Viaduct No. 4 Birdmill Viaduct. Testing by British Rail Bridge Engineer.

Testing of Westminster Tunnel Liverpool. Testing by Peter G Griffin B.Sc. C.Eng M.I.Struct E. (June, 1994)

Testing Proof Load Tests on Cintec anchor for Singnal Gantry Supports. Testing by Lang Technology Group (June, 1994)

The Transport Research Laboratory is the largest and most comprehensive centre for the study of road transport in the United Kingdom. For more than 60 years it has provided information that has helped frame transport policy, set standards and save lives.

TRL provides research-based technical help which enables its Government Customers to set standards for highway and vehicle design, formulate policies on road safety, transport and the environment, and encourage good traffic engineering practice.

As a national research laboratory TRL has developed close working links with many other international transport centres.

It also sells its services to other customers in the UK and overseas, providing fundamental and applied research, working as a contractor, consultant or providing facilities and staff. TRL's customers include local and regional authorities, major civil engineering contractors, transport consultants, industry, foreign governments and international aid agencies.

TRL employs around 300 technical specialists - among them mathematicians, physicists, psychologists, engineers, geologists, computer experts, statisticians - most of whom are based at Crowthorne, Berkshire. Facilities include a state of the art driving simulator, a new indoor impact test facility, a 3.8km test track, a separate self-contained road network, a structures hall, an indoor facility that can dynamically test roads and advanced computer programs which are used to develop sophisticated traffic control systems.

TRL also has a facility in Scotland, based in Edinburgh, that looks after the special needs of road transport in Scotland.

The laboratory's primary objective is to carry out commissioned research, investigations, studies and tests to the highest levels of quality, reliability and impartiality. TRL carries out its work in such a way as to ensure that customers receive results that not only meet the project specification or requirement but are also geared to rapid and effective implementation. In doing this, TRL recognises the need of the customer to be able to generate maximum value from the investment it has placed with the laboratory.

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TRL prides itself on its record for delivering projects that meet customers' quality, delivery and cost targets. The laboratory has, however, instigated a programme of continuous improvement and continually reviews customers satisfaction to ensure that its performance stays in line with the increasing expectations of its customers.

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Load test to failure on a ring-separated arch repaired using Cintec Anchor System

by S K Sumon

Unpublished Project Report
PR/CE/61/98
P6932

TRANSPORT RESEARCH LABORATORY

PROJECT REPORT
PR/CE/61/98

LOAD TEST TO FAILURE ON A RING-SEPARATED ARCH REPAIRED USING CINTEC ANCHOR SYSTEM

by S K Sumon

Prepared for:	**Project Record:**	**P6932**
	Customer:	**Mr Peter James** **CINTEC, Cavity Lock Systems**

Approvals	
Project Manager	
Quality Reviewed	

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LOAD TEST TO FAILURE ON A RING-SEPARATED ARCH REPAIRED USING CINTEC ANCHOR SYSTEM

ABSTRACT

This report describes a load test to failure on a three-ring-brick arch bridge. The arch was constructed with a layer of wet sand between the three rings to simulate ring-separation (delamination). It was then strengthened using the Cintec Anchor System. The test was carried out to find out the increase in load bearing capacity of the strengthened arch bridge.

Elastic tests were carried out by applying a series of 50 kN point loads and the maximum deflection was 2.2 mm, which was recorded on the edge of arch at the crown. The maximum load to failure, when the arch was loaded at quarter-span, was 410 kN. The arch failed in a gradually but ductile manner with crushing of the bottom brick at the crown which lead to the formation of the first hinge. During the loading to collapse the bottom ring fell away but the rest of the arch remained held together by the anchors. It was eventually collapsed after a further displacement of 200 mm at the load-line. The arch failed by crushing of masonry and a four hinge mechanism.

The results are compared with those obtained from a previous test on an arch that also had built in ring-separation but was not strengthened (Sumon, 1997).

1. INTRODUCTION

Many strengthening methods exist but they have never been tested and compared quantitatively. A test rig has been built in the Structures Laboratory at the Transport Research Laboratory to investigate this. The rig enables the construction of a 5 m span, 2 m wide, three-ring-brick arch which can then be tested to failure.

The Cintec arch was constructed without spandrel walls and no road surface, which had been left out to reduce the number of parameters being studied. Backfill was then placed and compacted. The fill was retained by a steel former which has been designed not to restrain movement of the arch ring. It was then strengthened (see section 3) and tested under controlled environmental conditions.

The arch was constructed on 6-8 November 1997, strengthened on 23 December 1997 and tested on 27 January 1998.

2. ARCH CONSTRUCTION

The arch had a layer of sand between the rings rather than mortar to simulate ring-separation (delamination). Ring-separation is one of the common defects found in many old arch bridges. A brief account of the construction of the arch is given below.

The procedure was to build one ring at a time working up from the springings to meet at the crown. Handmade bricks were used, which best simulated those used in pre 1900 arch bridges. They were weak bricks by modern standards and in some cases fissured and distorted. In accordance with current practice, cement, lime and sand were mixed to give a traditional lime mortar. The mortar was mixed using a cement mixer with the minimum amount of water added to give the required workability. The nominal width of mortar used in the joints was 10 mm though some tolerance was allowed near the crown to ensure that the arch was completed without the need to cut bricks.

3. ARCH STRENGTHENED WITH CINTEC ANCHOR SYSTEM

Eight Cintec anchors were used to strengthen the delaminated barrel. Drawing no. B0822A/001 gives the general layout and dimensions of the arch bridge. The positions and angles of installation of the anchors are shown in drawing no. B0822A/002. Fill was excavated approximately 1.2 m from the west-end and 1.4 m from the east-end of the arch to expose the extrados. Then using a long drilling rig, mounted to a purpose built scaffolding fixed to the arch rig, eight holes were drilled into the barrel. Two anchors (A) were installed through the barrel and into the concrete abutment on the west end. A further four anchors (B) and two (C) were installed which were contained in the barrel; on the west-end and east-end of the arch respectively.

4. MATERIALS USED

4.1 MORTAR CUBES

Nine mortar cubes were obtained in batches of three from the various mixes during construction. The mortar was made and tested according to BS 5628:1978 (BSI 1978). The cement:lime:sand ratio was 1:3:12 and the mean cube strengths obtained were 0.42 N/mm^2 and 0.71 N/mm^2 at 12 and 28 days, respectively.

4.2 BRICKS

Swanage Heathered Handmade type bricks were used. Tests were executed to BS 3921 British Standard Specification for Clay Bricks (BSI 1985); Appendix D. A mean compressive strength of 10 N/mm^2 was obtained from the manufacture's literature.

4.3 FILL

The fill used for the back fill was a Type 2 roadbase material.

5. INSTRUMENTATION

5.1 STRAIN GAUGES

Nine Gage Technique vibrating wire (VW) strain gauges (gauge length = 140 mm) were installed on the arch extrados. Small steel boxes were placed over the gauges to protect them from damage, and the fill material was placed and compacted. A further nine gauges were attached to the intrados on removal of the centring. These were to measure the longitudinal strain profile along the centre-line of the arch. The accuracy of the strain measurement was to within 1 micro-strain.

5.2 DISPLACEMENT GAUGES

Nine linear variable differential transformers (LVDTs); displacement transducers were attached to the arch soffit to measure vertical movement at the quarter-span, crown, and three-quarter-span. In each case one gauge was attached to the transverse centre-line and one at the edge. Two additional transducers were attached horizontally to the monitor abutment movement. Solartron "B" series transducers were used. They have a stroke of ± 25 mm with a non-linearity of <0.25 %.

5.3 DATA RECORDING EQUIPMENT

The vibrating wire strain gauge data was recorded using a Strainmanager ST1 system. The data from the LVDTs was recorded using an Orion 3350 data logger system. For consistency the same cabling system was used at all stages of testing.

6. LOAD TESTS AND RESULTS

6.1 ELASTIC TESTS (point load tests)

Procedure

To determine the elastic behaviour of the arch a series of single 50 kN point loads were applied to the fill. The load was applied on one quarter of the arch through a loading foot which simulated the area of a heavy goods vehicle tyre. The loads were applied from east to west using a 200 kN hydraulic jack, in this case, from a-f, g-l and m-r, shown in Figure 1.

Two tests were performed one before and one after the strengthening. In the first test only local strain data was obtained, whereas in the second test both strain and displacements were recorded. Generally all gauges returned to their initial position after the removal of the load indicating structure tested in the elastic range. The strain data was not plotted. The influence lines plotted from the displacement data are shown in Figures 2a, 2b and 2c.

The maximum deflection of 2.20 mm was recorded on the edge of arch at the crown. The results from both arches are summarised in Table 1 below:

ELASTIC TESTS	Maximum displacement, at the crown, when applying 50 kN point loads along longitudinal axis (+ ve ↓)		
Load positions --->	at centre-line (c/l)	at 0.42 m from c/l	at the edge of arch
TRL Arch	1.61 mm	1.71 mm	3.37 mm
Cintec Arch	1.05 mm	1.40 mm	2.20 mm

Table 1: Values of maximum displacement

6.2 LOAD TEST TO FAILURE

The objective of this test was to find out the load bearing capacity of the arch and the effectiveness of the applied strengthening method. The arch was tested until either hinges had formed or severe damage had occurred. The load was applied at the quarter-span on the west-end (see Figure 1) using a 3000 kN hydraulic jack. The structure was loaded in approximately 10 kN increments. The resulting strains and displacements were recorded at each increment. The formation and propagation of the cracks were recorded. The arch was loaded until it could not sustain any further increase in load.

Again the strain data was not plotted. Plots of the displacement under the load-line, opposite quarter span and crown are given in Figures 3a and 3b. A comparison with results obtained from the unstrengthened arch tested previously are given in Table 2 below.

TESTS TO FAILURE	Load to failure (kN)	Factor	Max. disp. @ max. load applied (mm)	Max. disp. after load removal (mm)
TRL Arch	200.00	1.00	27.40	23.40
Cintec Arch	410.00	2.05	16.50	11.40

Table 2: Summary of two load tests to failure

Discussions

The first cracks were recorded around the crown circumferentially between the top/middle, and middle/bottom rings on the north face at a load of 100 kN. Initially very little damage was observed under the load-line except the rings-separating mainly due to existing ring-separation. As the arch was loaded further crushing was observed on the bottom ring at the crown, which lead to the first hinge (at approximately 280 kN). The strengthening prevented the first hinge from forming under the load-line and delayed the formation of further hinges.

Damage continued to occur as the load was increased and further hinges formed at the load-line (at ≈320 kN), around the opposite quarter span (at ≈ 350 kN) and nearer to the springings on the load-line side (at ≈ 350 kN). Thus a four hinge mechanism had formed. The arch was loaded until significant creep and plastic deformation had occurred, and it could not sustain further load. The maximum load applied to the arch was 410 kN. The load was then removed and the arch sprung back but there was still considerably deformation (Figure 4). This suggests that the strengthening method had some elastic properties. The maximum displacement was 16.50 mm which dropped to 11.40 mm when the load was removed.

6.3 TEST TO COLLAPSE

All surface mounted instrumentation was removed and the knife edge was substituted with 'railway sleepers'.

Discussions

The arch was then loaded to collapse. It was difficult to maintain the applied load once the maximum load of 365 kN had been reached. Here the load could not be increased further as the structure had been considerably damaged during the test to failure. The arch was being pushed down at the load-line and up at the crown, and breaking up internally as indicated by the rapid dropping of load. At this point the loading system was switched over to displacement control. Following this as the displacement was increased the bottom ring started to crush at the crown. There was also high level of creep and plastic deformation taking place. This lead to the bottom ring falling away from the structure (the load at this point was only 11.50 kN), but the rest of the arch remained held together by anchors. Figure 5 and 6 shows the arch just before and just after the bottom ring fell away. This shows the exposed anchors, under the load-line, although considerably bent they are still holding the structure together.

The arch was eventually collapsed by a further displacement of 200 mm at the load-line.

7. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions may be drawn from the test carried out:

- The load bearing capacity of the arch was increased by a factor of 2.05.
- The first crack and hinge did not occur under the load-line.
- The installed anchors delayed the formation of hinges.
- The anchors added considerable strength to the bridge.
- The arch failed in a gradual but a ductile manner.
- On unloading the structure recovered indicating some elastic behaviour.

- The bond between the anchor and masonry was found to be good.
- The strengthening was relatively easy to install.

Although the test has been successful and a considerable strength increase has been obtained, it may be worth considering testing Cintec Anchor System in conjunction with the following:

- A delaminated arch that has been stitched.
- An arch sofffit that has been partially or completely repaired.
- An arch with no visible defect (for example arch with an imposed load restriction.)

8. ACKNOWLEDGEMENTS

The author would like to express his thank to Nigel Ricketts and the Civil Engineering Resource Centre (CERC) *test-team* for their involvement and contribution to the programme.

9. REFERENCES

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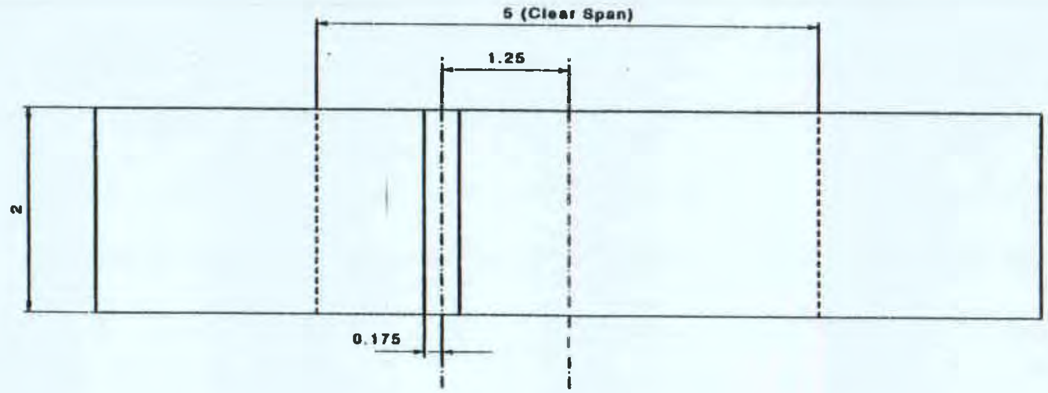

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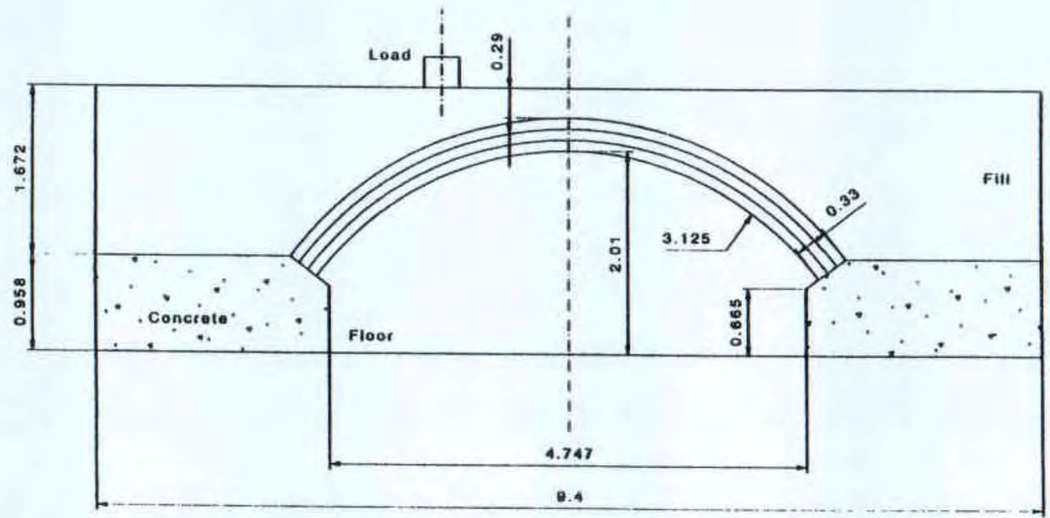

FRONT ELEVATION

General Arrangement of TRL Arch Test

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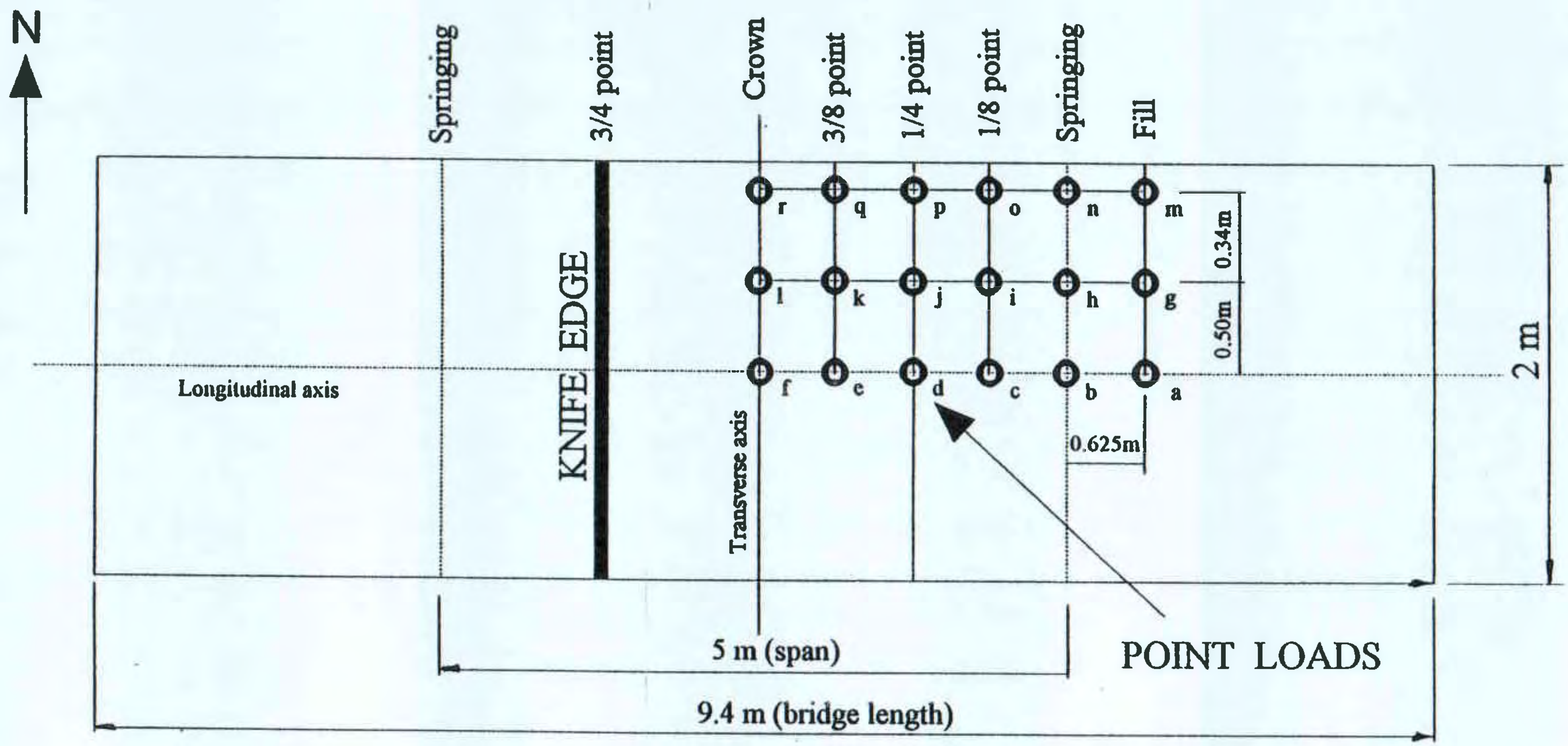
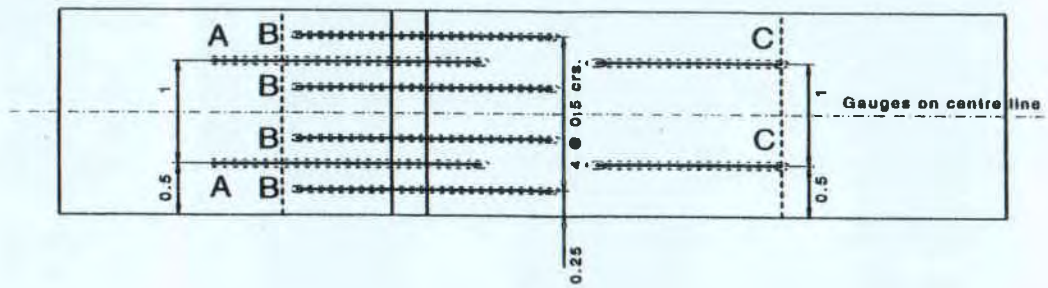


Figure 1: Plan view of arch loading positions



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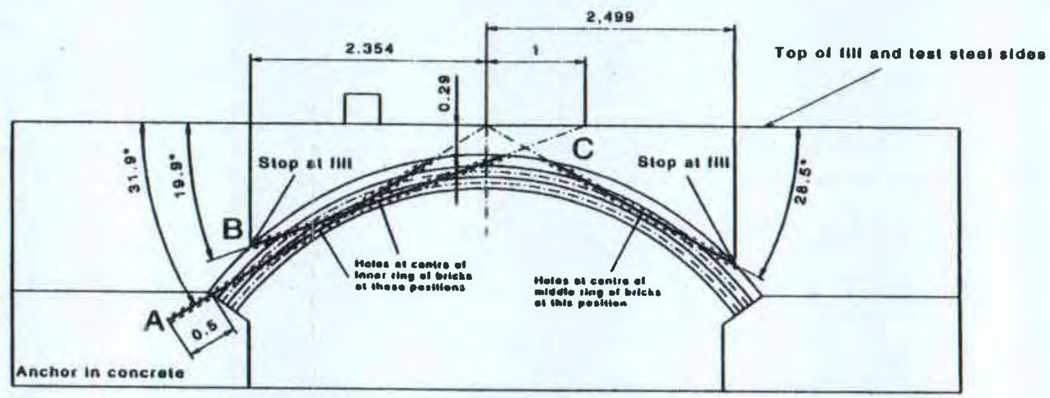

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General Arrangement of CINTEC Anchors

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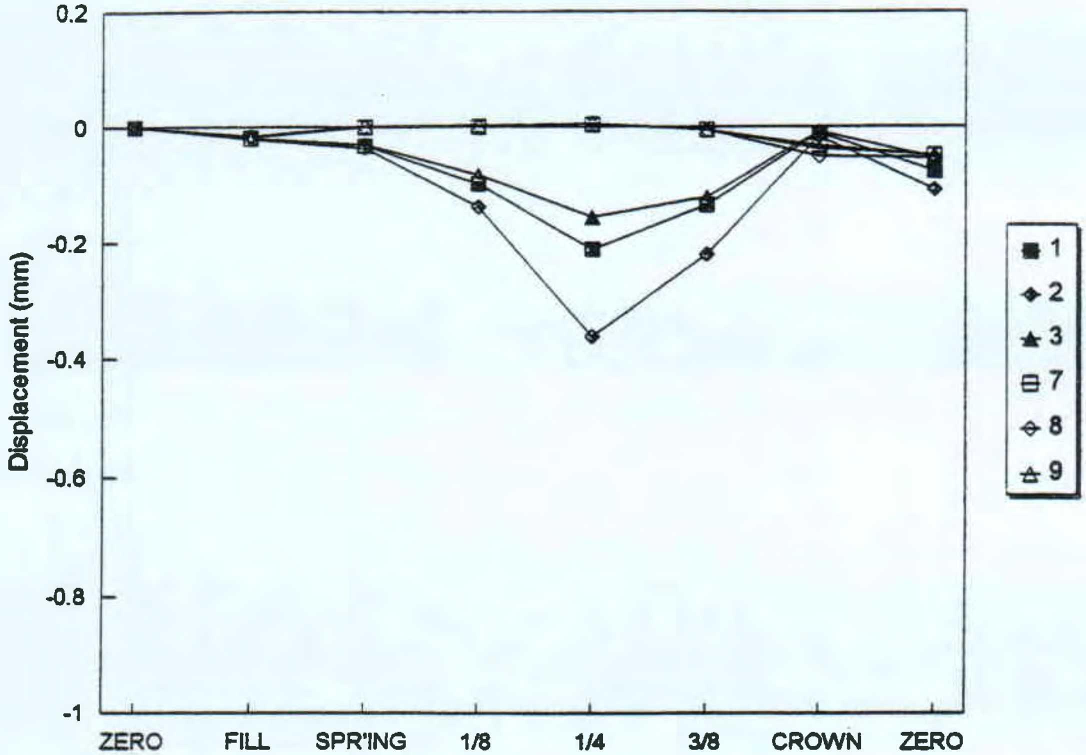
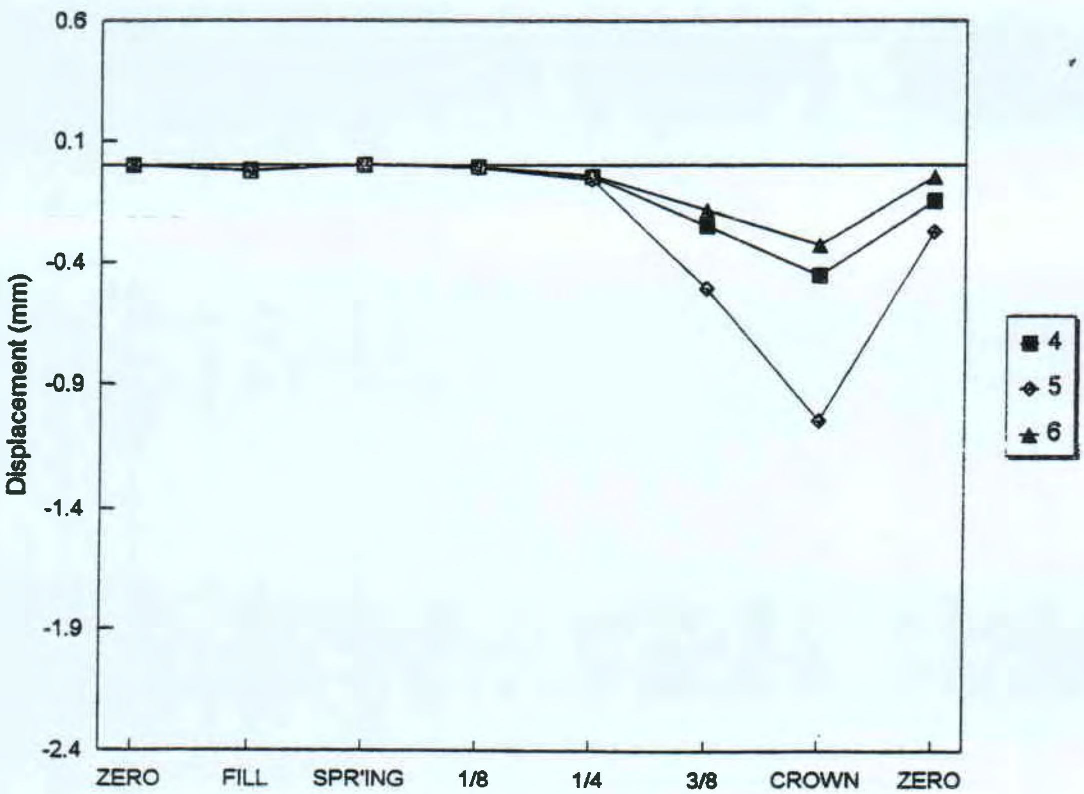
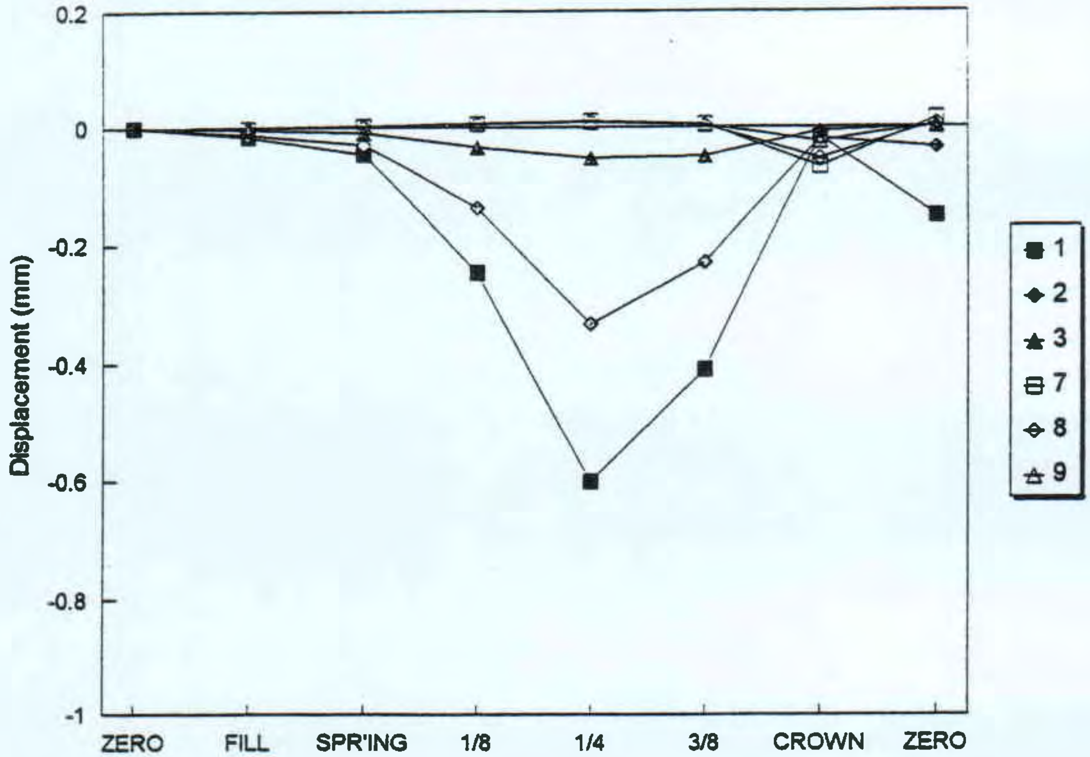


(i) LVDT gauges @ load-line and opposite load-line

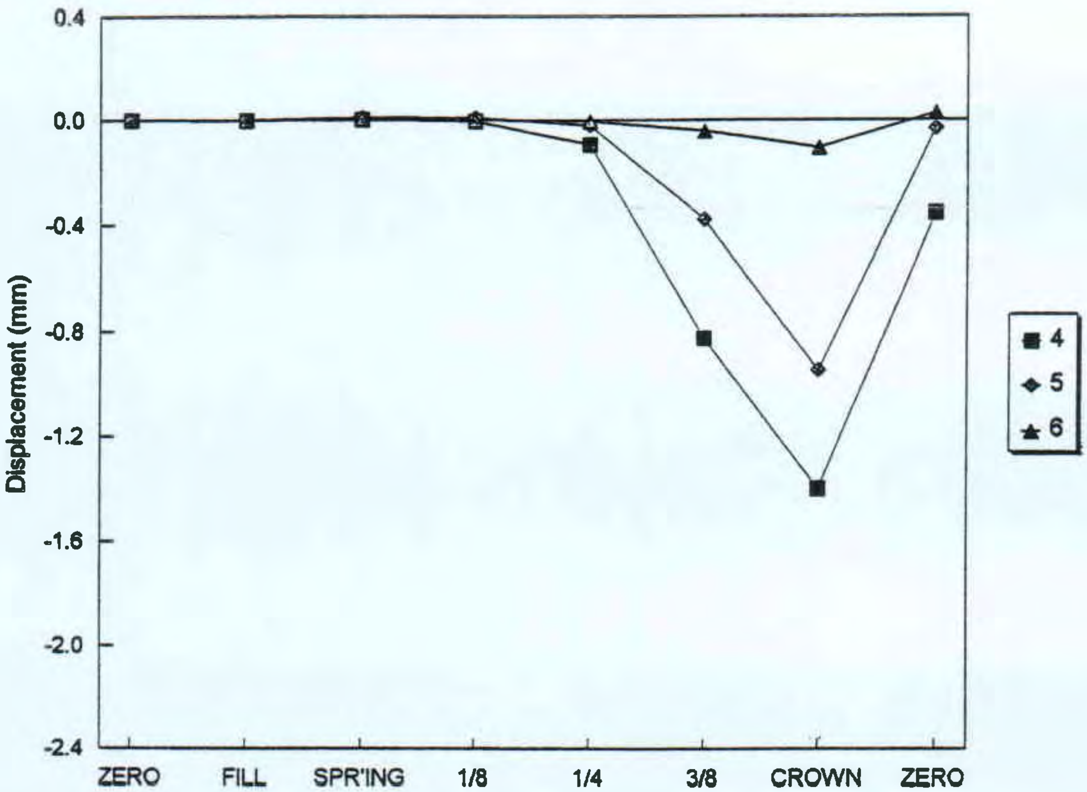


(ii) LVDT gauges @ crown

Figure 2a: Influence lines for displacement for 50 kN point loads applied along the centre-line

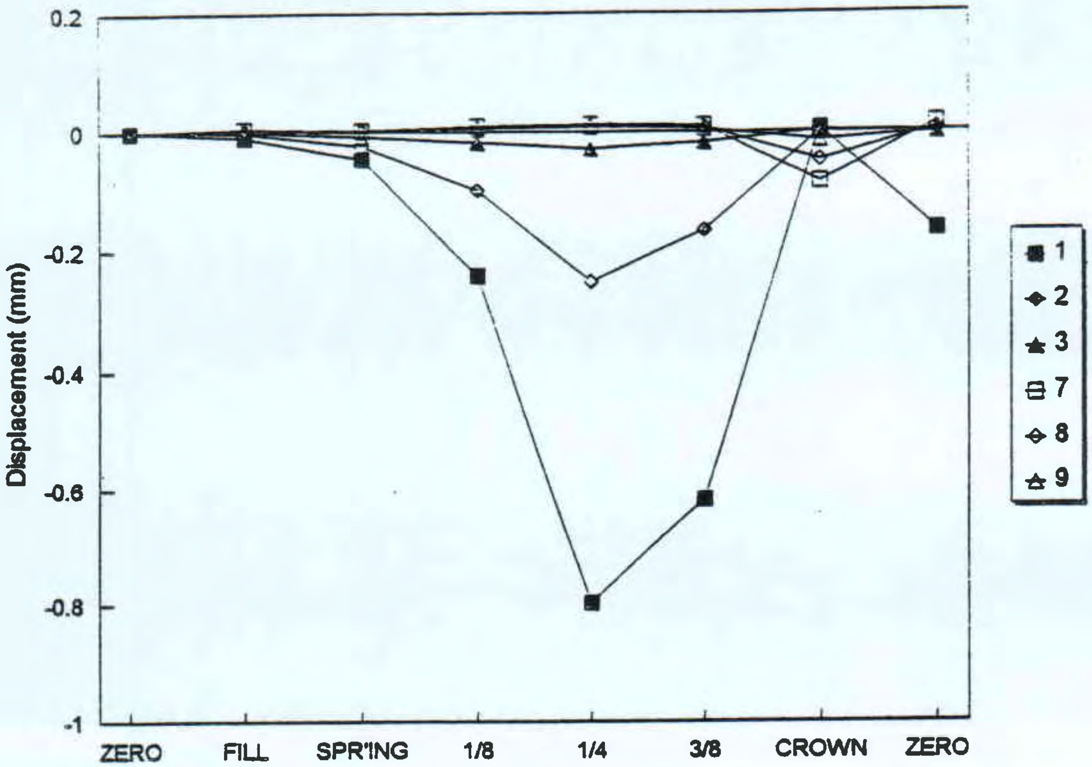


(i) LVDT gauges @ load-line and opposite load-line

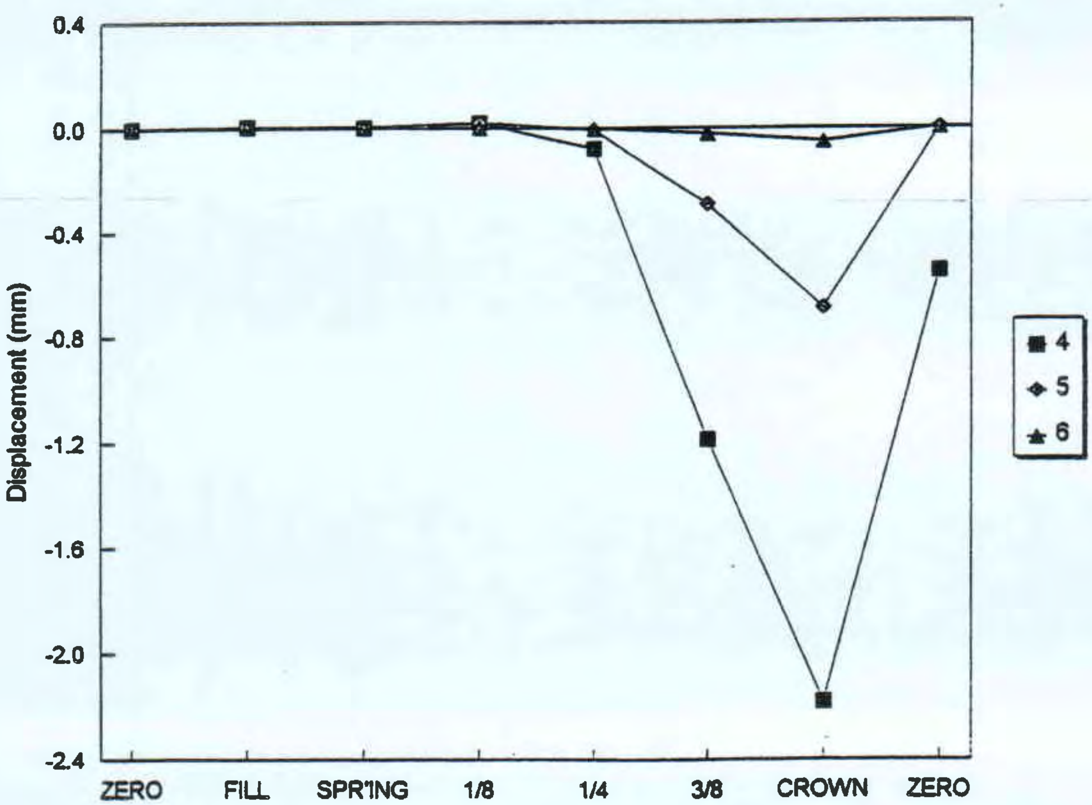


(ii) LVDT gauges @ crown

Figure No. [illegible] Influence lines for displacement for 50 kN point loads applied 0.50m from the centre-line



(i) LVDT gauges @ load-line and opposite load-line



(ii) LVDT gauges @ crown

Figure 2c: Influence lines for displacement for 50 kN point loads applied along the edge of arch

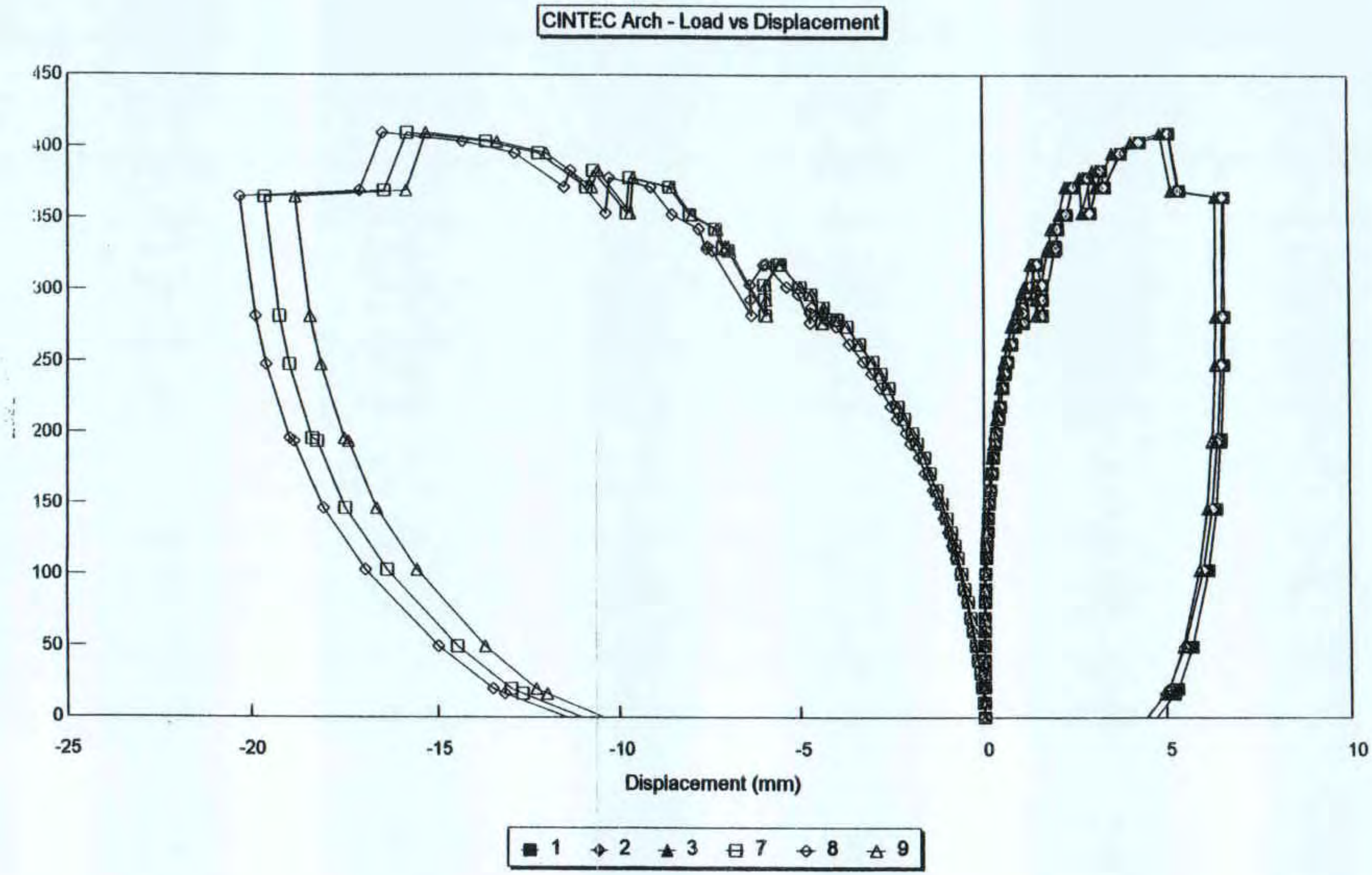


Figure 3a: Load vs displacement for LVDTs attached opposite quarter span (1,2,3) and under load-line (7,8,9)

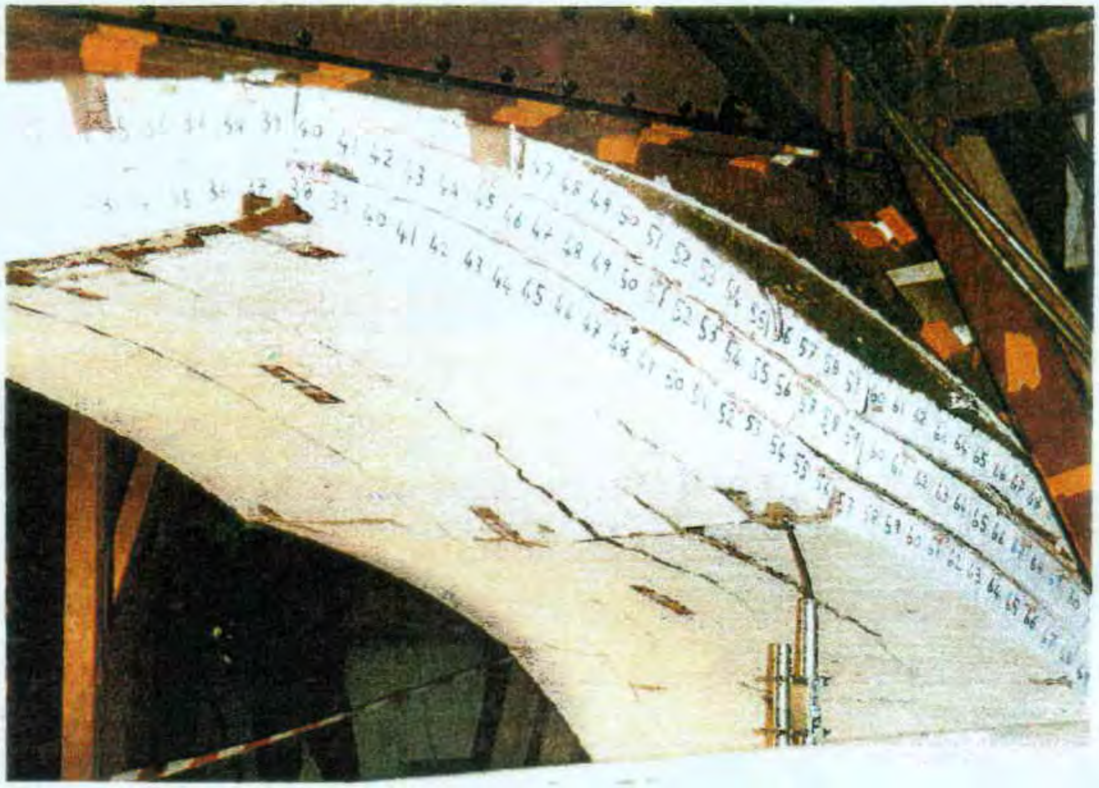


Figure 5: Arch just before bottom ring fell away

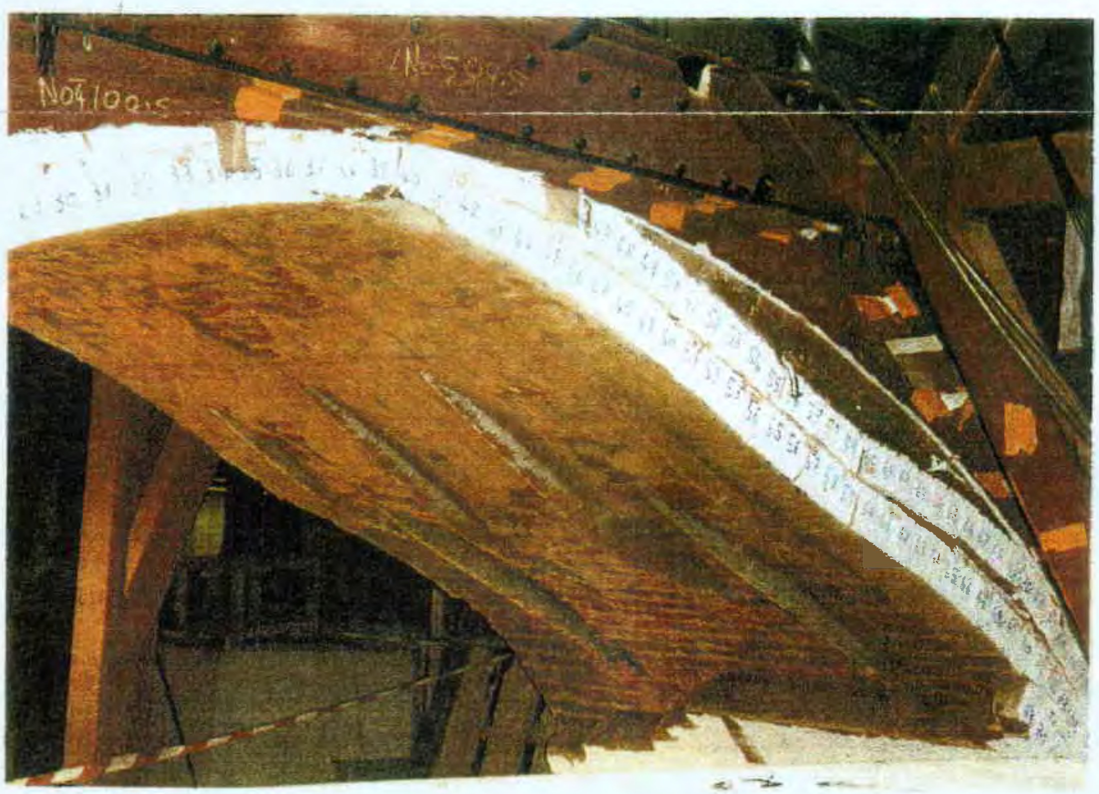


Figure 6: Arch after bottom ring fell away

Loading to failure of an Archtec strengthened brick arch using Cintec Multi-bar anchors

by A Sexton and G I Crabb

PR/IS/59/01
P9493

TRL Limited

PROJECT REPORT PR/IS/59/01

Loading to failure of an Archtec strengthened brick arch using Cintec Multi-bar anchors

by A Sexton and G I Crabb (TRL Limited)

Prepared for	Project Record:	P9493
	Client:	Cintec International Ltd

Approvals

Project Manager

Quality Reviewed

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LOADING TO FAILURE OF AN ARCHTEC STRENGTHENED BRICK ARCH USING CINTEC MULTI-BAR ANCHORS

ABSTRACT

This report been produced by TRL Limited under contract to Cintec International Limited to describe a load test to failure on a three-ring brick arch bridge. The arch was constructed with layers of wet sand between the three rings to simulate ring-separation. The bridge was then strengthened using the new Multiple Bar Cintec Anchor System. The test was performed to determine the increase in load bearing capacity of the strengthened arch bridge.

The arch was loaded at one quarter-span position. The maximum load reached was 448 kN. At this load the maximum vertical displacement under the load was 21.9 mm. This point was defined as "failure". The load was then removed and later re-applied and continued under displacement control until the arch collapsed. The mechanism was the development of hinges at the quarter points and springings with some crushing of masonry and shearing at the springing nearest the load.

The results are compared with those obtained from two previous tests. Firstly on an unstrengthened arch bridge (Sumon, 1997), and secondly on an arch bridge strengthened using Cintec Archtec anchors (Sumon, 1998).

1. INTRODUCTION

A test rig was been built in the Structures Laboratory at the Transport Research Laboratory to investigate this. The rig enables the construction of a 5 m span, 2 m wide, three-ring-brick arch which can then be tested to failure.

The arch was constructed without spandrel walls and no road surface, which had been left out to reduce the number of parameters being studied. Type 2 sub-base backfill was then placed and compacted. The fill was retained by a steel box, which has been designed not to restrain movement of the arch ring. It was then strengthened (see Section 3) and tested under controlled environmental conditions.

The arch was constructed on 27-29 April 2001, strengthened between 14th and 28th May 2001 and tested on 28 June 2001.

2. ARCH CONSTRUCTION

The arch had layers of sand between the rings rather than mortar to simulate ring-separation (delamination). Ring-separation is one of the common defects found in many old arch bridges. A brief account of the construction of the arch is given below.

The procedure was to build one ring at a time, on a steel centring, working up from the springings to meet at the crown. Handmade bricks were used, which best simulated those used in pre 1900 arch bridges. The bricks were weak by modern standards and in some cases fissured and distorted. In accordance with current practice, cement, lime and sand were mixed to give a cement/lime mortar. The mortar was mechanically with the minimum amount of water added to give the required workability. The nominal width of mortar used in the joints was 10 mm though some tolerance was allowed near the crown to ensure that the arch was completed without the need to cut bricks. The edges of the sand-filled inter-ring joints were pointed with mortar and the whole of the exposed brickwork was painted white to highlight crack formation.

3. ARCH STRENGTHENED WITH CINTEC ANCHOR SYSTEM

Twelve Cintec multi-bar anchors were used to strengthen the delaminated barrel. Fill was excavated approximately 1.2 m from the west-end and 1.4 m from the east-end of the arch to expose the extrados. Then using a long drilling rig, mounted to a purpose built scaffolding fixed to the arch rig, twelve 55mm diameter holes were drilled into the barrel from above. The anchors contained 6×S10 multi-bar stainless steel bars (grade 460) to BS 6744 (Figure 3). All the anchors were installed in 55mm diameter holes and grouted using Cintec Presstec grout in a Cintec fabric sock. Figure 1 gives the general layout and dimensions of the arch bridge. The positions and angles of installation of the anchors are shown in Figure 2, and a cross sectional diagram of the anchors is shown in Figure 3.

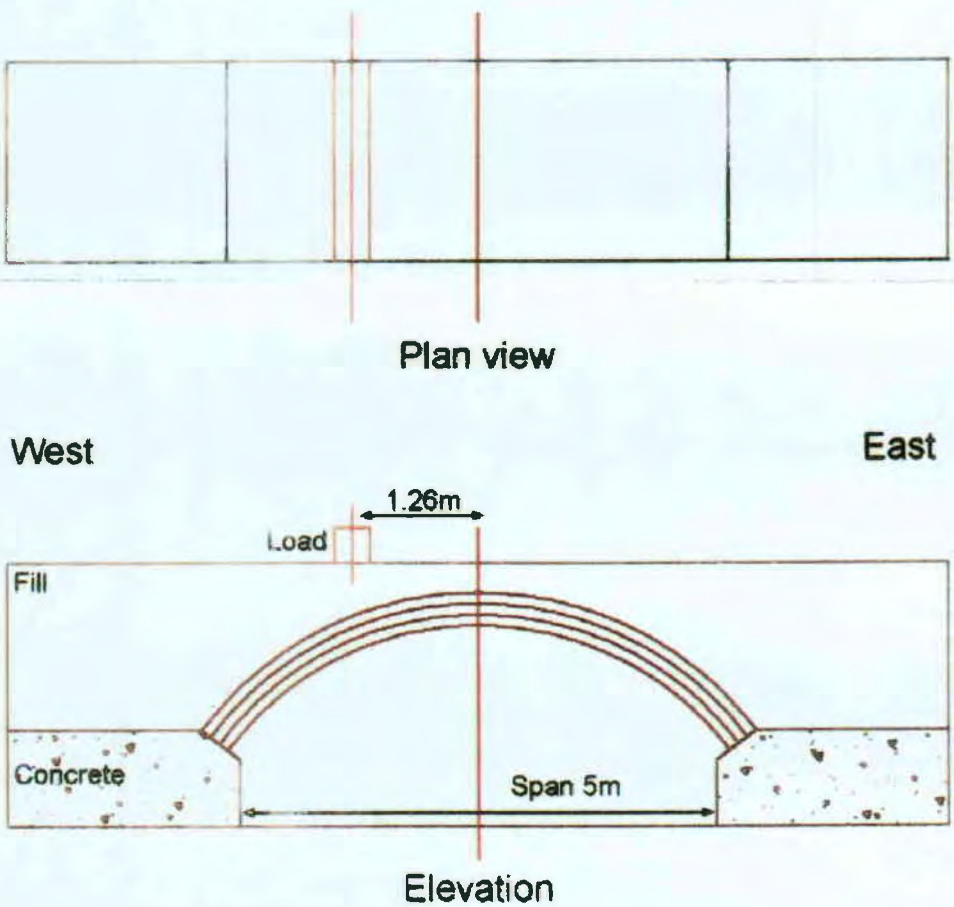

Figure 1. General layout of the arch bridge.

The anchors were arranged in three sets:

- three anchors were drilled and grouted into the abutment on the west side (Abutment Anchors),
- three anchors were drilled and grouted into under the east side third span point (Compression Anchors) and
- six anchors were drilled and grouted under the quarter point (Primary Sagging Anchors). One of the primary sagging anchors was equipped with electrical resistance strain gauges

4. MATERIALS USED

4.1 Bricks

Swanage Heathered Handmade type bricks were used. A mean compressive strength of 10 N/mm2 was obtained from the manufacture's literature.

4.2 Fill

The fill used for the back fill was a Type 2 sub-base material.

5. INSTRUMENTATION

5.1 Electrical resistance strain gauges

The second (of 6) Primary Sagging Anchor ("Smart Anchor") from the south side was equipped with strain-gauges. Twenty CEA-Series electronic resistance strain gauges were installed in pairs at 250mm intervals starting at 125mm from each end.

5.2 Displacement Gauges

Nine linear variable differential transformers (LVDTs); displacement transducers were attached to the arch soffit to measure vertical movement. There were three at each of the quarter-span crown (mid-span), and three-quarter-span positions. At each of these positions one gauge was attached on the longitudinal centre-line and one at each edge of the arch Solartron "B" series transducers were used. These have a stroke of ± 25 mm with a non-linearity of <0.25 %.

Table 1. Displacement gauge designations

	North	Centre	South
East ¼ point	1/4N	1/4C	1/4S
Mid span	1/2N	1/2C	1/2S
West ¼ point (load position)	3/4N	3/4C	3/4S

5.3 Data Recording Equipment

The vibrating wire strain gauge data was recorded using a Datataker system. The data from the LVDTs was acquired using an Orion 3350 data logger system. During loading of the arch data was acquired at approximately 2-second intervals. Data from both systems was transferred in real-time to an in-house logging program running on a PC. Selected data was plotted in real time and displayed simultaneously on four monitors, two for control of the test and two for informing the audience of test progress.

5.4 Crack monitoring

The formation and propagation of the cracks was highlighted on the south face of the structure in ink, on the south face only. The load at which each crack first appeared and its length were marked. These were recorded photographically (see accompanying CD). The arch ring voussoirs of the inner, central and outer rings of brickwork were numbered consecutively from 1 at the eastern end to 78, 81 and 85 respectively for identification

6. LOAD TESTS AND RESULTS

6.1 Load test to failure

The objective of this test was determine the load bearing capacity of the arch and the effectiveness of the applied strengthening method. The load was applied at the quarter-span on the west-end (see Figure 1) using a 3000 kN hydraulic jack. This bore on a 317mm wide "I" section spreader beam that spanned the full width of the arch fill. The structure was loaded in approximately 10 kN increments. The resulting strains and displacements were recorded at each increment. The arch was loaded until it could not sustain any further increase in load. Initially the test was load-controlled. When the displacement began to increase more rapidly a switch to displacement feedback was made to give better control of the failure.

A plot of the load versus displacement from the LVDTs can be seen in Figure 3.

A comparison with results obtained from the unstrengthened arch and the previous Cintec arch tests are given in Table 2 below.

Table 2: Summary of load tests to failure

Test to failure	Load to failure (kN)	Factor	Max. disp. @ max. load applied (mm)	Max. disp. after load removed (mm)
TRL Arch	200	1.00	27.40	23.40
Previous Cintec Arch	410	2.05	16.50	11.40
Cintec Arch	448	2.25	21.93	17.95

The first cracks were recorded around the crown circumferentially between the top/middle, and middle/bottom rings at a load of 80 kN in the region of middle ring voussoir 74 which was between the load line (at outer ring voussoir 61) and the springing (85). Initially very little damage was observed under the load-line. Increasing ring-separation in the region of voussoir 74 was the most visible sign of damage throughout this phase of the loading.

Damage continued to occur as the load was increased and hinges began to form at the load-line, at the opposite quarter span point and nearer to the springings on the load-line side. The arch was loaded until significant creep and plastic deformation had occurred, and it could not sustain further load. The maximum load applied to the arch was 448 kN. The load was then removed and there was some elastic recovery but there was still considerable deformation (Figure 4). This suggests that the strengthening method had some elastic properties. The maximum displacement was 21.93 mm which dropped to 17 95 mm when the load was removed.

Table 3. Min and Max data for arch test to failure.

LVDT's

	kN	mm								mm
	Load	1/4N	1/4C	1/4S	1/2N	1/2C	1/2S	3/4N	3/4C	3/4S
Max	448.20	5.33	5.29	5.36	5.40	5.27	5.29	0.00	0.00	0.00
Min	-0.79	0.00	0.00	0.00	-0.30	-0.33	-0.30	-27.01	-26.48	-24.57

Electrical resistance strain gauges

	microstrain									
	ERG1	ERG3	ERG5	ERG7	ERG9	ERG11	ERG13	ERG15	ERG17	ERG19
Max	116.00	128.00	197 00	43.00	34.00	3.00	4.00	197.00	718.00	258.00
Min	1.00	0.00	1.00	78.00	-226.00	-406.00	-308.00	-94.00	-4.00	-1.00
	microstrain									
	ERG2	ERG4	ERG6	ERG8	ERG10	ERG12	ERG14	ERG16	ERG18	ERG20
Max	21 00	227 00	262 00	458 00	1310 00	2019 00	1163 00	156 00	4 00	29 00
Min	1.00	1.00	63.00	-2.00	-1.00	-1.00	-3.00	-8.00	-299.00	-189.00

6.2 Test to collapse

All surface mounted instrumentation was removed prior to this phase of the test to avoid damage.

The arch was loaded to collapse under displacement control. The load was re-applied and reached a maximum of 421 kN before reducing steadily. The arch was pushed down at the load-line and up at the crown, and was breaking up internally as indicated by the rapid dropping of load. A high level of creep and plastic deformation was taking place. The ring separation in the region of voussoir 74 increased substantially. The arch remained held together by anchors until total collapse occurred by rotation at the hinges combined with some shearing at the western springing.

7. CONCLUSIONS

The following conclusions may be drawn from the test carried out:

- The load bearing capacity of the arch was increased by a factor of 2.24.

- The first crack and hinge did not occur under the load-line.
- The installed anchors delayed the formation of hinges.
- The anchors added considerable strength to the bridge.
- The arch failed in a gradual but a ductile manner.
- On unloading the structure recovered indicating some elastic behaviour.
- The bond between the anchor and masonry was found to be good.
- The strengthening was relatively easy to install.

8. REFERENCES

Sumon, S.K. 1997. Repair and Strengthening of Damaged Arch with Built-In Ring-Separation. Seventh International Conference on Structural Faults + Repair-97, held at Edinburgh, p. 69-75.

Sumon, S.K. 1998. Load test to failure on a ring-separated arch repaired using Cintec Anchor System. Unpublished Project Report PR/CE/61/98. Transport Research Laboratory, Crowthome.

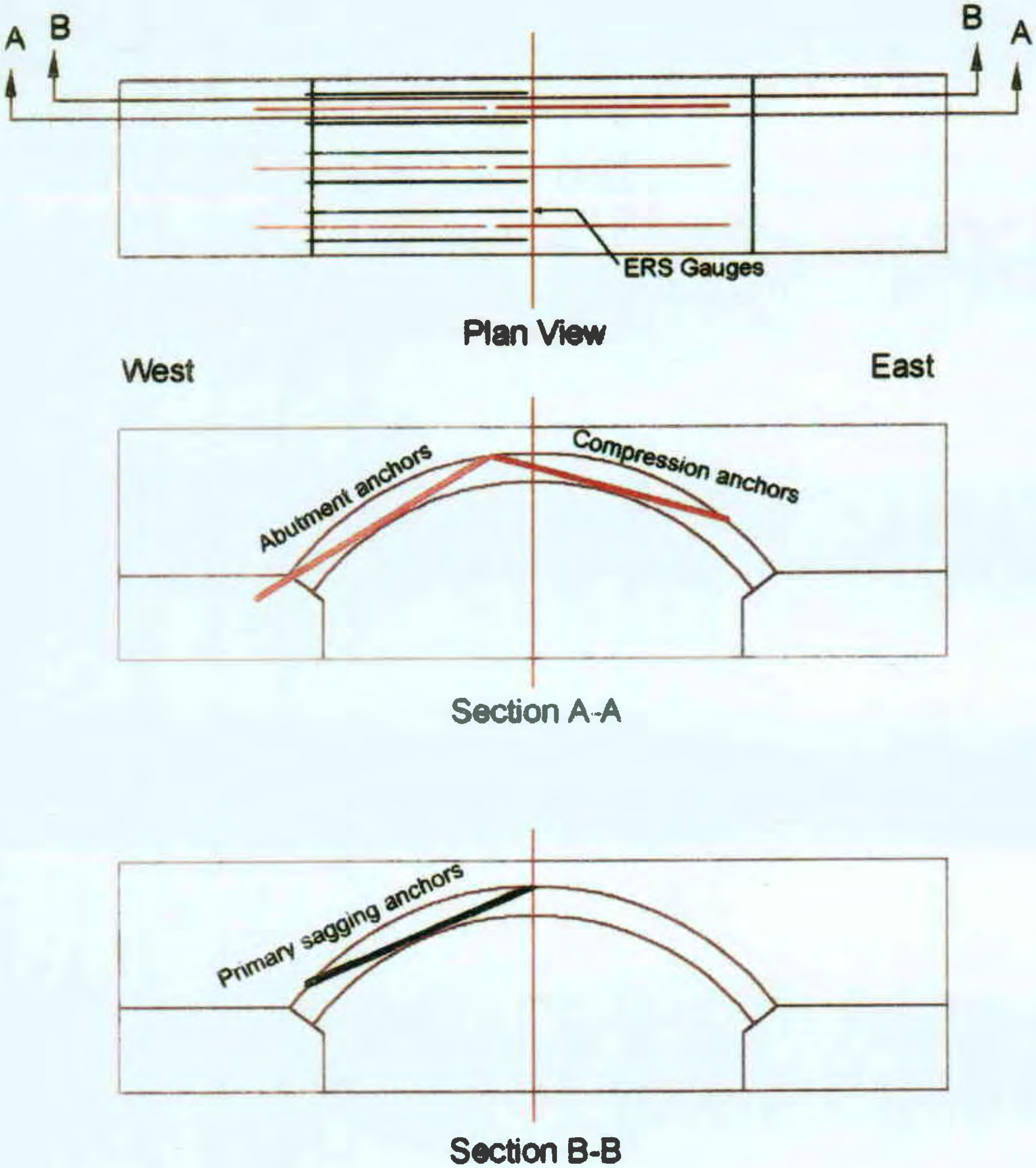

Figure 2. Layout and location of anchors.

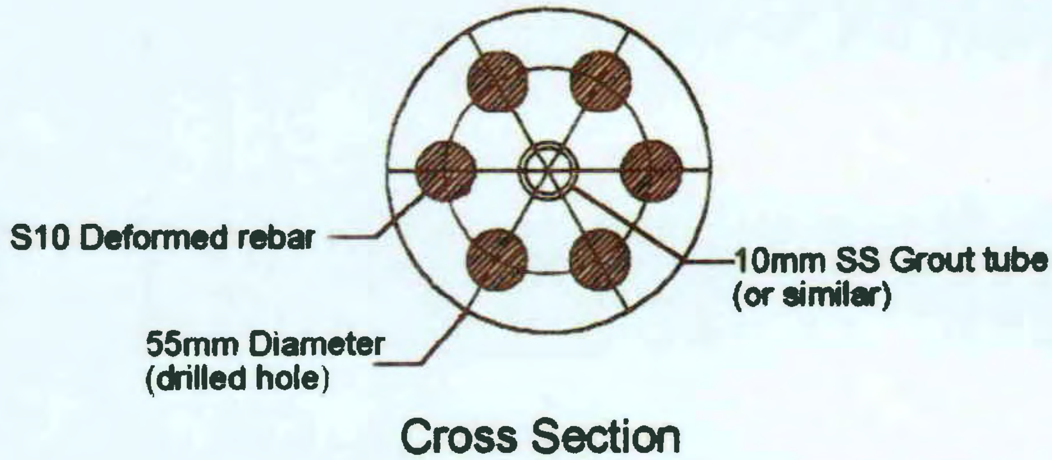

Figure 3. Cross Section of Cintec Multi-bar anchor.

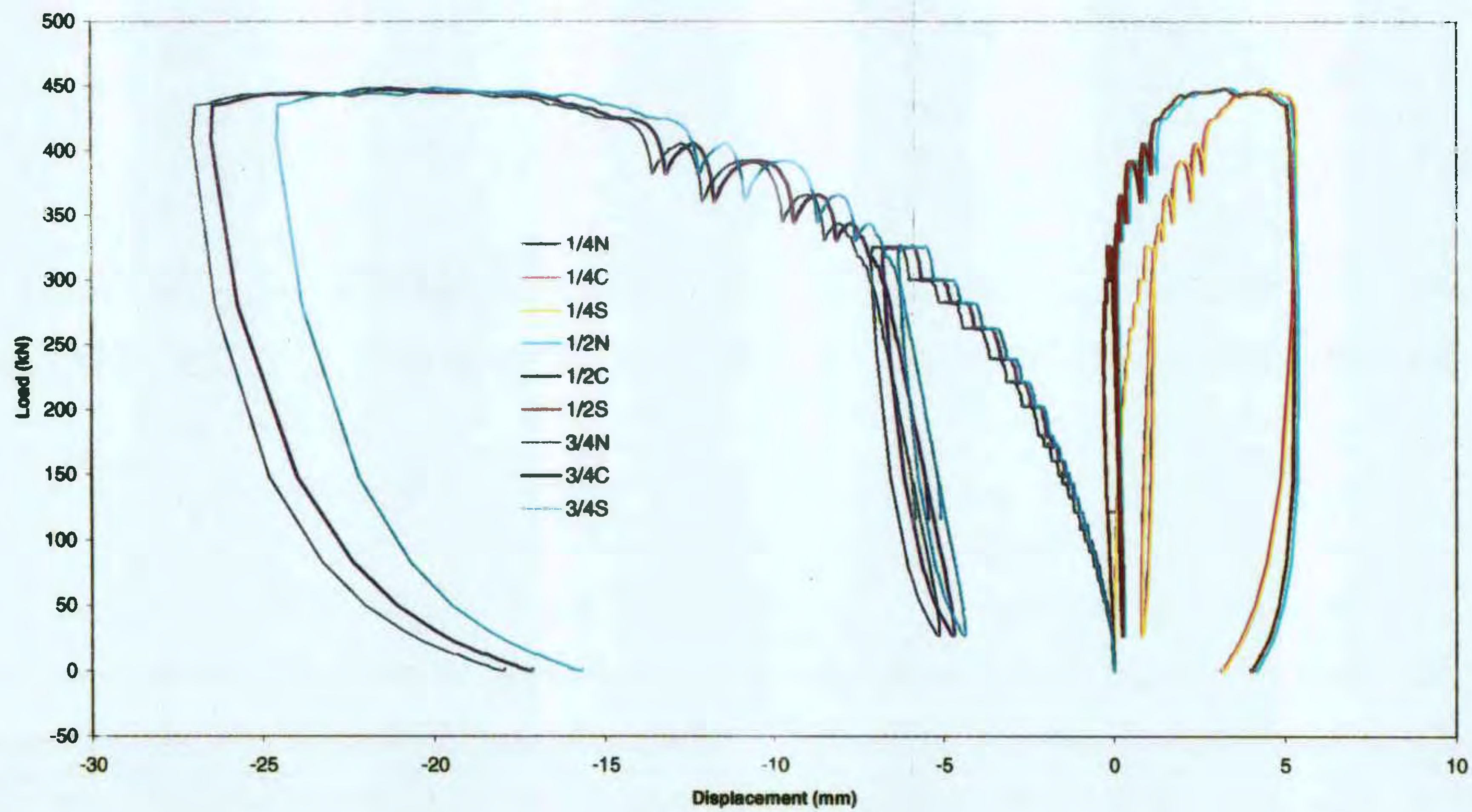

Figure 4. Load verses displacement for LVDT's attached opposite quarter span and under load-line

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Report No. B1660A/017/LT
February 2000

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STRENGTHENING AND LOAD TESTING

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STRENGTHENING AND LOAD TESTING

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APPENDIX A

APPENDIX B

ABSTRACT

Baber Bridge, a masonry arch carrying the A315 across the river Crane in Hounslow, was strengthened in December 1999 by the Archtec system. The Archtec system involves insertion of grouted stainless steel reinforcement in to the arch barrel. The strengthening was designed to increase the load carrying capacity by 60 per cent and enable the bridge to carry 40 tonne vehicles. On 14 December 1999, before strengthening, the bridge was load tested using a semi-trailer having a pair of close-spaced rear axles loaded to 18 tonne. After strengthening, on 12 January 2000, the bridge was again load tested. Comparisons of the responses confirmed that the strengthening had stiffened the bridge and reduced deflections by about 20 per cent. Measurements were also made under loads of 26 tonne and 34 tonne. Correlations between calculated and measured responses were very good.

1. INTRODUCTION

1.1 Background

The Environmental Services Department of The London Borough of Hounslow contracted Cintec International Ltd to strengthen Baber Bridge to a load carrying capacity of 40 tonne assessment live load. As part of the programme of work Gifford and Partners designed a strengthening scheme using Cintec anchors and carried out structural analyses before and after strengthening. The Transport Research Laboratory load tested the bridge, also before and after strengthening. Cintec were also asked to investigate the feasibility of strengthening the parapets to vehicle containment standard, and installing transverse tie-bars to restrain the spandrel walls. Schemes were prepared by Gifford and Partners but in the event did not proceed. The design to strengthen the parapet is outlined in Appendix A.

Baber Bridge is located on the A315, a heavily trafficked two-lane single carriageway road. The bridge crosses the River Crane near to Feltham and south-east of London Heathrow Airport.

The River Crane is normally slow moving and fairly shallow; at the time the site was inspected the depth was 0.5 to 1.0m. However, it is prone to rapid rises in level during storm conditions. The bed was fairly firm and composed of gravel and stones. There was no towpath or access beneath the bridge.

In view of the heavy traffic on the A315, Cintec was instructed to work from beneath the bridge to avoid road closures.

1.2 The Bridge

Baber Bridge is a single-span brick arch constructed circa 1798, having a span of 6.08m on the southern (downstream) side and 7.73m on the northern (upstream) side, see Figures 1 to 5. It was believed that it had been asymmetrically widened some time after original construction. On the south side there was a footpath with a raised kerb. The parapets were constructed in brick, uniform with the rest of the bridge.

The intrados and southern elevation, up to the base of the parapet, had been gunited. The gunite exhibited fine map cracking which had initiated through shrinkage when it was originally placed. There was a longitudinal crack on the intrados indicating splitting between the spandrel and barrel. This is a fairly common occurrence and is not considered to be structurally significant. On occasions transverse tie-bars and pattress plates are often fitted to restrain outward movement of the spandrels.

The exposed brickwork was soft and friable and this may have been the reason why the rest of the bridge had been gunited.

Figure 1 South Elevation

Figure 2 North Elevation

Figure 3 Roadway

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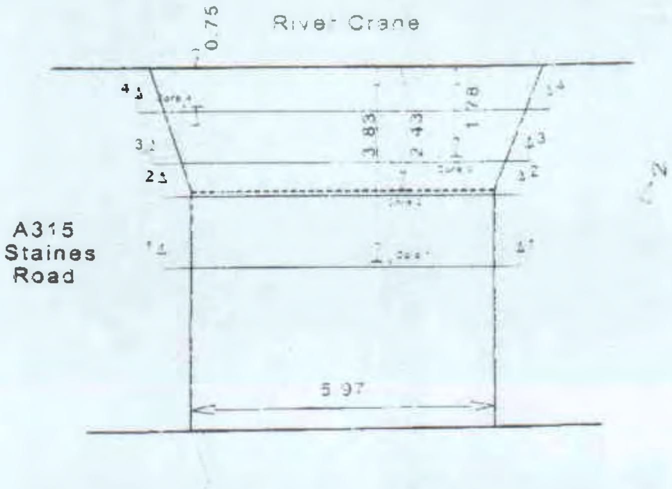

Figure 4 Plan

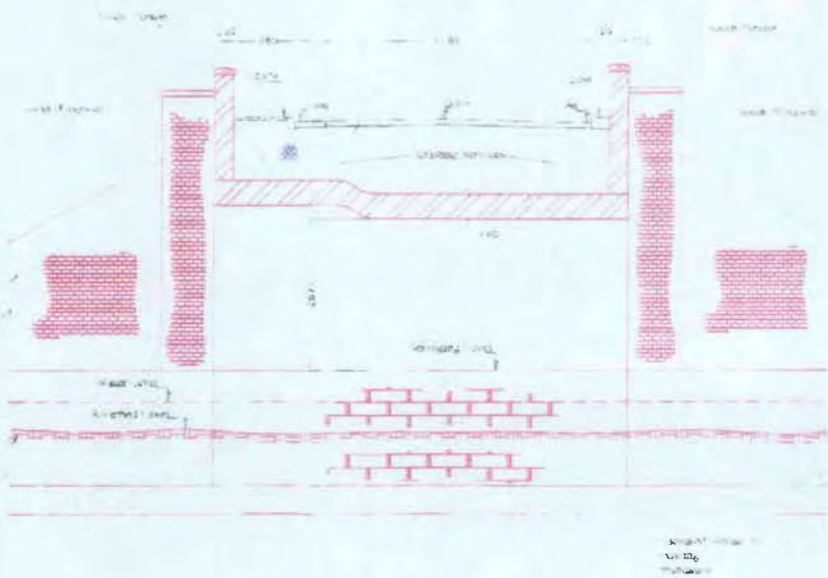

Figure 5 Cross-Section

The thickness of the arch barrel appeared to be about 440mm as evidenced from the surface of the north side. However, the through-thickness is not necessarily the same as on the side elevation.

Utilities (gas, telephone and stormwater) were marked on drawings of the bridge but precise locations were not given.

2. STRENGTH ASSESSMENT

The load carrying capacity had been previously assessed and found to be inadequate, using the MEXE method and the MAFEA program (Masonry Arch Finite Element Analysis). The Assessment Engineer considered the MEXE method to be the more meaningful in this instance. Bearing in mind the friable brickwork and the need for guniting, a condition factor of 0.6 was assigned. Gifford carried out further MEXE sensitivity analyses and confirmed that, with axle lift-off accounted, the strength is insufficient.

As part of the design procedure towards strengthening the bridge, it was necessary to carry out a more comprehensive structural analysis using ELFEN (Welsh for element), a discrete element program. This was also required in support of the load tests to be carried out by TRL.

2.1 Condition of the Arch Barrel

Investigative coring was carried out to provide information required for the ELFEN analyses:

- thickness of the brick arch barrel
- condition of the brick in the barrel
- thickness of the gunite cover

Messrs Castle and Prior carried out the work on 25 August 1999. Four 50mm diameter cores were drilled into the intrados and the results were as follows:

- the brick barrel is a minimum 440mm thick
- brickwork in the northern (upstream) section of the barrel is in good condition
- brickwork in the southern (downstream) section is in poor condition
- thickness of the gunite varies between 35mm and 55mm, it is reinforced with steel wire mesh of 2mm diameter, and is in good condition

The poor condition of the brickwork on the southern side of the bridge is consistent with the need to preserve and strengthen it with reinforced gunite.

2.2 Location and Condition of Utilities

A small trench was dug across the width of the bridge on the night of 13 December in order to determine the position of the utilities, and most importantly, gas and storm water pipes that could be damaged by injudicious load testing. The location of the trench is shown in Figure 6. Prior to digging, it was noted that there appeared to be a smell of gas and, as the trench was dug, the smell became stronger. This was in accordance with comments by a gas engineer who had visited the site recently and noted that there were some 120 known leaks in the environs. The gas pipe was exposed and shown to be 230mm diameter cast iron having surface rust but no serious corrosion. The storm water pipe was 500mm diameter cast iron. Locations of the pipes are shown in Figure 7.

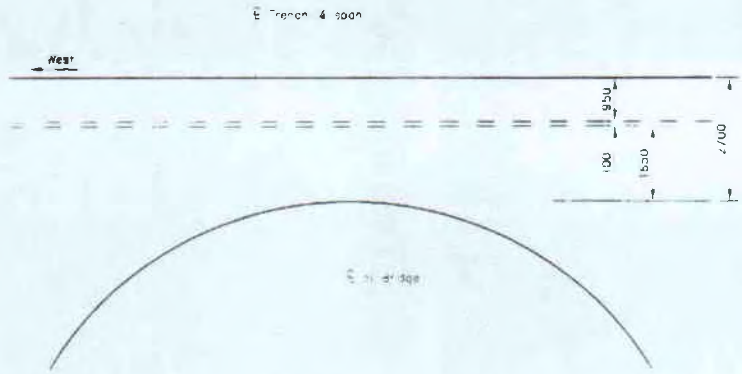

Figure 6 Position of Investigative Trench

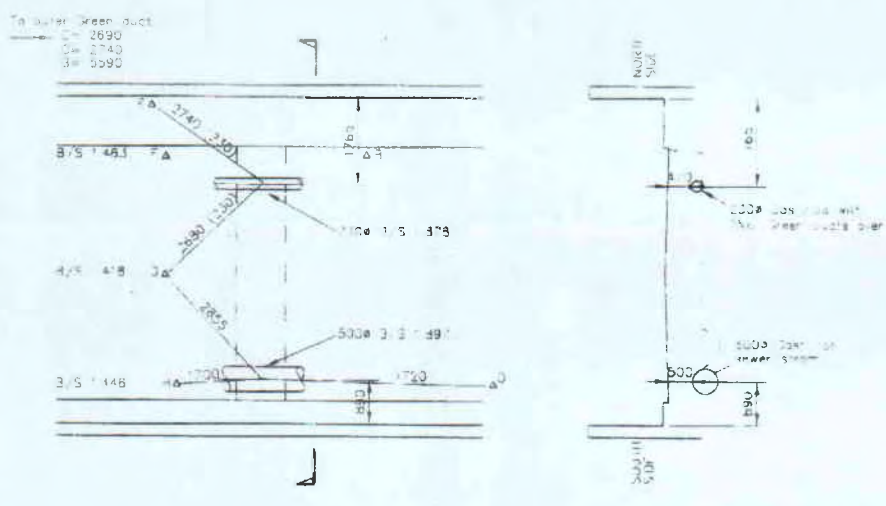

Figure 7 Positions of Utilities

3. FINITE DISCRETE ELEMENT ANALYSIS

3.1 Background

The origins of the discrete element technique can be traced back to the late 1960s when it was developed to investigate the behaviour of jointed rock. Later improvements included the introduction of deformable behaviour and more recently work has concentrated on improved physical models and better computational efficiency. The computer program ELFEN by Rockfield Software Ltd at The University of Wales, Swansea, represents the state of the art in this technology. Gifford and Partners exclusively use ELFEN, the only industrial quality finite discrete element package in the construction industry.

The finite discrete element method is well suited to simulation of non-homogenised continua such as masonry, concrete and soil. By representing separate parts that can deform and interact with each other, highly dynamic and non-linear systems, both in two-dimension and three-dimension, can be modelled more simply at a fundamental level. Many thousands of parts can be represented each with prescribed friction/contact laws at their boundaries. The capability to evolve further parts by fracturing into separate fragments is also possible by using limiting tension non-linear material models and advanced mesh adaptivity schemes. Efficient solvers based on explicit dynamic algorithms enable many classes of problem to be solved that would be near impossible by conventional analysis. Typical applications include

- Numerical simulation of masonry used for buildings and bridges under dynamic and static loads. Considerable experience has been gained in seismic, blast and ultimate static strength applications.

- Numerical modelling of vehicle impact with experience gained in crashing of light passenger vehicles and the simulation of major vessel collision protection systems.

- The prediction of the ultimate strength of complex reinforced concrete assemblies in bridges.

3.2 Verification of the Methods of Analysis

In view of the fact that the ELFEN structural analysis is in advance of the state-of-the-art represented in current standards and other programs, it was verified against full-scale tests on Torksey Bridge and laboratory large scale model tests conducted by TRL[1,2,3]. Torksey Bridge was located across a drainage channel near the village of Torksey, Lincs. It had a span of 4.9m and was constructed in brick. The arch barrel was composed of three rings. The spandrels were disconnected from the barrel by wide cracks so that behaviour could be approximated to two-dimensional. In November

1996, it was tested to collapse using a transverse line load at the quarter-span[3]. The collapse load was 109 tonne compared with a calculated value, using ELFEN, of 108 tonne.

A full-scale model brick arch, very similar to Torksey Bridge, was constructed in the TRL laboratory[2] The arch barrel was composed of three rings of brick, the rings being separated by layers of sand so that they were not bonded together. This represents the commonly found fault in arch barrels of ring separation. The arch was 5m span, 1.25m rise at mid-span and 2m wide. There were no spandrels but, instead, steel containment walls, not connected to the arch barrel, enable fill to be placed and compacted in the normal way. This generated a two-dimensional structural action similar to that at Torksey.

Loading was by a hydraulic jack positioned on a transverse beam so that a nominal line load was applied across the top of the model bridge at its quarter-point. Loading was applied in increments of 1.0 tonne. When the response became significantly non-linear, the control was changed to deflection. This enabled the load-deflection characteristics to be fully investigated and the collapse mechanism to be observed beyond maximum load.

The model bridge failed at a load of 20 tonne compared with the calculated value of 18.6 tonne.

An identical model arch was constructed in the laboratory using the same formwork and lime-based mortar. The same bricklayer was employed to ensure the same quality of construction. The model was strengthened using eight Cintec anchors positioned longitudinally in the arch ring and tangential to the curvature. The work was carried out from the top of the arch.

Loading was applied in the same way as for the un-strengthened arch. Failure occurred at 41 tonne, an increase in strength of over 100 per cent.

4. STRENGTHENING SCHEME

4.1 Analytical Model

The bridge structure, including the brick barrel, abutments, fill and Cintec anchors, was analysed using the Archtec version of ELFEN. A two-dimensional plane strain model was written to meet the requirements of Baber Bridge and the general modelling procedures developed by Gifford for analyses of masonry structures were followed.

The masonry was represented non-homogeneously by separately modelling the brick units and mortar. The fill was modelled as a non-linear continuum. Material models were as follows:

Masonry units:	non-linear von Mises in the compressive domain
Masonry-to-mortar:	Coulomb friction
Fill:	non-linear Rankine
Cintec anchors:	non-linear von Mises
Masonry-fill-grout:	Coulomb friction

The initial and permanent stresses were calculated as construction events before the introduction of the arch barrel strengthening and live loading. The idealised structure is shown in Figure 8.

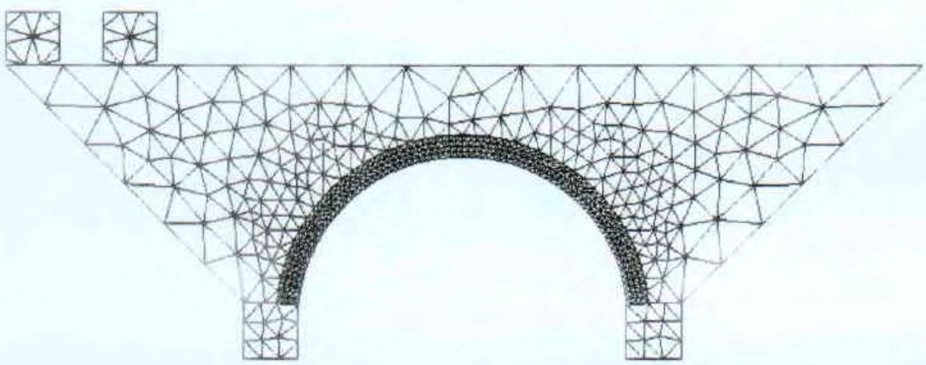

Figure 8 Idealisation of Baber Bridge

The longitudinal section of the bridge was based on the critical section corresponding to the most severe effects of fill depth and load distribution. The transverse distribution of live load was in accordance with BD21/97.

Structural element stiffness was calculated in the analysis using an accurate representation of the masonry components. This was equivalent to preventing direct tension from developing across any mortar joint. Development of cracks associated with different loads therefore caused structural elements to have automatically calculated stiffnesses.

Mobilisation of active and passive pressure effects in the fill were calculated directly in the analysis. The pressures develop as the barrel deforms with the fill being able to support the carriageway live loading and developed thrust lines by biaxial compression.

Material properties adopted for the analysis were as follows:

Masonry type:	brick
Mortar:	1:3 lime/sand
Characteristic strength:	2.3N/mm^2

For Cintec anchors:

Reinforcement type:	Stainless steel Type 2 ribbed
Characteristic strength:	490N/mm^2
Grout/masonry ultimate shear strength:	1.8N/mm^2

4.2 Disposition of Reinforcement

Archtec is a novel system of strengthening masonry arches where stainless steel reinforcing bars are inserted and grouted into the masonry. The use of stainless steel and a high performance grout ensures that there will be enhanced durability. Most importantly, the bars and grout are contained within a 'sock', which protects the surrounding masonry from being displaced or otherwise damaged by the grouting pressure of 3 to 4 bars. During inflation, the sock deforms to fill any voids or cavities that may be present and permits sufficient 'leakage' of grout to develop chemical and mechanical bonding with the masonry resulting in a structural connection. The efficacy of this connection is evaluated by pull-out tests. The reinforcement is positioned in the arch barrel in a longitudinal direction and tangential to the curvature. Depending on the condition of the structure, reinforcement may also be positioned in the barrel in a transverse direction.

In consequence of the requirement to avoid closures of the heavily trafficked A315 road, the strengthening scheme was designed to be installed from beneath the arch barrel. Drilling was from scaffolding installed in the river bed and so called J-bars were fitted. A J-bar is composed of a pair of bent bars connected by a mechanical coupler. The positioning of holes drilled for J-bars is shown in Figure 9.

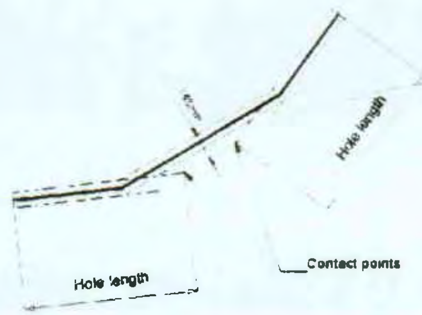

Figure 9 J-bar Detail

Installation of a J-bar required two 85mm diameter holes to be drilled in the relative positions shown in Figure 9 and an interconnecting slot to be chased out of the surface of the intrados, see Figure 10. The bent bars are inserted into the holes and connected by a screw coupler. When grouting has been completed, the slot is made good using material that matches the intrados, in this case gunited mortar having a characteristic dark colour.

The disposition of the J-bar strengthening scheme is shown in Figure 11. Seven stainless steel ribbed bars were installed in each of the ¼-points of the arch. This scheme was calculated to raise the load carrying capacity of the bridge by 60 per cent.

Figure 10 Installation of J-Bars

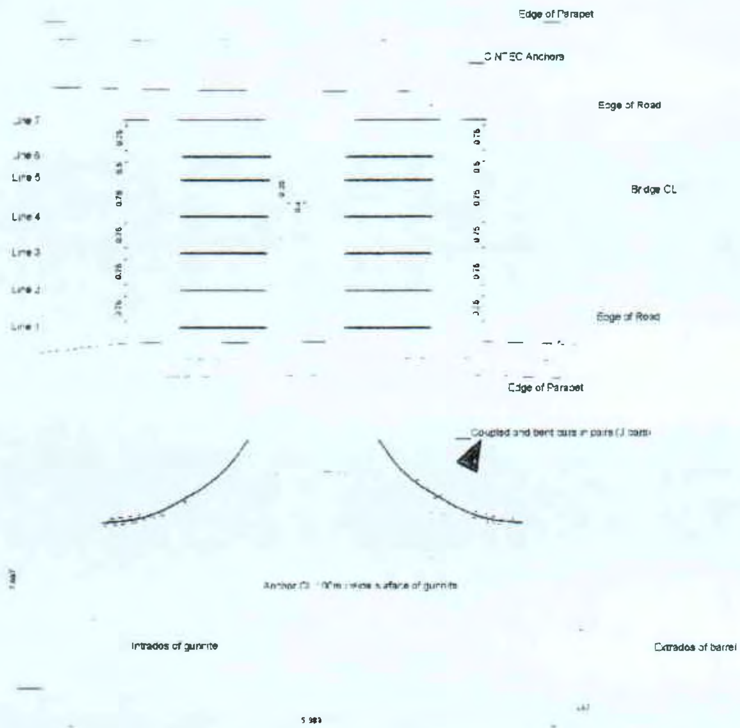

Figure 11 The Strengthening Scheme

5. LOAD TESTING

The load tests were carried out by TRL using a 12.2m (40ft) semi-trailer having a single twin-axle bogie with axles spaced 1.4m apart, as shown in Figure 12. The axles were loaded using 0.5tonne concrete blocks transferred by crane. The loading was measured using portable weigh pads, Figure 13.

Figure 12 Loading Arrangement for Semi-trailer

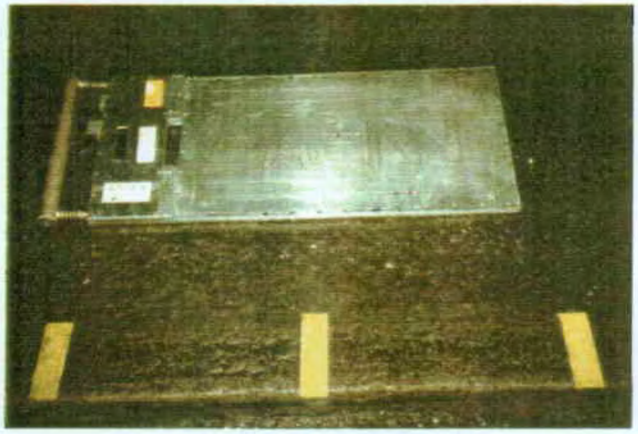

Figure 13 Weigh Pad

The general procedures for planning and carrying out the load tests were in accordance with the ICE Guidelines for Supplementary Load Testing (4). The load tests were carried out at night to minimise interruption to traffic and, most importantly, at a time when temperatures in the structure are stable. On many structures transient temperature effects can be very significant.

5.1 Instrumentation

Instrumentation was fitted beneath the arch, working off scaffolding. Deflection gauges were linear variable displacement transducers (LVDT's) having a stroke of 50mm and resolution to 0.001mm. They were fixed to rigid steel stands and their moving parts attached to the structure by wires and hooks fixed to the arch with adhesive. Strain measurements were by surface-mounted vibrating wire gauges (VW gauges) having a gauge length of 140mm, range of 3000 micro strain, and resolution of 0.5 micro strain. Four of the 17 gauges were fitted with temperature measurement. The gauges were fitted to mounting plates which could be left in place in order that the gauges could be removed after the first test and replaced in the same positions for the second. Locations of the gauges are shown in Figure 14 and a photograph of gauges in location is shown in Figure 15.

When the bridge was strengthened some additional newly developed instrumentation, a Smart anchor, was fitted. This is described in Appendix B.

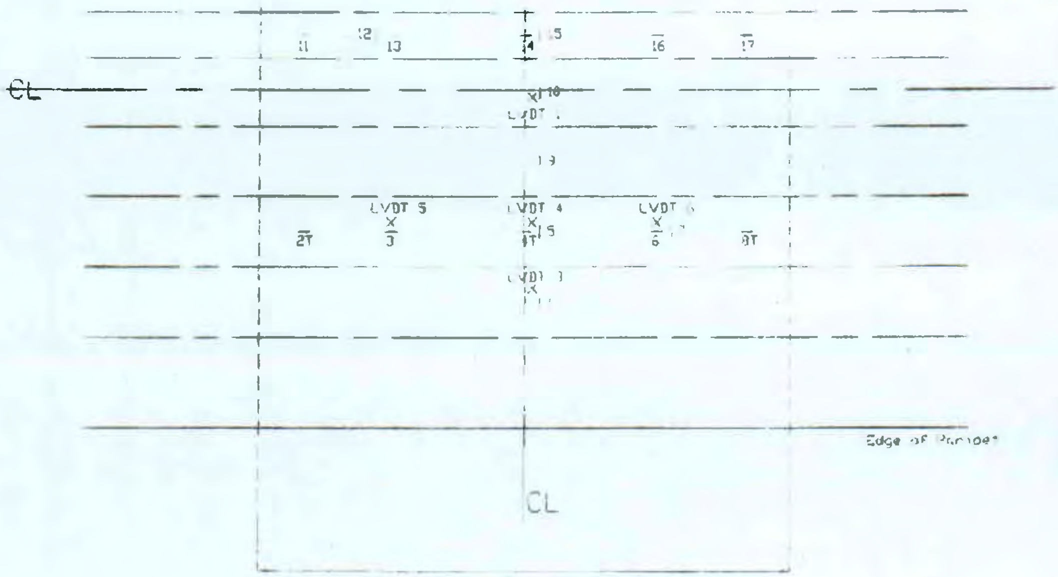

Figure 14 Locations of Instrumentation

Figure 15 Deflection and Strain Instrumentation in Location

5.2 Load Test Before Strengthening

The load test before strengthening was carried out on the night of 14 December 1999.

The twin-axle was loaded to nominally 18 tonne and weighed; its actual weight was 18.775 tonne. It was then moved across the bridge in increments of 0.5m. The runs across the bridge were in Lane 1, along the centre line, and in Lane 2. A full set of deflection and strain measurements was recorded at each position.

The deflection and strain data are provided in a separate volume[6].

Results of the load test are shown as longitudinal influence lines of deflection in Figure 16. There are three influence lines related to gauges 5 and 6 located at the quarter points and gauge 4 at the centre of the arch. The maximum deflection was 0.12mm when the axle was positioned at the centre of the arch.

The first loading position was with the centre of the first of the axles vertically above the springing point of the arch and the second axle 1.4m behind. This loading produced deflections across the arch and confused the starting conditions. In order to simplify interpretation of the influence lines, deflections were normalised to zero at position zero on the span. This simplification facilitated comparisons between calculated and measured deflections. Uplift exhibited by the measured influence line for the quarter-point at 2m (LVDT 5), when the vehicle was positioned at 8m, is associated with loading by the second axle and enhanced by normalisation. In fact, all gauges returned to zero when the vehicle was completely removed.

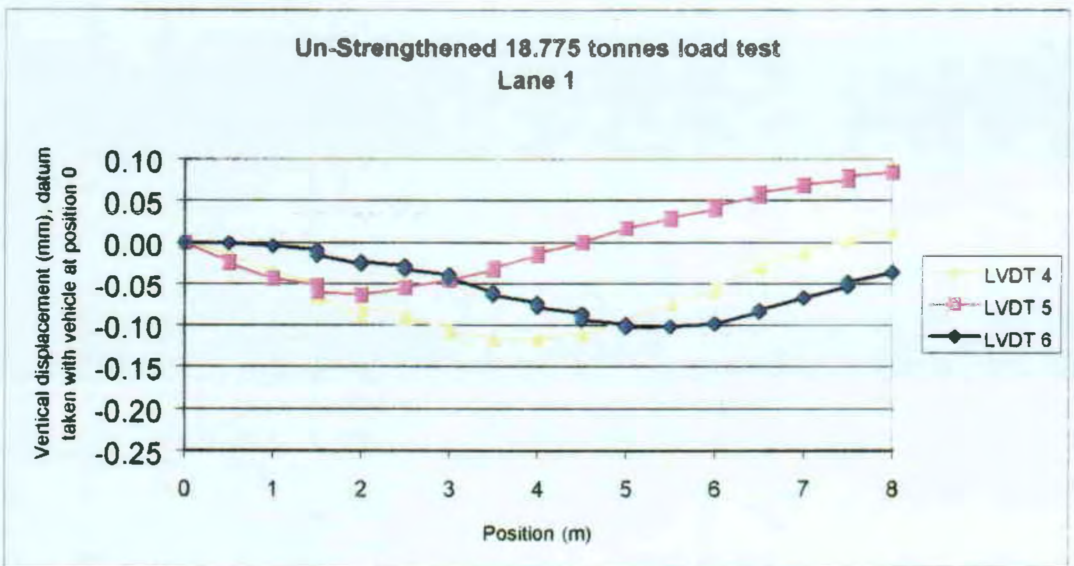

Figure 16 Longitudinal Influence Lines Measured Before Strengthening

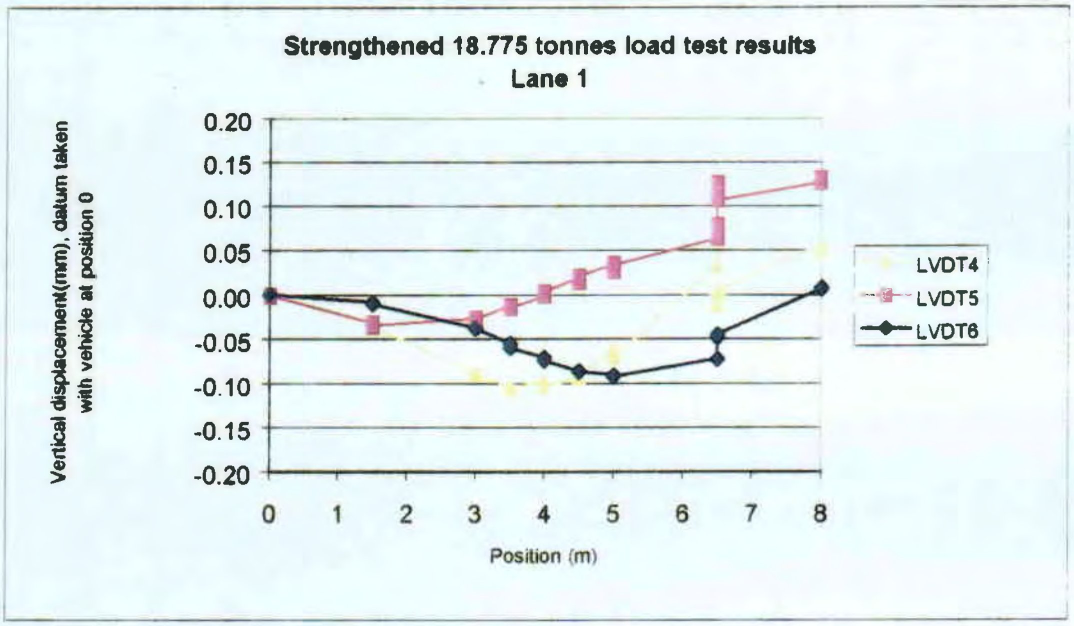

Figure 17 Longitudinal Influence Lines Measured After Strengthening

5.3 Load Test After Strengthening

The load test after strengthening was carried out on the night of 12 January 2000. The ambient temperature was around 5°C, it was rather windy and a tarpaulin was erected to protect the gauges. The runs with a loading of 18 tonne were repeated as for the test before strengthening in order to provide a like-for-like comparison of behaviour but several of the loading positions were deleted to ensure there was time to carry out three load tests.

After completion of the runs with 18 tonne, similar tests were carried out with 26 tonne (measured load 27.206 tonne) and 34 tonne (measured load 35.425 tonne). For the higher loads it was necessary to increase the rigidity of the axle suspension using specially designed steel plates and screw jacks. The higher axle loads were measured using portable weigh pads as before.

The deflection and strain data are provided in a separate volume[6].

Results of the 18 tonne load test on the strengthened arch are shown as longitudinal influence lines of deflection in Figure 17. As with the unstrengthened arch there are three influence lines related to the LVDT gauges 5 and 6 at the $^1/_4$-points and gauge 4 at the centre of the arch. The maximum deflection was 0.10mm when the axle was at the centre of the arch.

The influence lines were similar to the unstrengthened case but were spoiled at 6.5m and 8m. When the loading was at 6.5m, readings taken for the Smart anchor caused minor disturbance of the scaffolding and the apparent step changes in deflections of LVDT's 4, 5 and 6. Further problems occurred during the load test at 28 tonne due to the tarpaulin blowing onto LVDT's 4 and 5 so that reliable measurements were limited to LVDT 6. In the load test at 36 tonne, LVDT 5 gave erratic and unreliable readings.

Influence lines measured by LVDT 6 at the far quarter-point, for the three loads, are given in Figure 18. Maximum deflections, at 5m, were 0.1mm, 0.15mm and 0.22mm for the three loads.

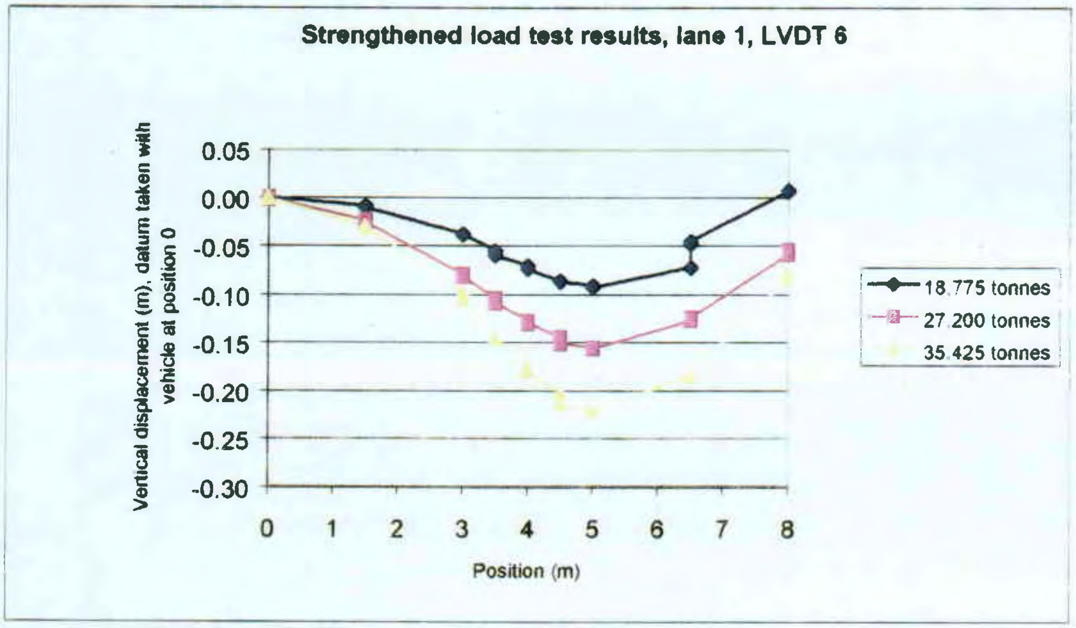

Figure 18 Longitudinal Influence Lines Measured After Strengthening

6. CALCULATED RESPONSES TO LOAD

Using the ELFEN program, as described in Section 3, responses of the bridge were calculated for the unstrengthened and strengthened cases. These are shown as longitudinal influence lines of deflection in Figure 19 and 20.

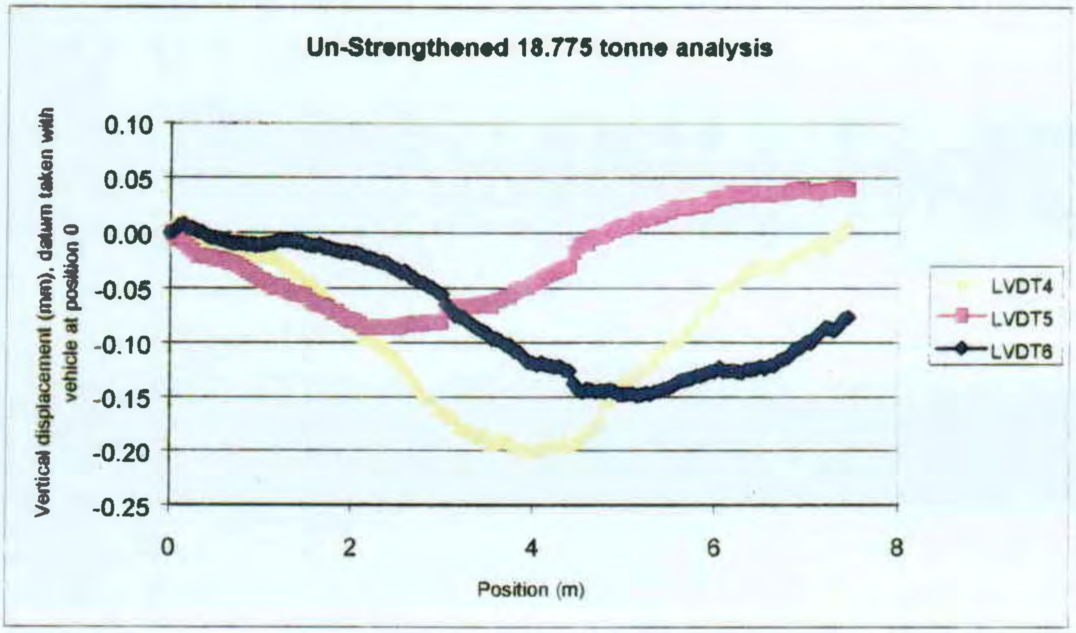

Figure 19 Longitudinal Influence Line Calculated Before Strengthening

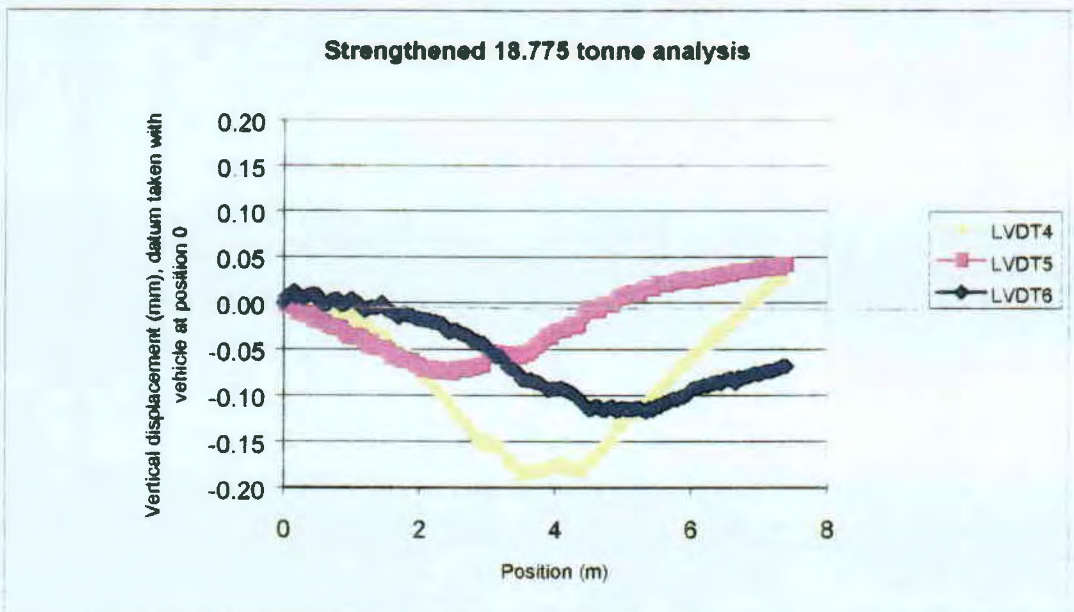

Figure 20 Longitudinal Influence Line Calculated After Strengthening

6.1 18 Tonne Load Test

The influence lines calculated for the $^{1}/_{4}$-points and centre span accurately represented the behaviour and deflections measured in the loading tests. Maximum values of calculated deflection are 0.20mm for the unstrengthened bridge and 0.18mm after strengthening. Maximum values for the different cases are summarised in Table 1.

Table 1 Summarised Deflections at 18 Tonne Load

	Deflections, mm						Strength Factors
	Quarter -point LVDT 5		Centre LVDT 4		Quarter-point LVDT 6		
	Measured	Calculated	Measured	Calculated	Measured	Calculated	
Unstrengthened	0.06	0.08	0.12	0.20	0.10	0.15	
Strengthened	0.03	0.07	0.10	0.18	0.09	0.12	
Average stiffening factor: Measured Calculated							1.4 1.2
Hidden 'performance' factor: Unstrengthened Strengthened							1.5 1.8

The measured deflections were very low, maximum values for the different influence lines varying between 0.06mm and 0.12mm before stiffening. This is not unexpected as arches are intrinsically stiff structures. Arches are rarely load tested partly because their structural action has only recently become calculable and partly because the small deflections demand very accurate instrumentation which is not always available. In one of the few comparable studies Gifford load tested Welland Flood Arch(5), a 7.04m span arch bridge on the A1trunk road. When loaded by a 32 tonne truck having an 18.5 tonne back axle, deflection at centre span was 0.36mm.

In Table 1, two average 'performance factors' have been calculated, defined in the following sections.

Stiffening factor is the ratio of unstrengthened to strengthened deflections. It follows that if the strengthening has been effective, the reinforcing anchors will produce a measurable reduction of deflection under load, ie an increased stiffness. From the data shown in Table 1, averaged stiffening factors have been calculated from the measured and calculated deflections. The factors are 1.4 and 1.2 respectively.

Hidden 'performance' factor is the ratio of calculated to measured deflections. The term 'hidden strength' is commonly used in relation to structural actions that, for one reason or another, cannot be taken into account when assessing load carrying capacity. This hidden strength commonly causes assessed strength to be less than the true strength and likewise for performance (deflections and strains). From the data in Table 1, averaged hidden 'performance' factors have been calculated from responses of the unstrengthened and strengthened bridge. The factors are 1.5 and 1.8 respectively. In the case of masonry arches it is considered that some of the hidden performance is due to there being improved transverse distribution.

6.2 26 Tonne and 34 Tonne Load Tests

The calculated influence lines for the higher loads are shown in Figure 21. As for the 18 tonne test, the calculated influence lines accurately represented the behaviour and deflections measured in the loading tests.

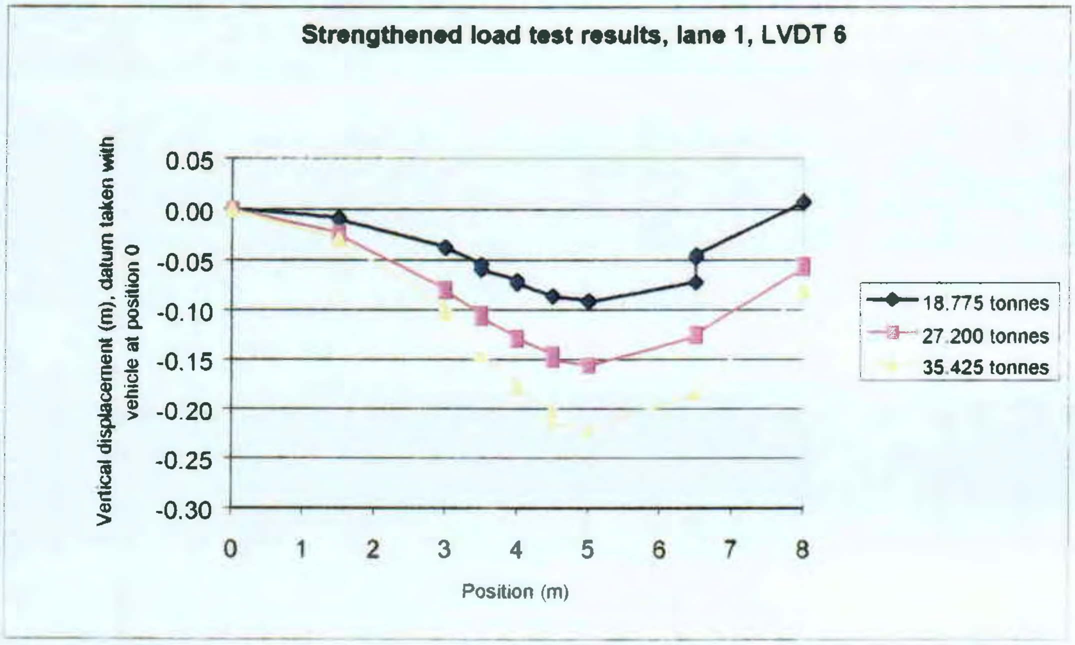

Figure 21 Longitudinal Influence Lines Calculated After Strengthening

Maximum deflections at the 6m quarter-point (LVDT6) are compared with the measured values in Table 2.

Table 2 Summarised Deflections at 18, 26 and 34 Tonne

Nominal Load, tonne	18	26	34
Measured load, tonne	18.775	27.200	35.425
Measured deflection (LVDT6), mm	0.10	0.15	0.22
Calculated deflection, mm	0.12	0.18	0.25

The designed increase in ultimate capacity of the bridge was 60 per cent. This bears a sensible relationship to the stiffening factors which indicate increases in stiffness of 20 per cent calculated and 40 per cent measured. The designed action of the anchors is to strengthen against the formation of hinges near to the $^1/_4$-points. Increased stiffness, although a secondary issue, is a useful indication that the anchors are effective.

6.3 Smart Anchor

Measurements from the prototype Smart anchor (Appendix B) indicate that the J-bars attracted stress and acted compositely with the structure of the arch barrel. This is reassuring as it confirms that grouting of the anchor has successfully connected it to the brickwork.

7. CONCLUSIONS

7.1 Design

(i) A scheme to strengthen the masonry parapets of Baber Bridge to P2 containment was successfully designed, Appendix A. In the event it was not implemented.

(ii) Archtec strengthening was designed and installed to raise the load carrying capacity by 60 per cent. The strengthening and performance of the bridge were calculated using the ELFEN program.

7.2 Load Tests

(i) Supplementary load tests were carried out before strengthening using a semi-trailer having a twin-axle close spaced bogey loaded to 18 tonne. The maximum measured vertical deflection was 0.12mm at centre span.

(ii) Load tests were also carried out after strengthening. Under the 18 tonne axle load, the maximum deflection was 0.10mm at centre span, 20 per cent less than unstrengthened.

(iii) Calculated deflections under 18, 26 and 34 tonne nominal axle loads correlated very well with measured values. The calculated values were consistently higher than measured and responses were essentially linear.

(iv) Responses calculated using the ELFEN program correctly predicted the shapes of the influence lines and provided deflections that were consistently greater than the measured values. This is encouraging as it indicates a degree of conservatism in the analysis and design.

(v) Under the 18 tonne axle load, average stiffening factors were found to be 1.2 calculated and 1.4 measured. Average hidden 'performance' factors were 1.5 unstrengthened and 1.8 strengthened.

7.3 Smart Anchor

(i) A prototype Smart anchor was installed as part of the strengthening. The load tests confirmed that it attracted stress and acted compositely with the arch barrel.

8. REFERENCES

1. J Page, *A Guide to Repair and Strengthening Techniques for Brick and Stone Masonry Arch Bridges*. TRL Contractor Report 284, Crowthorne, 1996
2. S K Sumon and N Ricketts, *Strengthening of Masonry Arch Bridges*. Chapter in Arch Bridges. Publ Thomas Telford, London, 1995
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4. The Institution of Civil Engineer. *Guidelines for the Supplementary Load Testing of Bridges.* Publ Thomas Telford, 1998.
5. S Mehrkar-Asl. *Load Testing of Welland Flood Arch No. 2.* Unpublished Report, 1999.
6. Baber Bridge Hounslow A315. *Strengthening and Load Testing. Appendix C Test Data* 1999.

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APPENDIX A

DESIGN OF PARAPET STRENGTHENING

In the meeting held on 18 November 1999 at Hounslow it was agreed that Gifford and Partners would investigate strengthening the parapets to vehicle containment standards. On investigation it was found that this would have required largely rebuilding them, partly in order to achieve the required high strength but also because of the geometric requirements. The P6 standard requires a height of 1.5m which is well in excess of the present height. A further problem with the P6 standard was the requirement for progressively increasing strength. This aims to make the fixings stronger than the parapet and the structure supporting the parapet stronger than the fixings to ensure that failure of the parapet does not lead to major damage to the bridge. This would have required major strengthening works to the bridge itself which was not considered viable. It was therefore decided to consider strengthening to P2 standards.

A strengthening scheme was devised based on BD 52/93 requirements for in situ reinforced concrete P2 parapets. Because of the differences between existing masonry and new concrete construction, it was not possible to follow all the detailed requirements of the standard. The strengthening was, however, designed to achieve the strength specified by the standard. This was achieved by designing Cintec anchors to enable the masonry wall to act as a reinforced masonry wall. Because the wall is substantially thicker than a conventional concrete P2 parapet, it was possible to achieve the strength under reversed moments which the standard would require, without providing anchors on both faces. This was done by placing the anchors further from the tension face than normal, enabling them to resist moments in either direction. The design called for 16 mm high yield vertical ties at 1m centres and for two 12mm longitudinal ties.

It was not possible to prove that the connection between the spandrel walls and the rest of the bridge was strong enough to resist the forces imposed by the design impact with the parapet. Also, the design described above assumes that the parapets are fixed at a point reasonably close below the carriageway surface. If the parapet was fixed at a lower position, more vertical anchors would be required. It was therefore necessary to design additional transverse ties to hold the parapet in place. 16mm high yield ties at 2m centres provided the required strength.

In the event, it was decided not to proceed with parapet strengthening and the design work was therefore abortive.

APPENDIX B

SMART ANCHOR

The Baber bridge Smart Anchor was made up of two S25 stainless steel type 2 ribbed bar sections joined with a coupler as shown in Diagram 1 below. The sensors were placed on a 50mm section of the bar that had been turned to 24mm in diameter, on each bar this section started 200mm in from the non-coupled end of the bar. J-bars J122 and J112 were the two bars used to form the anchor to which the sensors were fitted.

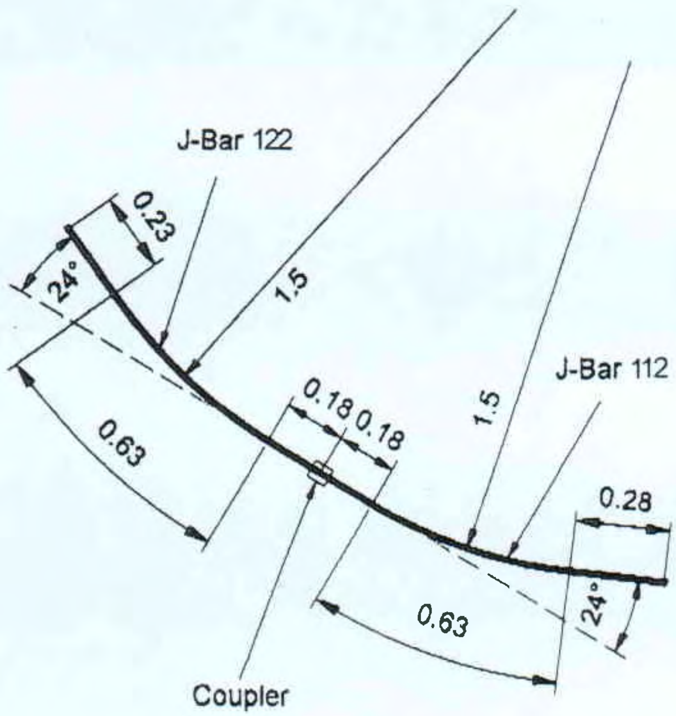

Diagram 1 The 'J' -bar Anchor on which the Sensors were Mounted

For the purpose of the test only one of the anchors was fitted with sensors, as this test was primarily designed to see if the anchor would respond in a real scenario, as opposed to an emphasis on the highly accurate measurement of structure movement. The position of the anchor within the bridge is shown in Diagram 2.

Sensor Positions

The positions of the sensors on the anchor are shown in Diagram 3 'Baber Bridge J-bar Anchor'. The sensors fitted are made up of four strain gauges arranged on the bar in a pattern designed to measure axial extension or compression.

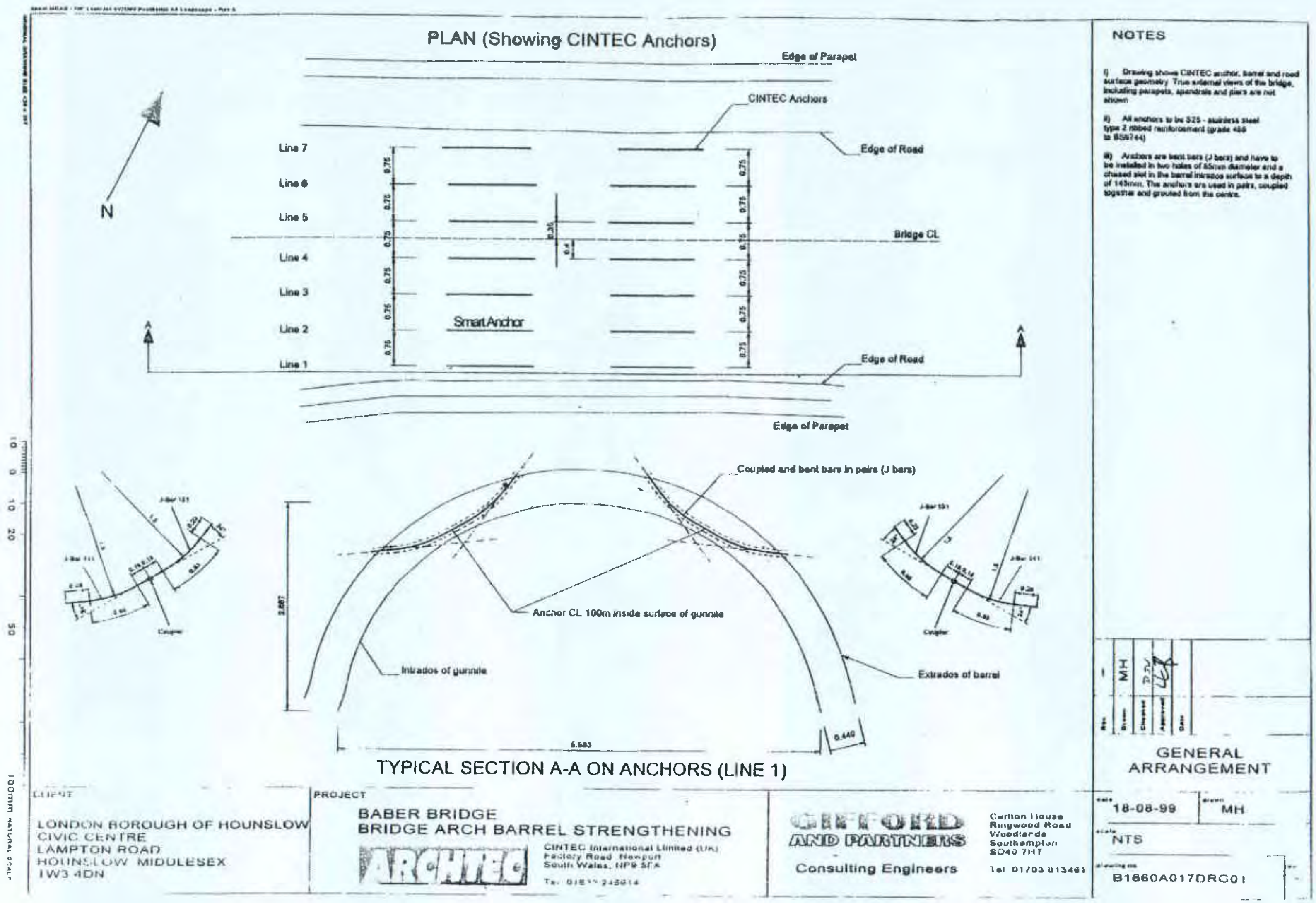

Diagram 2 Plan Showing the Position of the Anchor in the Bridge

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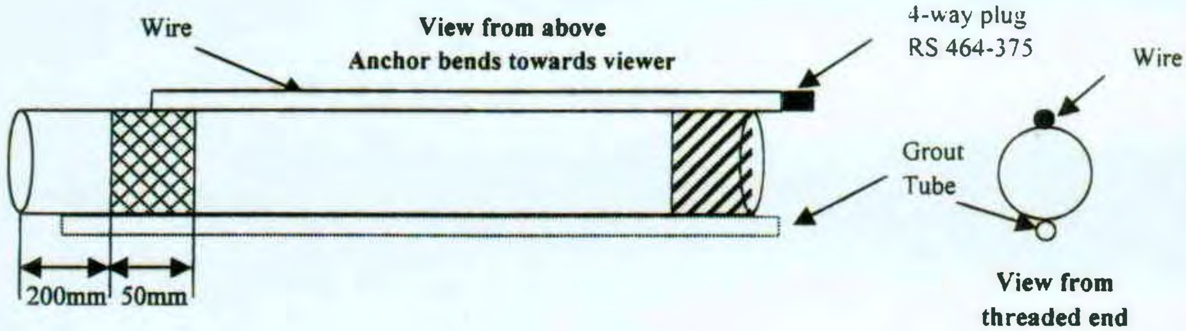
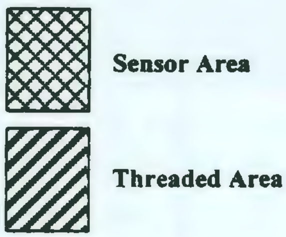
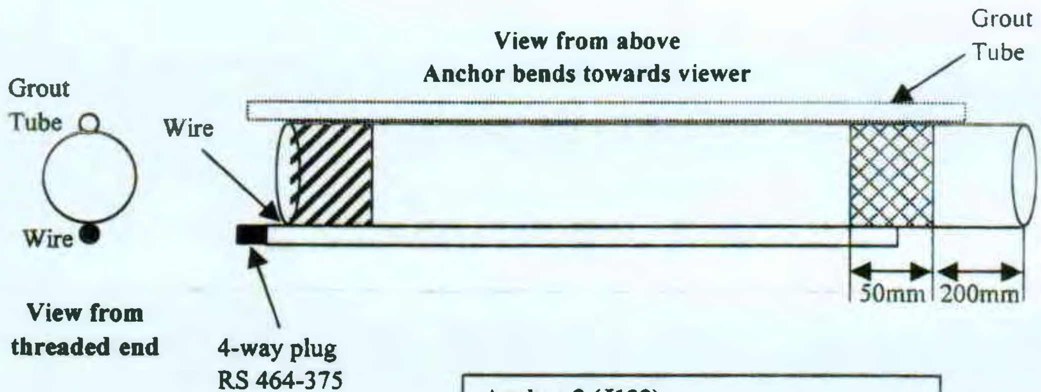


Anchor 1 (J112):
Length 1090mm
Wire Length 940mm of 4-way 7/0.2 cable
RS part 358-141





Anchor 2 (J122):
Length 1040mm
Wire Length 900mm of 4-way 7/0.2 cabl
RS part 358-141

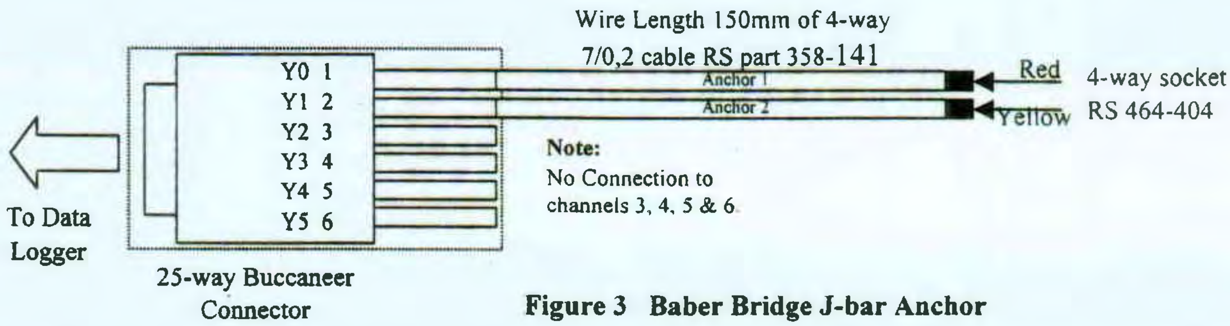
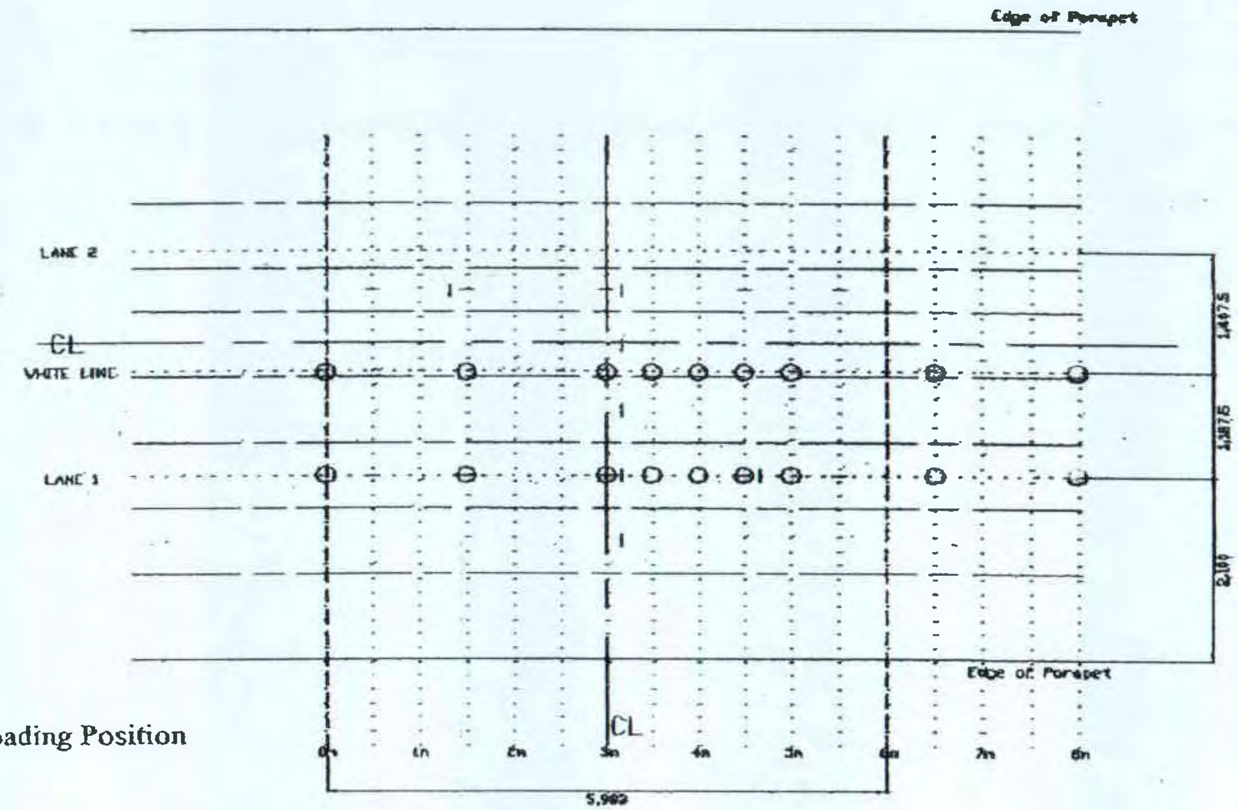


Figure 3 Baber Bridge J-bar Anchor



○ = Loading Position

Diagram 4 Loading Positions

Baber Bridge Test Results:

Bar J112	
Length	1.09m
Bridge 1:- Sensor Area	
Diameter	24mm
Length	50mm
Position	200mm in from the non-threaded end of the bar

Bar J122	
Length	1.05m
Bridge 2:- Sensor Area	
Diameter	24mm
Length	50mm
Position	200mm in from the non-threaded end of the bar

Initial Connection Test **Table 1**

Raw Result			**Normalised**	
Bridge 1	Bridge 2	Dummy	Bridge 1	Bridge 2
2153	2154	2152	2153	2154
2153	2154	2152	2153	2154
		Average	2153	2154

1st Run – Smallest Load

2nd Run – Middle Load

3rd Run – Largest Load

1st Run Lane 1 **Table 2**

Raw Result			**Normalised**	
Bridge 1	Bridge 2	Dummy	Bridge 1	Bridge 2
2154	2153	2153	2153	2152
2153	2154	2153	2152	2153
2153	2153	2152	2153	2153
		Average	2152.67	2152.67

1st Run White Line **Table 3**

Raw Result			**Normalised**	
Bridge 1	Bridge 2	Dummy	Bridge 1	Bridge 2
2156	2155	2153	2155	2154
2156	2155	2154	2154	2153
2156	2154	2156	2152	2150
		Average	2153.67	2152.33

2nd Run Lane 1 **Table 4**

Raw Result			**Normalised**	
Bridge 1	Bridge 2	Dummy	Bridge 1	Bridge 2
3170	3173	3189	2133	2136
3140	3169	3192	2100	2129
3175	3142	3198	2100	2130
		Average	2111	2131.67

2nd Run White Line **Table 5**

Raw Result			**Normalised**	
Bridge 1	Bridge 2	Dummy	Bridge 1	Bridge 2
3131	3144	3171	2112	2125
3180	3144	3199	2133	2097
3172	3171	3201	2123	2122
		Average	2122.67	2114.67

3rd Run Lane 1 **Table 6**

Raw Result			**Normalised**	
Bridge 1	Bridge 2	Dummy	Bridge 1	Bridge 2
3228	3223	3265	2115	2110
3245	3227	3241	2156	2138
3216	3243	3271	2097	2124
		Average	2122.67	2124

3rd Run White Line **Table 7**

Raw Result			**Normalised**	
Bridge 1	Bridge 2	Dummy	Bridge 1	Bridge 2
2153	2154	2152	2153	2154
2153	2155	2152	2154	2155
2153	2154	2152	2153	2154
		Average	2153	2154.33

Test Results Analysis

Bridge 1 **Lane 1**

	Average Result	µstrain
Test	2153	0
1st Run	2152.67	-10.745
2nd Run	2111	-1365.787
3rd Run	2122.67	-986.574

Bridge 2 **Lane 1**

	Average Result	µstrain
Test	2154	0
1st Run	2152.67	-43.293
2nd Run	2131.67	-726.491
3rd Run	2124	-975.848

Bridge 1 **White Line**

	Average Result	µstrain
Test	2153	0
1st Run	2153.67	21.810
2nd Run	2122.67	-986.574
3rd Run	2153	10.74

Δ

Bridge 2 **White Line**

	Average Result	µstrain
Test	2154	0
1st Run	2152.33	-54.360
2nd Run	2114.67	-1279.045
3rd Run	2154	10.74

Δ Note that these two results do not appear to be correct.

Conclusion

The first set of results (the Initial Connection Test) combined with some electrical resistance tests, showed that the sensors attached to the anchor had survived the process of installation. The measurements taken and the analysis performed show that the sensors have registered that the forces on the bars have changed as the load increased. This is shown by the change in the µstrain value given in the tables above. The fact that the output from the sensors has changed shows that in principle the Smart Anchor works, although more work is needed to make the readings meaningful. On this anchor the sensors installed are designed to measure axial extension or compression of the anchor. Due to the shape of the anchor and the angle at which it has been installed, the results will not give an accurate picture of what is happening to the anchor. To improve the results further sensors would be needed to give a better representation of the effect of the load on the anchor.

CINTEC ANCHOR TESTING

British Rail
Kennet Bridge Reading UK

BY:

OVE ARUP AND PARTNERS
(FEBRUARY 1988)

KENNET BRIDGE READINIG

KENNET BRIDGE, READING
Project Ref. 16435/04 Kennet Bridge, Reading.
BRITISH RAILWAY BOARD,
BRITISH RAIL WESTERN REGION ,
WESTERN HOUSE, 1 HOLBROOK WAY,
SWINDON, WILTS, SW1 1BY

OVE ARUP & PARTNERS,
CAMBRIAN BUILDINGS,
MOUNTSTUART SQUARE,
CARDIFF CF1 6QP.

PRELIMINARY LOAD TEST REPORT KENNET BRIDGE, READING FEBRUARY 1988

1.0 INTRODUCTION

Drawing **SK 03** shows the elevation and section of the trial holes and test holes for the Kennet Bridge, Reading. The trial holes, nos. 1 and 4 were used to determine the thickness of the wall construction together with the nature of the fill material for 2 metres beyond the front wall. The test holes, nos. 2, 3, and 21 were 76mm diameter holes to contain 25mm diameter HYS bars with a grout feed pipe (l0mm diameter) surrounded by grout, filling the drilled hole.

The results of the trial holes and test holes were as follows:

Hole#	Bwk	Clay	Rubble	Clay	Bwk	Total
			Length			
1	1.0m	2m	-	-	-	3m
2	1.5m	6.8m	-	-	1.2m	9.5m
3	1.0m	4.0m	0.9m	3.1m	1.0m	10m
4	1.0m	2.0m				3m
5	1.0m	3.0m	-	-	-	
6	1.0m	3.0m with chalk				

These holes were drilled by D.S.Wilcox using rotory coring under compression air.

Hole#	Bwk	Clay	Bwk	Clay
21	1.1m	7.5m	0.95m	0.25m

Hole#	Pre-drilled	Clay	Chalk	Bwk	Clay
1	3m	4.15m	-	0.05m	-
4	4m	1.8m	1.2m	1.30m	0.75m
5	5m	3.05m	-	1.30m	0.35m
6	4m	4.2m	-	0.6m	0.10m

These holes were drilled by Temple Hazell Associated using rotory percussive drilling in the hard masonry and rotory drag bit in the fill materials.

2.0 ANCHOR INSTALLATION

The two 25mm diameter anchors in a sleeve of 9.5m and 10.0m lengths were installed in the drill holes in one continuous length, but with a separation between the rear 5 metre length and the front length so that each could be grouted separately. For the test anchors, nos. 2 and 3, only the rear 5 metre length was to be grouted so that the anchorage provided by the buried wall could be ascertained. Unfortunately due to problems with the sock tearing in drill hole number 2, this anchor was grouted over its full length. However, the length embedded in the front wall will be drilled out or over-drilled so that the required test load can be applied. The over-drilling damaged anchor #2 and a replacement anchor #21 was installed.

3.0 TESTING

The test procedure is included in **Appendix A.** The basic procedure was applied only to anchor 3#, anchor 21# was test loaded under maximum load only.

Dial gauges were mounted against the end of the threaded anchor and of the end of the travelling rod of the test rig. The nett deflection is the difference between the two dial gauge readings.

Anchor Nos. 2 and 3 were to be tested loaded. However, because anchor 2 was inadvertently grouted to the front wall brickwork, test loading started with anchor 3. Although intended as a 5 tonne capacity anchor in the final solution, it was decided to test load anchor 2 to 9 tonne. Since it was the first anchor test loaded it was considered prudent initially to remove the deflection gauges at 30kN test load.

After the initial pre-loading, the loading was applied in 5kN increments to 30kN. Problems with the gauge slipping on the jack led to abandonment of this series of readings at 30kN and the load was removed from the anchor. For completeness, the deflection readings obtained are given in **Table 1.0.** After adjustment of the dial gauges and the test rig, a pre-load was again applied as previously. Deflection readings were taken up to 30kN and as the load was decremented back to zero. Incremental loading was then applied in 5kN increments and deflection readings recorded up to 55kN, when the gauges were removed. Loading then continued up to 70kN, when the travelling arm reached the limit of its extension, see **Table 1.0** at the end of the text and **SK03.**

The load was removed from the anchor and steel wedges inserted against the face of the brickwork so as to provide increased travel for the jack. Pre-load was applied and removed as previously and load was then applied to the anchor in increments of l0kN up to 90kN. The load was applied up to 98kN and held at 95kN for some minutes before being left at 90kN for the twenty four test.

As a rough check on the dial gauge readings, the extension of the threaded anchor from the face of the wall was measured by metal tape in the zero load position and after 90kN of load had been applied. The nett elongation based on these measurements was 7mm. However, the scope for error in the readings is large because of the uneven surface of the brickwork and the difficulty in reading the tape accurately for small readings. The check value compares with a pro-rata reading from the dial gauges of 3. 9mm. If the elongation was based on the 25mm steel bar alone (ignoring the grout and any friction with the clay), the free length would give an elongation of 4.9mm.

On return to site on Wednesday 30th December, the test load had dropped to approximately 75kN. No oil leakage was immediately apparent in the vicinity of the jack but heavy rain may have washed it away. The load was returned to 90kN and held for 15 minutes without loss of load. The load was then released from the anchor.

Anchor #21 was test loaded to 105kn and the load held for 30 minutes. The test was then concluded.

4.0 CONCLUSIONS

To test and trial holes have shown that it is feasible to drill 80mm diameter holes through the clay fill material and through the far brickwork wall. At the lower level of the test holes, the stiff clay is more easily drilled than the clay in the higher holes 1 and 4. The latter clay proved exceedingly difficult to progress through and alternative techniques (e.g. rotary percussive drilling in the clay) should be considered for the actual anchor installations.

The test loading for anchor No. 3 satisfactorily established the elastic performance of the anchor under load up to 30kN. The load/deflection curve is essentially linear up to 50kN. A test load of at least 76kN was satisfactorily applied to this test anchor for a twenty four hour period; the load dropped from 90kN to 76kN over this period.

Problems with the grout injection occurred on the test anchor 2 and 3 could be attributed in part to the small diameter of grout feed pipe. It is recommended that the overall diameter of the drill hole be increased to at least 105mm to accommodate a larger diameter grout feed pipe. This diameter is the minimum now recommended by Cintec NV if sufficient cover is to be provided by the injected grout to protect the steel anchor.

Test anchor #21 was satisfactorily tested under a load of 105kn, the load being held for thirty minutes. It was not considered necessary to record the anchor elongation.

ANCHOR No. 3

APPLY PRE-LOAD TO 10kN in 5kN INCREMENTS
DECREMENT LOADING TO ZERO IN 5k N INCREMENTS

LOAD kN	1st INCREMENTAL LOADING DIAL GAUGE mm.			1st DECREMENTAL LOADING DIAL GAUGE mm.			INCREMENTAL LOADING TO TEST LOAD DIAL GAUGE. mm		
	G1	G2	NETT	G1	G2	NETT	G1	G2	NETT
0	0	0	0	0	0	0	0	0	0
5	0	0	0	-	-	-	0	0	0
10	0	0.17	0.17	0	0.58	0.58	0	0.18	0.18
15	0	0.53	0.53	-	-	-	0	0.50	0.50
20	0	0.80	0.80	0.12	1.17	1.05	0	0.81	0.81
25	0.08	1.07	0.99	-	-	-	0.08	1.08	1.00
30	0.20	1.35	1.15	0.20	1.35	1.15	0.20	1.39	1.19
35							0.30	1.69	1.39
40							0.42	2.08	1.66
45							0.55	2.55	2.00
50							0.71	3.10	2.39
55							0:84	3.25	2.41
60				GAUGES REMOVED					
65									
70				MAXIMUM TRAVEL REACHED					
75									
80									
85									
90									
95									
100									

G1 - Gauge on anchor
G2 - Gauge on travelling arm

TABLE 1.0 LOAD-DEFLECTION READINGS

APPENDIX A

TEST PROCEDURES FOR ANCHORS AT KENNET BRIDGE, READING FOR BRITISH RAIL WESTERN REGION

Anchor 1 - test load to 9 tonnes as per **dwg C35248**
Anchor 2 - test load to 5 tonnes as per **dwg C35248**

1. Attach jack to the test anchor

2. Apply pre-load of 10kN in 5kN increments to the test anchor, allowing 1 minute between increments.

3. After 1 minute at 10kN, decrement the load in 5kN decrements with 1 minute between decrements.

4. Set dial gauge to be used to measure elongation of test anchor to zero; accuracy of gauge to be 0.01mm or better.

5. Apply load in increments of not more than 5kN and hold for 1 minute.

KENNET BRIDGE, READING
Project Ref. 16435/04 Kennet Bridge, Reading.
BRITISH RAILWAY BOARD,
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SWINDON, WILTS, SW1 1BY

OVE ARUP & PARTNERS,
CAMBRIAN BUILDINGS,
MOUNTSTUART SQUARE,
CARDIFF CF1 6QP.

6. Read the dial gauge at each load increment.

7. Repeat steps 5 and 6 up to:
60 kN for anchor 1
30 kN for anchor 2

8. Hold load for 1 minute. .

9. Decrement load in 5kN decrements, holding the load for 1 minute at each decrement.

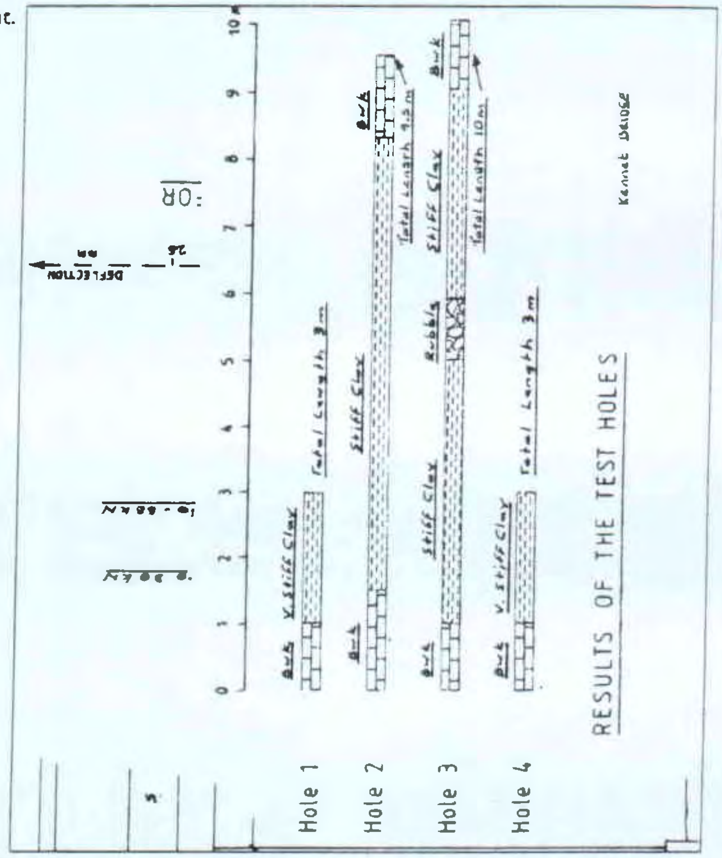
10. Record the dial gauge reading at each decrement.

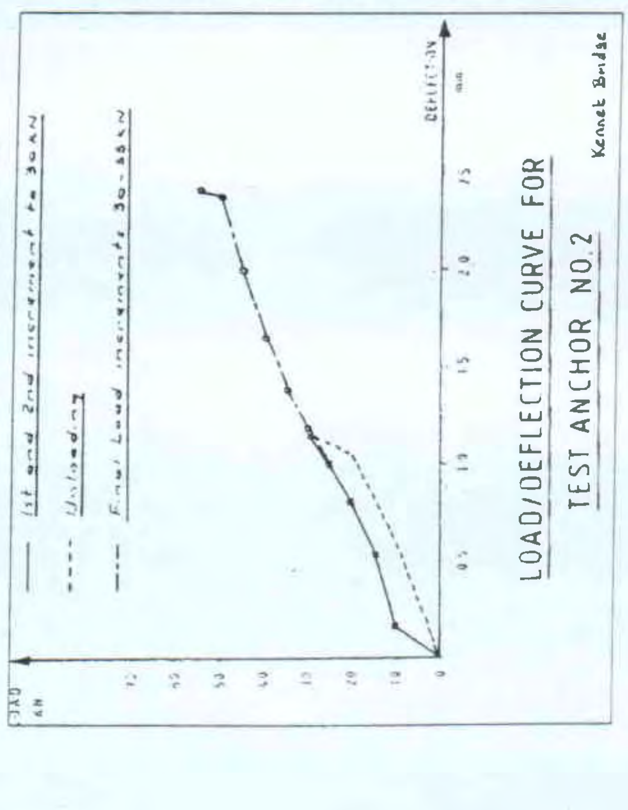
11. Repeat steps 5 and 6 up to:
90 kN for anchor 1, 50 kN for anchor 2
but removing the dial gauge at:
60 kN for anchor 1, 30 kN for anchor 2

12. Record any change in the adjacent brickwork during the loading, with particular note made of maximum failure load.

Care should be taken by means of visual observation and monitoring of the dial gauge readings to ensure that the wall does not fail prematurely and damage the instrumentation. If in doubt, the dial gauge may be removed after 50% of the test load is reached.

If anchor failure should occur, a full description with sketches of the failure mode should be made.





APPENDIX B

ANCHOR No. 3

APPLY PRE-LOAD TO 10kN in 5kN INCREMENTS
DECREMENT LOADING TO ZERO IN 5k N INCREMENTS

LOAD kN	1st INCREMENTAL LOADING DIAL GAUGE mm.			1st DECREMENTAL LOADING DIAL GAUGE mm.			INCREMENTAL LOADING TO TEST LOAD DIAL GAUGE. mm		
	G1	G2	NETT	G1	G2	NETT	G1	G2	NETT
0	0	0	0	0	0	0	0	0	0
5	0.12	0	0.12	-	-	-	0.10	0	-0.10
10	-	-	-	0.46	0.31	-0.15	0.25	0.14	-0.09
15	0.27	0.25	-0.02	0.60	0.73	0.13	0.39	0.45	0.06
20	0.38	0.60	0.22	0.70	1.14	0.44	0.51	0.78	0.27
25	0.50	0.98	0.48	0.76	1.29	0.53	???	1.08	0.50
30	0.62	1.29	0.67				0.70	1.35	0.65
35							0.82	1.20	0.38
40							Dial slippeed off		
45							during this test		

G1 - Gauge on anchor
G2 - Gauge on travelling arm

TABLE B.1 INITIAL LOAD-DEFLECTION READINGS

CINTEC ANCHOR TESTING

British Rail
Bridge 325 Abington UK

BY:
S B WOODHOUSE B.ENG (HONS) C.ENG.
M.I.STRUCT.E
(APRIL 1993)

Installation of Ground Anchors Through Railway Bridge Abutment

Cintec Ground Anchor Installation at bridge 325 Abington

INTRODUCTION:

Cintec International Ltd has developed a system of ground anchors incorporating the patented grout techniques utilised in the Cintec System of anchor fixings. The bridge section of the Civil Engineering Department of Intercity Railways, British Rail, permitted the installation of trail ground anchors through the abutments of bridge number 325 on the Edinburgh / Carlisle Railway line for testing.

GENERAL DESCRIPTION:

In general terms the anchors have the following features:

a) A high tensile steel bar (ribbed type 2) forming the central element and load transferral mechanism to the abutment wall.

b) The reinforcement bar has been epoxy coated to provide the first layer of corrosion resistance in accordance with British Standard for Ground Anchors BS8081: 1989.

c) The corrugated sleeve of UPVC forms the second barrier against moisture and therefore corrosion resistance. The corrugations form a shear key to permit the transfer of forces from the ground to the central bar and then back to the structure.

d) The elements in a,b and c above are within a polyester fabric sock which expands to contain the pressurised grout, the sock becomes formed to the shape of the cored or drilled hole. Plastic centralisers are used to ensure the correct positioning of the corrugation relative to the bar. Drawings and sketches are attached showing details.

e) The grout forms the interlocking mechanism between the steel bar and the grout interface. The grout is a patented formulation developed specifically for anchor applications, it is delivered under pressure and is designed to obtain compressive strength capabilities of between 40 – 50 N/mm^2. Shrinkage is avoided by the use of additives premixed with the grout. The grout itself, being cementitious provides a highly alkaline protective environment against potential corrosion of the steel and the passage of moisture in the unstressed areas.

f) The sock arrangement used in the trial anchors has features such that the remote end (that which is in contact with the soil) can be inflated independently of the near sock (that which is contact with the structure). With this arrangement the remote end was tested in order to establish the load capabilities. After testing the outer sock was inflated to form the bond with the abutment structure.

g) Relatively low steel stresses were involved in the anchor testing to eliminate unnecessary elastic extension and subsequential relaxation losses may be neglected.

h) The outer sock forms a secure bond with the abutment structure thus avoiding the need for unsightly anchor heads visible on the outside.

i) Each stage of the inflation process is monitored by a 'check sock', that is a small sock that inflates at the external end of the anchor indicating that the remote or unseen sock is fully inflated.

The anchor component parts and design with regard to corrosion resistance comply with the requirements of BS8081: 1989 the British Standard for Ground Anchorage for Permanent Anchors.

INSTALLATION:

From a scaffolded access platform, a mining barrel was used to core the hole through the abutment structure and into the embankment behind. The anchors were inclined at 20° to the horizontal beneath the bridge structure, and at 30° to the horizontal at wing wall locations. The anchors were inserted into the preformed holes and the two sections of the inner sock inflated. The grout is inserted at pressure from a pressurised container (89 PSI, 0.61 N/mm^2) The outer sock was not inflated in order that each of the anchors could be subsequently be test loaded.

Sufficient time was permitted for the cementitious grout to cure before any load testing operations were carried out.

GROUND CONDITIONS:

The abutments are located either side of a vehicular access route through the railway embankment. The embankment was built approximately 100 years ago from nearby materials and consisted of gravel, sands with clay and silt. Given the soil profile found, the behaviour of the anchors would inevitably be unpredictable and large resultant test loadings were not anticipated.

TESTING:

The testing was carried out using a hydraulic jack with a calibrated dial gauge measuring the tensile load applied in tonnes. Each of the anchors was tested with the resulting loads tabulated in the following tables. The loads were applied in 4 tonne increments with a minimum of 10 minutes between each rise in the load. Several of the anchors were left for extended periods at the higher loads which coincided with the limit of the testing equipment. One anchor number 2 with the load applied overnight to see if any slippage had occurred. A small relaxation was apparent, although it could not be established if this was due to anchor creep or the testing apparatus deflecting.
The location of anchors is indicated in drawing C2162/Sk 1.

The results obtained were of larger magnitude than could have been anticipated given the actual ground conditions. In general the loads obtained varied between 13 – 20 tonnes. The bond stress or cohesion at the soil / interface has been calculated to vary between 81.3 and 219.7 KN/m^2. Anchor number 1 has an unusually low value of 93.8 KN/m^2, however this particular hole was left exposed for some considerable time after the mining barrel was removed before the anchors were fitted due to an equipment malfunction which may have led to some localised collapse of the substrate. Anchor number 5 also has an unusually low bond stress of 81.3 KN/m^2, this anchor was inserted into the area of the sloping embankment, which would not have had the benefit of the loading consolidation as the area underneath the railway tracks. The remaining results varied between 140.6 to a maximum of 219.7 KN/m^2 which reflects the variable nature of the substrate.

As the sock is inflated under pressure with grout, it expands to fill the shape of the hole, thus filling any irregularities in shape and size. A combination of different factors is anticipated to develop the load capacities obtained as follows.

1) Forming an irregular wedge by the shape of the hole and sock inflation, thus creating the need to shear the soil in order for the anchor to fail.
2) The grout 'milk' extrudes through the sock and partially bonds to the surrounding granular material, thus enlarging the effective diameter of the anchor.
3) Localised compaction of the surrounding material due to the pressurised grout inflation.

The installation and testing was witnessed by:

Mr Kader of British Rail Intercity Civil Engineering Dept.
Mr Barnet of British Rail Intercity Civil Engineering Dept
Mr Dimmick of Cavity Lock Systems (now Cintec International)
Mr Parry of Cavity Lock Systems (now Cintec International).
Mr Woodhouse of Fordham: Johns Partnership.

The anchors were installed in the period February – May 1992 and tested between June 1992 and December 1992.

DESIGN OF ANCHORS:

The following outlines the basic principals involved in assessing the design parameters and considerations in relation to the capacity of the ground anchors.

STEEL TENDON

The steel tendon in the anchors tested comprised of a high tensile steel bar, (epoxy coated for protection).

The bar area was established by the formula: $$\text{Area required} = \frac{\text{Load}}{\text{Fy}}$$

Where:- Load = working load multiplied by an appropriate factor of safety (200Kn)
Fy = characteristic strength of the steel (460 N/mm^2).

For the test anchors, the area required = $$\frac{200\times10^3}{460} = 434.8\ \text{mm}^2$$

Bar diameter 40mm provides area of 1256 mm^2, F.O.S. = 2.88
Bar diameter 32mm provides area of 804 mm^2, F.O.S. = 1.85

The steel stresses in this case were maintained at the low levels shown in order to avoid significant elastic extensions and therefore potential relaxation losses.
The steel bar utilised in the tests was a high yield ribbed bar (type 2) which has raised ribs on the surface for increased bond capability.

The bond between the grout and the bar can be established from the equation:-

$Fbu = B\sqrt{fcu}$ where fbu = the design ultimate anchorage bond stress.

$Fbu = 0.7\sqrt{40}$ B = coefficient dependent on type (0.5 x 1.4 = 0.7)

= 4.43 N/mm² fcu = compressive strength of grout (40 N/mm²)

DESIGN OF FIXED ANCHOR LENGTH:

The pull out capacity of the test anchors can be shown as:- $Tf = \pi DLS$

Where S = the shear, bond and skin friction at Substrate/rock interface (Kn/mm²)

D = diameter of fixed anchor (m)

L = Length of fixed anchor (m)

Tf = pull out capacity in (Kn)

The values of S varied between 81.3 to 219.7 Kn/m². For design purposes the lowest value should be used and a factor of safety of 4 utilised to limit ground creep in permanent anchors.

For design of anchors at specific locations the nature and behaviour of the substrate must be established by testing. Full-scale load tests are recommended to confirm laboratory results.

FIXED ANCHOR DESIGN IN ROCK

$$Tf = \frac{\pi D L\, Tult}{\text{Factor of Safety}}$$

Where Tult = the ultimate bond or skin friction at sock / rock interface

The value of Tult will vary dependant on rock type, condition and discontinuities. A minimum fixed anchor length of 3m is recommended to account for local variations and a factor of safety of 3 to 4 be applied dependent upon the circumstances of usage.

FIXED ANCHOR DESIGN IN COHESIONLESS SOILS

The substrate at the testing location falls into this category although clay and silts were present.

$$Tf = \frac{\pi DLS}{\text{Factor of Safety}}$$

The value of S must be found by testing. A factor of safety of 4 should be used and a minimum length of 4m is recommended.

FIXED ANCHOR DESIGN IN COHESIVE SOILS

$$Tf = \frac{\pi D L \alpha Cu}{\text{Factor of Safety}}$$

Where α • •adhesion factor 0.3 – 0.45 verified by testing.
Cu = average undrained shear strength of substrate.

The value •and Cu must be found by laboratory tests or full-scale tests. The factor of safety should be of the order of 3 to 4 and a minimum length of 3m is recommended dependent upon consistency.

ANCHOR BOND TO STRUCTURE

Should the anchor be required to bond to the structure (as opposed to an anchor head arrangement) the following equation may be used:-

$$Ts = \frac{\pi DLB}{\text{Factor of Safety}}$$

Where Ts = ultimate bond to the structure material (Kn)

B = bond between sock and structure (Kn/m ²)

The value of B will vary dependent upon material, values of 600Kn/m² are reasonable (subject to testing) for solid concrete or masonry.

DISCUSSION

The general conditions at each location will dictate the design stresses to be used in assessing the ultimate capacity of an individual anchor. Where laboratory tests are not available, full-scale insitu tests are required to establish the lower bounds of the substrate capacity.

A minimum fixed anchor length of three metres is recommended to account for local variables in substrate conditions.

In order to reduce the possibility of long term ground creep, factors of safety should be applied. These factors should be of the order of 3 to 4 dependent on soil consistency, life expectancy and their importance to the structure.

The fixed anchor length must be located beyond the critical zone, such as the wedge failure, slip circle, rock discontinuities in order to be effective. The free anchor length will depend upon the geometry of the location.

The anchors can act as a restraint, only accepting load if movement occurs, or they can be pre-stressed to a set load to provide an active force.

A feature of the Cintec System is that a choice of connections can be achieved with regard to fixing to structure. Traditional anchor head details may be used where periodic re-stressing or monitoring is required. Where the structure is suitable, the anchor may be bonded to the material as a permanent fixing, without the requirement for surface apparatus.

GENERAL DESIGN CONSIDERATIONS

Where ground anchors are being utilised, careful consideration should be given by the designer to the following points:-

a) Detailed field and laboratory tests to establish soil characteristics.
b) Full-scale load tests to confirm laboratory predictions.
c) Assessment of consequences of potential long-term creep.
d) Overall length of anchor, fixed anchor length, failure planes.
e) Effects of anchor groups if anchors closely spaced
f) Likely stress losses due to tendon relaxation.
g) The free anchor length can be released from the grout by use of smooth tubes forming the second barrier of corrosion resistance, thus avoiding stressing ground close to structure.
h) The factor of safety to be applied.
i) Reference should be applied to the British Standard BS.8081 1989 or other appropriate document for advice on usage and design.

CONCLUSION

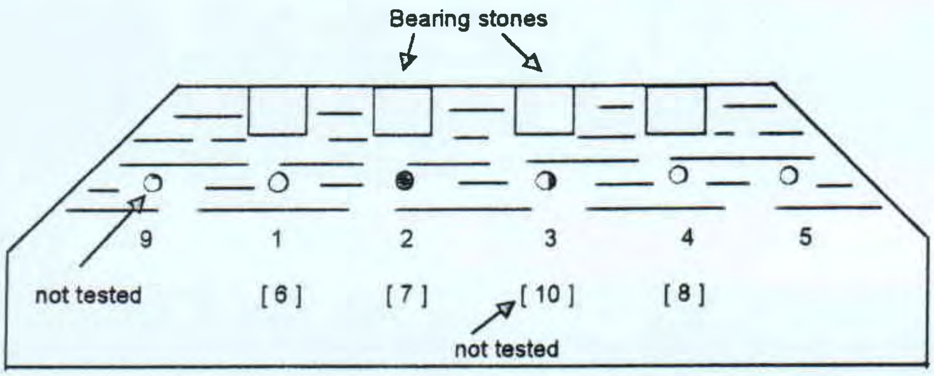
The testing of the ground anchors showed that the Cintec System could be successfully used in even the most difficult of ground conditions and achieve results in excess of expectations.

Careful appraisal of all factors must be given by the designer, to the points raised in the design considerations section, in order to fully realize the potential of the system

S. WOODHOUSE B. Eng (Hons) C.Eng M.I.Struct.E.

23rd APRIL 1993

Date: April 1993 Scale: / Drawing No: C2162/Sk 1
Drawn: J.S. Design S.W. Project: BRIDGE 325, ABINGDON
Drawing Title: **GROUND ANCHOR DETAIL TO ABUTMENTS**



ELEVATION OF NORTH & SOUTH ABUTMENT SHOWING GROUND ANCHORS
SOUTHERN ANCHORS 1 - 5
NORTHERN ANCHORS 6 - 8

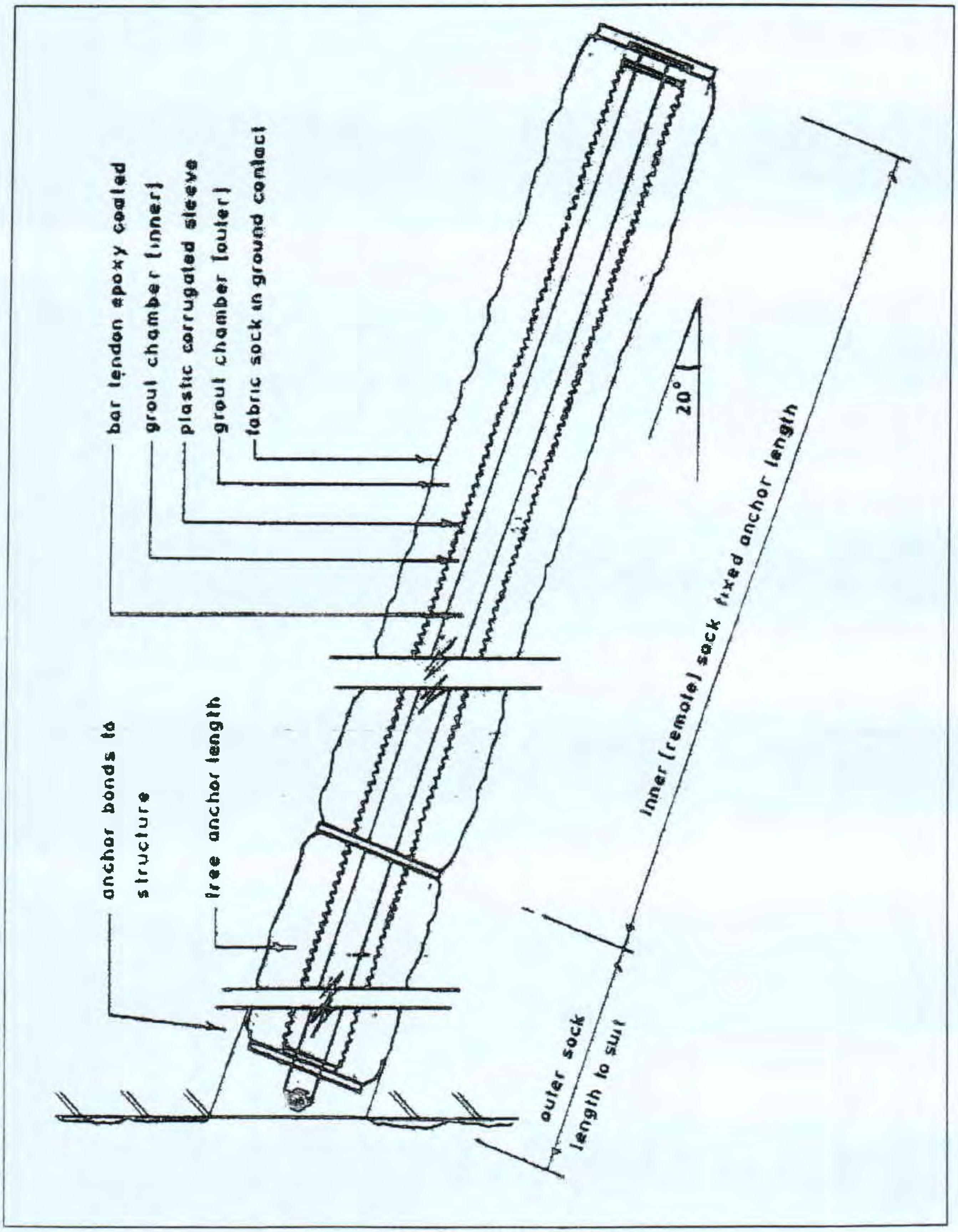
ANCHOR NUMBER	ANGLE OF INCLINATION	TOTAL LENGTH (M)	FIXED ANCHOR LENGTH OR LENGTH OF EMBEDMENT (M)	HOLE DIAMETER (MM)	TEST LOAD [T]
1	20°	5.45	4.1	124	15
2	20°	3.95	2.6	124	18
3	20°	3.45	2.1	124	18
4	20°	3.95	2.6	124	19
5	30°	5.45	4.1	124	13
6	20°	4.45	3.1	124	18
7	20°	4.45	3.1	124	17
8	20°	4.95	3.6	124	20

Date: April 1993 Scale: / Drawing No: C2162/Sk 3
Drawn: J.S. Design S.W. Project: BRIDGE 325, ABINGDON
Drawing Title: **GROUND ANCHOR TEST RESULTS**

Anchor number	Angle of inclination	Total Length (m)	Fixed anchor length or length of embedment (m)	Hole diameter (mm)	Soil anchor Interface (mm ²)	Test Load (T)	Test Load (KN)	Shear stress Soil / anchor Interface (N/mm²)	Shear stress soil anchor interface (KN/m²)
1	20°	5.45	4.1	124	1.599×10^6	15	150	0.0938	93.8
2	20°	3.95	2.6	124	1.014×10^6	18	180	0.1775	177.5
3	20°	3.45	2.1	124	0.819×10^6	18	180	0.2197	219.7
4	20°	3.95	2.6	124	1.014×10^6	19	190	0.1873	187.3
5	30°	5.45	4.1	124	1.599×10^6	13	130	0.0813	81.3
6	20°	4.45	3.1	124	1.209×10^6	18	180	0.1488	148.8
7	20°	4.45	3.1	124	1.209×10^6	17	170	0.1406	140.6
8	20°	4.95	3.6	124	1.404×10^6	20	200	0.1424	142.4

Date:	April 1993	Scale:	/	Drawing No:	C2162/Sk 2		
Drawn:	J.S.	Design S.W.		Project:	BRIDGE 325, ABINGDON		

Drawing Title: **GROUND ANCHOR DETAILS**



CINTEC ANCHOR TESTING

British Rail
Viaduct No 4 Birdmill Viaduct UK

BY:
BRITISH RAIL ENGINEER G BARNETT

Cavity Lock Systems Limited - Test Data Reports

Project :Viaduvt No 4 :- BirdmillViaduct,
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Assistant Bridge Engineer for
British Rail,Waverley Station,
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REPORT ON USE OF CINTECANCHORS

Job Location

The location of the job where the Cintec Anchors were used was at Viaduct No. 4, Birdsmill Viaduct, on the Edinburgh to Bathgate Railway between Newbridge Junction and Uphall.

Description of Fault

The spandrel wall above pier No. 4 from the Bathgate (Uphill) end of the viaduct is bulging seriously over an area between arches numbers 4 and 5 below stringer course of the parapet on the up side of the viaduct.

Solution

A 3m deep trial core hole was drilled through the spandrel above pier 5 to ascertain the exact construction of the viaduct. The result obtained showed that the spandrel outer wall was approximately 1m deep and behind this wall was a series of voids between dwarf internal walls which followed the line of the arch. Cavity Lock Systems took away samples from the core and they asked Ove Arrup to test them and design a suitable anchor to retain the spandrel by tying the anchor through the dwarf walls. The resulting design was a 3m long anchor constructed of square hollow sections and a grout retaining sock.

Installation

15 Number anchors were supplied and Holemasters (Livingston) were employed to drill the holes on a pattern supplied by Ove Arrup i.e. a 600mm grid pattern. The anchors were installed and grouted up by the staff from Cavity Lock Systems over a period of 4 days. The whole operation was carried out very successfully and proficiently by the Cavity Lock Systems staff and I was very surprised at the simplicity of the operation. Following one unsuccessful test carried out on one of the anchors where the end plate supplied failed due to the wrong size of plate, a second test was carried out up to a pull out of 80Kn which was approximately 18% above what was required and no visible damage was ascertained.

Conclusion

My personal opinion of the Cintec system can only be a favourable one and these anchors could be successful in a wide range of applications in the construction and engineering industries.

G Barnett
Assistant Bridge Engineer
British Rail
Waverley Station
Edinburgh

CINTEC ANCHOR TESTING

British Rail
Westminster Tunnel Liverpool UK

BY:
PETER G GRIFFIN B.SC M.I.STRUCT ENG
(JUNE 1994)

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Report: No : 01:A1:
21st June 1994

WESTMINSTER TUNNLEL, LIVERPOOL
TRACK LOWERING
GROUNDANCHORSTO RESTRAINTUNNEL
BRICK LINING WALLS
TEST OF PROPOSED GROUNDANCHOR FOR
BRITISH RAIL INFRASTRUCTURE SERVICES
CINTEC HARKE GROUNDANCHOR
Supplied, manufactured, and installed by Cavity Lock Systems Limited.

TEST REPORT :Testing of a ground anchor in the side wall ofWestminster tunnel on 19th June 1994.

INTRODUCTION :The anchor had been installed on 12th June 1994, and was in accordance with the parameters set out in the soils report issued by Infrastructure Services for tender purposes.The materials, and installation process were in accordance with the manufacturers requirements.

Minor deviations were noted.The 3m fixed inner lennth of the anchor, was, due to the thickness of the wall approximately 800mm behind the brickwork, 1m specified.

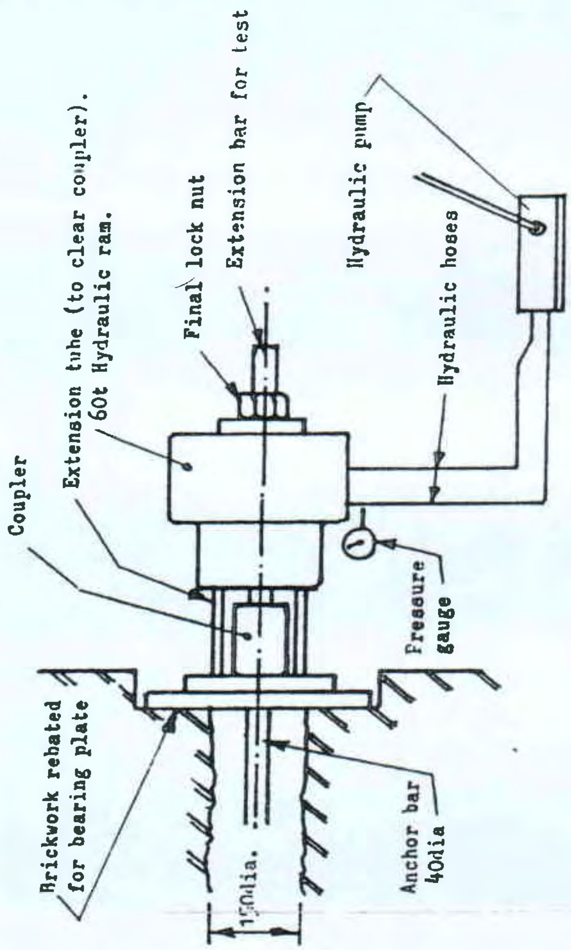
The diameter of the anchor was increased to 150mm from the 125mm specified, for practical reasons.

The anchor bar used, macalloy 500 Re-bar, 40mnn nominal diameter, having a failure load of 691kN, and a yield load of 628kN.The area of the bar, 1256mm, with a modulus of elasticity of $205^{kN}/mm^2$

The Presstec Grout used to inflate the inner sock of the fixed anchor length, has an ultimate crushing strength of $40^{n}mm^2$.

The hydraulic ram used for the test had a capacity of 600kN , related to a hydraulic pressure of 10,000 p.s.i.The 50kN increments were based on this pressure\load relationship

TESTARRANGEMENT :



The test equipment was set up as shown in diagrammatic sketch above.At this stage a small load was applied to the anchor to stabilise the equipnnent. The dial gauge was fixed in position via a bracket attaclned to the wall, to measure movement off a square bar welded to the final locknut.

Reqd load kN.	Time interval mins	Ram pressure p.s.i. Start	End	Extension Gauge mm. start	end	Ram extension mm.
50	10	1000	900	0.60	0.59	
100	10	1650	1550	0.60	0.595	
150	10	2500	2400	1.35	1.325	4.5
200	10	3330	3200	1.65	1.615	5.0
250	10	4300	4200	0.20	0.225	
300	10	5200	5100	0.225	0.23	
350	10	5900	5850	0.225	0.235	
400	10	6700	6700	0.235	0.24	
450	30	7600	7400	0.242	0.255	7.0

Dial gauge re-set.

Note:The dial gauge readings following re-set

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show very little variation. Measurement of the ram extension shows 2.0mm. This indicates that dial gauge was for some reason not functioning correctly.
The free length of bar from the front of the fixed 3m anchor sock is 2m approximately.
The theoretical extensions on this length are:

For full test load of 450kN	= 450x1000x2000/256x205x1000
	=3.50mm
For increment of 50kN	=0.39mm
Total ram extension measured	=7.00mm
Ram extension at 150kN	=4.50mm
Ram extension over 300kN	=2.50mm
Theoretical extension 300kN	=2.33mm
Dial Gauge readings over	
150kN to 200kN. 1.650 - 1.350	=0.30mm

CONCLUSIONS :The anchor sustained the full 450kN test load satisfactorily with a loss of load over 30mins of 10kN approximately.The anchor sustained each load increment satisfactorily with a loss of load over 10mins of 5kN approximately, reducing as the loading increased. It appeared that the dial gauge was not functioning correctly, However, the check extensions taken measuring the projection of the ram confirm that the anchor performed satisfactorily.The test demonstrates clearly that up to and including the test load of 450kN, the anchor and the sandstone in which it is embedded performed very satisfactorily.

Peter G.Griffin BSc.C.Eng. M.I.Struct.E.

CINTEC ANCHOR TESTING

Proof Load Tests on Cintec Anchors for Signal
Gantry Supports
Fenchurch Street Station, London, UK

BY:

LAING GROUP TECHNOLOGY
(JUNE 1994)

Cavity Lock Systems Limited - Test Data Reports

Laing Technology Group Limited
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Client : WT Specialist Contracts
Unit 4 & 8, Arundel Mews, Brighton, BN2 1GD
Project: Proof load tests on anchors for signal gantry supports - Fenchurch Street Line.
Date of test : 8th, 10th, 15th & 17th June 1994

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Our ref:
RN/SJ
Date : 20 June 1994
Your ref :
Contract : 94160
Code : RT22030

Client : WT Specialist Contracts
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BN21GD

Project: Proof load tests on anchors for signal gantry supports - Fenchurch Street Line.

Date of test : 8th, 10th, 15th & 17th June 1994

Object of test : To proof load grout fixed anchors as follows:

1. 2no. anchors, installed at an angle of 30°, to a load of 130kN (1.5 x working load) in compression. At location UR 120.

2. 2no. anchors, installed at an angle of 20°, to a load of 140kN (1.5 x working load) in tension. At locations UR 112, 117 and 120.

3. 2no. anchors, installed normal to the wall, to a load of 25kN (1.5 x working load) in shear. At location UR 112, 117 and 120.

4. 2no. anchors, installed at an angle of 20°, to a load of 48kN (1½ x working load) in tension at location UR 516.

5. 3no. anchors, installed normal to the wall at the lower level, to a load of 75kN (1½ x working load) in shear at location UR516.

6. 1no. anchor, installed normal to the wall at the upper level, to a load of 120kN (1½ x working load) in shear at location UR516.

Method of Test:

1. Compression tests

Load was applied in increments of 10kN by means of a 300kN capacity hydraulic ram acting through a 1250mm long, 89mm O.D. tubular strut to the face of the anchor located 900mm into the viaduct wall. A reaction base for the ram was provided by a rolled steel column section inclined at 60° to the horizontal and restrained at its nominal mid span by tie bars to the tension anchors installed in the wall and a timber baulk and packing between its lower end and the wall, as shown in plates 1 & 2.

The applied load was monitored by a precision pressure gauge calibrated with the hydraulic ram against a proving ring (lab no. 2349) traceable to national standards.

Displacement was measured by two dial indicators mounted diametrically opposite each other on the tubular strut to measure axial movement of the strut relative to the viaduct wall. See plate 2.

Each increment of load was held for a brief period prior to reading the displacement gauges and the maximum proof load was maintained for 1 hour prior to unloading in five increments. The residual displacement was then recorded.

2. Tensile tests

Load was applied in increments of 10kN by means of 300kN capacity hollow hydraulic ram reacting on a bridge supported at 600mm centres and adapted to enable the force to be applied axially to the anchor bars installed at 20° to the viaduct wall. See plate 3.

The applied load was monitored by a precision pressure gauge calibrated with the ram against a proving ring (lab no. 2349) traceable to national standards.

Displacement was measured by a dial indicator mounted from the wall and arranged to measure the axial displacement of the anchor bar. Each increment of load was held until stable prior to reading the displacement and the maximum proof load was maintained for 1 hour prior to unloading.

3.

3.1 Shear tests (locations UR 120, 117 & 112)

The two shear anchors were tested simultaneously each providing a reaction base for the other. Shear plates 38mm thick were located on each anchor and fixed by screwing a nut on the anchor bar and lightly tightened using a normal spanner. See plate 4.

Load was applied in 5kN increment by means of

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Date of test : 8th, 10th, 15th & 17th June 1994

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a 120kN hydraulic ram positioned between the shear plates parallel to the viaduct wall. The load was measured using a precision pressure guage calibrated with the ram against a proving ring (lab no. 2348) traceable to national standards.

Displacement was measured by dial indicator mounted on the wall, one for each anchor assembly and arranged to measure movement of the shear plate in the direction of the applied force. Each increment of load was held until stable prior to reading the displacement and the maximum proof load was maintained for 1 hour prior to unloading.

3. 2 Shear tests (locations UR 516)

The 3 no. lower shear fixings were loaded using the method described in 3.1, but in increments of 10kN. The upper shear fixing was loaded using a frame consisting of a shear plate on the fixing, as previously described, with a beam over and tie bars loaded by a 300kN hyraulic ram reacting on the two outer shear fixings at the lower level. See plates 5 & 6. Load was applied in increments of 10kN. Load and displacement were measured as described in 3.1.

Results

The results are presented in tabular form -

Compression anchor No. 1 (first loading)	Table1
Compression anchor No. 1 (second loadng)	Table2
Compression anchor No. 2	Table3
Tension anchor No. 1 & 2 UR120	Table4
Tension anchor No. 1 & 2 UR117	Table5
Tension anchor No. 1 & 2 UR112	Table6
Tension anchor No. 1 & 2 UR516	Table7
Shear anchor No.1 & 2UR120	Table8
Shear anchor No.1 & 2UR117	Table9
Shear anchor No.1 & 2UR 112	Table10
Shear anchor No.1 & 2UR516	Table11
Shear anchor No.2 & 3UR516	Table12
Shear anchor Upper UR516	Table13

Results

Location UR 120 Anchor No. 1 Compression load test - 1st test loading

Date of test: 8th June 1994

Applied Load kN	Displacement mm	Time	Comments
0.0	0.0	15.50	
10.0	0.12		
20.0	0.12		
30.0	0.22		
40.0	0.41		
50.0	0.62		
60.0	1.01		
70.0	1.18		
80.0	1.41		
90.0	1.77		
100.0	2.09		
110.0	-		
120.0	2.19		
130.0	2.45	16.28	Hold for 1hr
130.0	3.01	17.10	
130.0	3.06	17.28	
100.0	2.95		
70.0	2.44		
40.0	2.23		
20.0	1.87		
0.0	0.97		

Recovery = 68%

Table 1

Results

Location UR 120 Anchor No. 1 Compression load test - 2nd test loading

Date of test: 8th June 1994

Applied Load kN	Displacement mm	Time	Comments
0.0	0.0	18.12	
10.0			
20.0			
30.0			
40.0			
50.0			
60.0			
70.0	1.08		
80.0			
90.0			
100.0			
110.0			
120.0			
130.0	1.83	18.26	Hold.
130.0	2.00	18.36	
130.0	2.03	18.50	
130.0	2.06	19.00	
130.0	2.07	19.10	
130.0	2.10	19.20	
130.0	2.12	19.30	
70.0	1.74		
0.0	-0.3		

Total Recovery

Table 2

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Date of test : 8th, 10th, 15th & 17th June 1994

Results

Location UR 120 Anchor No. 2 Compression load test

Date of test: 8th June 1994

Applied Load kN	Displacement mm	Time	Comments
0.0	0.0	20.55	
10.0	0.6		
20.0	0.25		
30.0	0.55		
40.0	1.17		
50.0	1.35		
60.0	1.55		
70.0	1.79		
80.0	2.05	)	Sudden realignment
90.0	2.02	)	of tubular strut.
100.0	2.41		
110.0	2.42		
120.0	2.43		
130.0	2.43	21.38	
			Hold.
130.0	2.44	22.08	
100.0	2.44		
70.0	2.41		
40.0	1.99		
20.0	1.48		
0.0	0.12		

Recovery = 95%

Table 3

Commentary of Compression Tests

During the loading sequence, the tubular strut tended to re-align its axis relative to the set angle. In order to restrain this movement, timber packing was inserted in the hole around the tube. See plate 2. However, vertical movement in the order of 7mm was recorded at the jack end of the tube and there was some lateral movement. This represents a very small angular change in the direction of loading, in the order of 0.3° and would have no significant effect on the magnitude and direction of the applied load. It would however affect the recorded values of displacement since the movement gauges were attached to the tubular strut, parallel to its axis - see plate 2, ie at 30° to the viaduct wall relative to which movement was measured. Consequently the vertical movement refered to the above would result in the gauges travelling up a 30° ramp and recording movement additional to the actual displacement of the anchor occured. anchoranchor.It is the writters considered opinion that the movements recorded were due largely to the above effect plus compressive strain the the 1250mm long tubular strut and therefore no significant displacement of the anchor occurred.

Results

Location UR 120
Tensile load tests
Date of test: 10th June 1994

Applied Load kN	Anchor 1	Anchor 2	Time Anchor 1	Time Anchor 2
0.0	0.0	0.0	9.35	11.07
20.0	0.02	0.01		
40.0	0.03	0.03		
60.0	0.04	0.04		
80.0	0.05	0.06		
100.0	0.08	0.08		
120.0	0.11	0.10		
140.0	0.12	0.13	9.45	11.12
140.0	0.12	0.13	10.45	12.12
0.0	0.0	0.0	10.46	12.12

Recorded movement represents elastic strain in the projecting anchor rod. No movement of the anchor or signs of damage to surrounding brickwork.

Table 4.

Results

Location UR 117
Tensile load tests
Date of test: 15th June 1994

Applied Load kN	Anchor 1	Anchor 2	Time Anchor 1	Time Anchor 2
0.0	0.0	0.0	8.05	9.35
20.0	0.08	0.00		
40.0	0.10	0.03		
60.0	0.12	0.03		
80.0	0.14	0.04		
100.0	0.17	0.06		
120.0	0.20	0.07		
140.0	0.24	0.08	8.13	9.42
140.0	0.24	0.08	9.10	10.35
0.0	0.16	0.00	9.15	10.30

Recorded movement represents elastic strain projecting anchor rod. No slip or creep or signs of damage to surrounding brickwork.

Table 5.

Results

Location UR 117
Tensile load tests
Date of test: 15th June 1994

Applied Load kN	Anchor 1	Anchor 2	Time Anchor 1	Time Anchor 2
0.0	0.0	0.0	13.55	15.14
20.0	0.0	0.0		
40.0	0.02	0.0		
60.0	0.03	0.02		
80.0	0.05	0.04		
100.0	0.06	0.05		
120.0	0.09	0.07		
140.0	0.10	0.08	14.09	15.28
140.0	0.10	0.08	15.06	16.30
0.0	0.02	0.00		

Recorded movement represents elastic strain in projecting anchor rod. No movement of the anchor or signs of damage to surrounding brickwork.

Table 6

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Project: Proof load tests on anchors for signal gantry supports - Fenchurch Street Line.
Date of test : 8th, 10th, 15th & 17th June 1994

Results

Location UR 516
Tensile load tests
Date of test: 17th June 1994

Applied Load kN	Anchor 1	Anchor 2	Time
0	0	0.0	
10	0	0.01	
20	0	0.01	
30	0.01	0.01	
40	0.01	0.02	
48	0.02	0.02	
48	0.02	0.02	after 30mins
0.0	0.0	0.0	

Table 7.

Results
Location UR 120
Shear load tests
Date of test: 8th June 1994

Applied Load kN	Anchor 1	Anchor 2	Time
0.0	0.0	0.0	16.40
5.0	0.1	0.14	
10.0	0.35	0.51	
15.0	0.65	0.84	
20.0	1.00	1.19	
25.0	1.32	1.52	16.46
25.0	1.48	1.60	17.46
0.0	0.36	0.23	

Table 8 see comment p.15

Location UR 117
Shear load test
Date of test: 15th June 1994

Applied Load kN	Anchor 1	Anchor 2	Time
0.0	0.0	0.0	10.40
5.0	0.30	0.04	
10.0	0.44	0.15	
15.0	0.56	0.27	
20.0	0.62	0.38	
25.0	0.93	0.49	10.48
25.0	1.03	0.51	12.00
0.0	0.17	0.28	12.08

Table 9 see comment p.15

Results Location UR 112/Shear load tests/Date of test: 10th June 1994

Applied Load kN	Anchor 1	Anchor 2	Time
0.0	0.0	0.0	13.35
5.0	0.15	0.13	
10.0	0.62	0.58	
15.0	0.14	0.88	
20.0	1.73	1.10	
25.0	2.37	1.31	13.42
25.0	2.59	1.34	14.45
0.0	0.46	0.40	14.50

Table 10 see comment p.15

Location of UR 516
Shear load tests - lower anchors no. 1 & 2
Date of test: 17th June 1994

Applied Load kN	Anchor 1	Anchor 2	Time
0	0.0	0.0	10.46
10	0.50	0.10	
20	0.90	0.32	
30	1.32	0.57	
40	1.66	0.81	
50	1.96	1.05	
60	2.27	1.28	
75	2.76	1.61	10.57
75	3.02	1.81	11.55
0.0	1.68	0.38	
Reload			
40	2.42	1.33	12.03
75	3.00	2.22	12.06
0.0	1.69	0.40	12.11

Table 11 see comment p.15

Results
Location UR 516
Shear tests - lower anchors no. 2 & 3
Date of tests: 17th June 1994

Applied Load kN	Anchor 1	Anchor 2	Time
0	0.0	0.0	12.15
10	1.97	1.60	
75	2.63	3.71	
0.0	1.49	1.44	12.25
Reload			
10	1.55	2.17	
20	1.82	2.89	
30	1.95	3.23	
40	2.17	3.79	
50	2.30	4.13	
60	2.42	4.45	
75	2.62	4.87	12.45
75	2.82	5.11	13.46
0	1.53	1.50	13.50

Table 12 see comment p.15

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Date of test : 8th, 10th, 15th & 17th June 1994

Results
Location UR 516
Shear tests - upper anchor
Date of test: 17th June 1994

Applied Load kN	Disp. mm 1st loading	Disp. mm 2nd loading	Time 1st load	Time 2nd load
0.0	0.0	1.25	10.06	11.20
10	0.05			
20	0.20			
30	0.34			
40	0.55			
50	0.70			
60	0.90	2.14		
70	1.10			
80	1.27			
90	1.49			
110	1.94			
120	2.21	2.25	10.12	11.23
120	2.28	2.28	10.17	11.28
120	2.28	2.28	11.15	11.35
0.0	1.25	1.25		

Table 13 see comment p.15

Comments on the shear tests

1. Locations UR120, UR117, & UR112
All these anchors sustained the 25kN proof load with no visible signs of distress to the surrounding masonary. There was no measurable increase in displacment ie 'creep', after the load had been applied for about 10 minutes. The results show some perminent displacement due to 'bedding' of the test shear plate on the anchor bar and of the anchor to the surrounding masonary.

2. Location UR516 - lower anchors.
These anchors were tested in pairs, anchors no. 1 & 2 together and 2 & 3 together. Anchor 1 & 2 were loaded in 10kN, increments up to the proof load of 75kN. This was maintained for 1 hour. There was no measurable creep after about 10 minutes and no visible signs of distress to the anchors or surrounding masoriary. However, displacement recovery for anchor no. 1 was less than 50% due we believe to 'bedding' down of the anchor in the masonary. In the light of this, it was decided to reload the anchor in two increments and record displacements. Recovery of displacement in this load cycle was 100 %.

In view of the experience testing anchor 1 & 2, the loading procedure for test anchor 2 & 3 was changed. The proof load was applied in two increments and then removed in order to ' bed ' in the anchors. The anchors were then loaded in 10kN increments to 75kN which was maintained for 1 hour. Recovery over this load cycle was better than 97% and there was no visible signs of distress to the anchor or masonary.

3. Location UR516 - upper shear anchor.

This anchor was loaded in 10kN increments to the proof load of 120kN which was maintained for 1 hour. There was no measurable creep after about 5 minutes of achieving the proof load and no visible signs of distress to the anchor or masonary. However, displacement recovery was less than 50%, the anchor was therefore reloaded in two increments to the proof load. Recovery over this load cycle was 100%.

Tested by: D Winter, Senior Technician
Approved by: R. Newby CEng MIMechE Associate Engineer

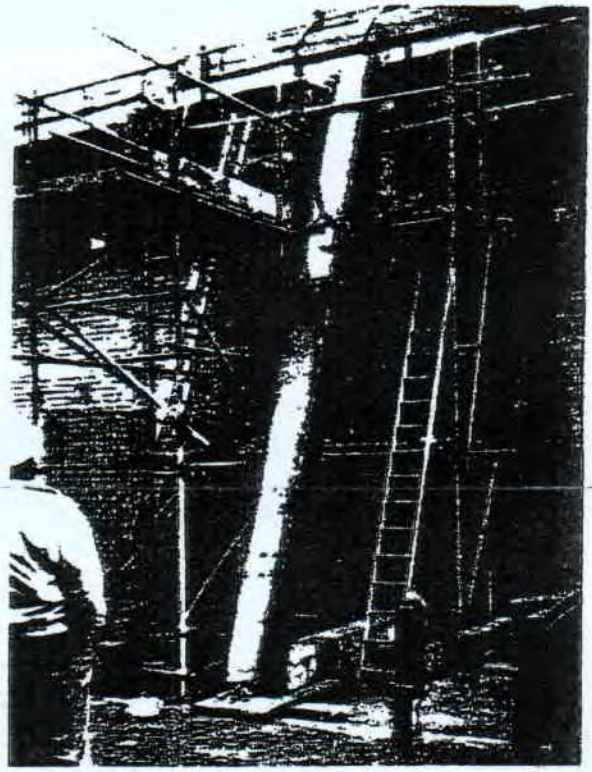


Plate 1
Arrangement of compression test reaction frame.

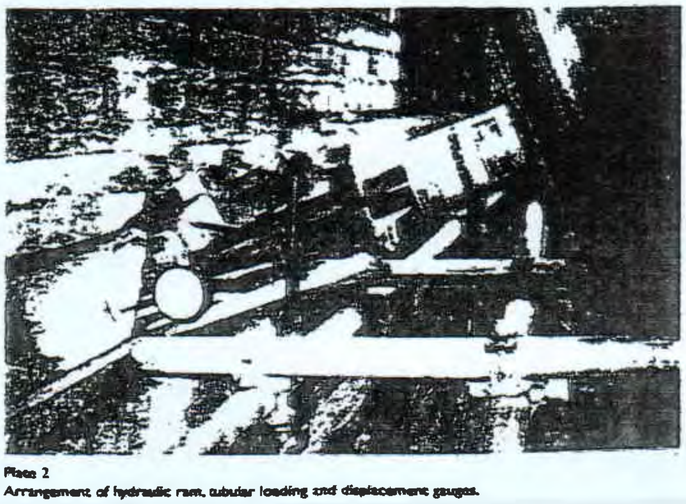


Plate 2
Arrangement of hydraulic ram, tubular loading and displacement gauges.

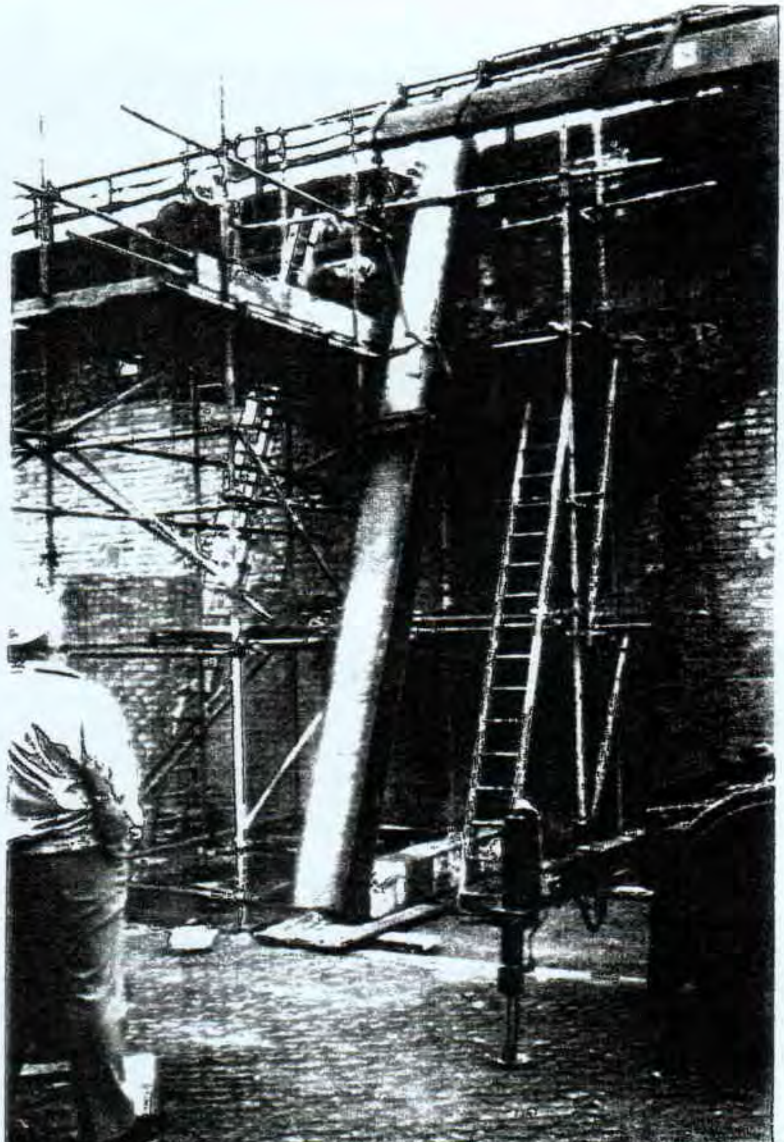
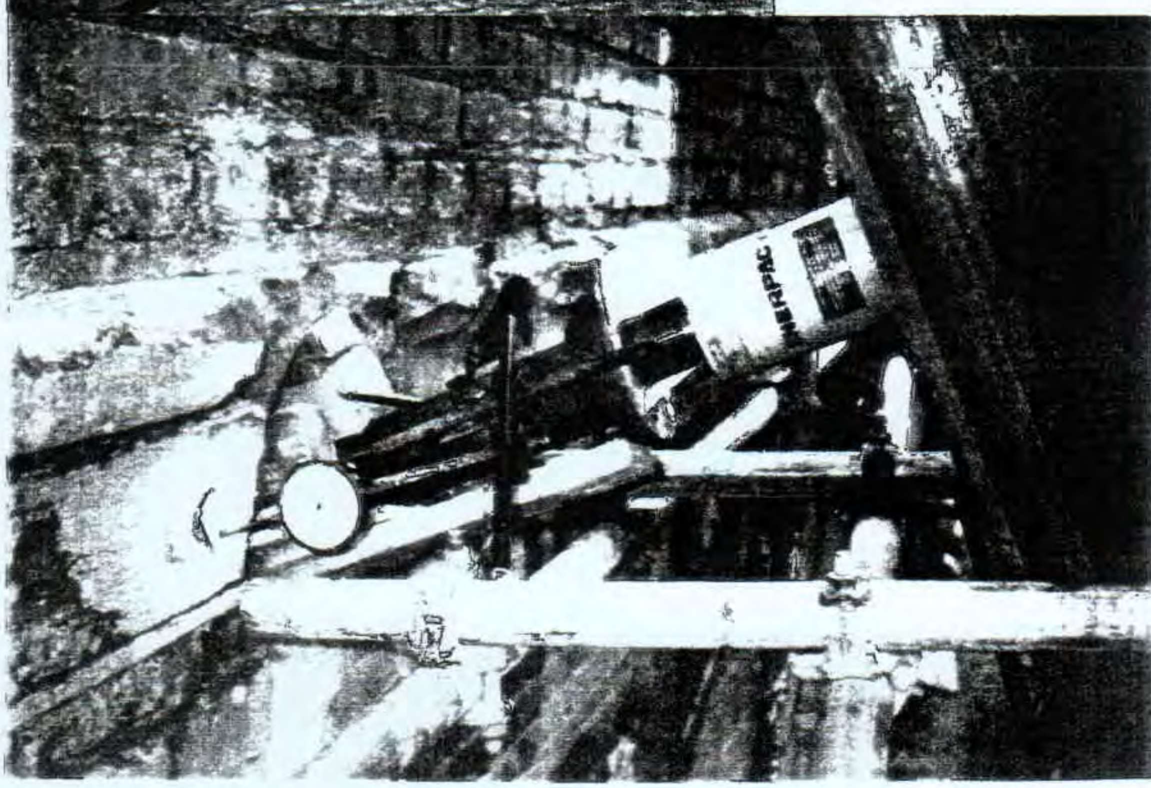


Plate 1.
Arrangement of
compression test
reaction frame.



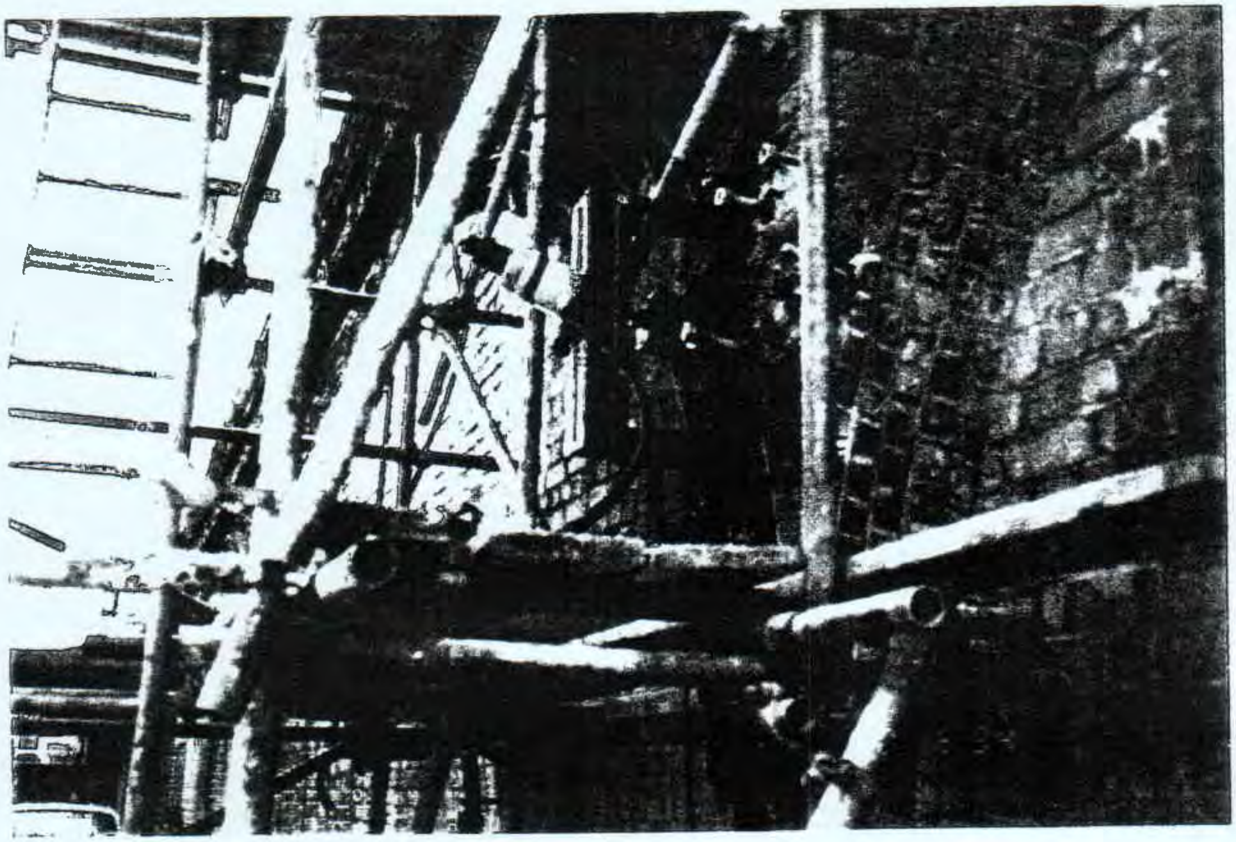
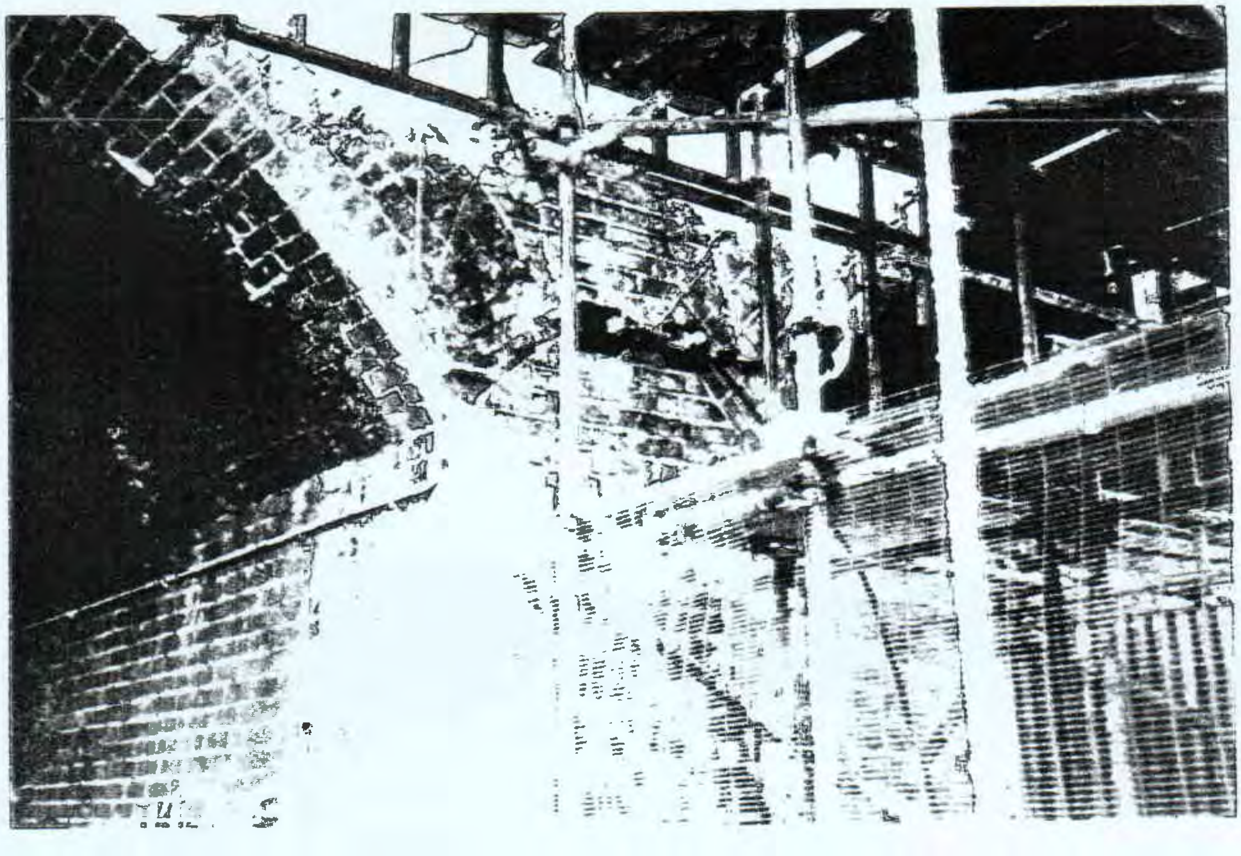


Plate 3. Arrangement of Tensile Test Rig



[illegible]

Plate 5. Arrangement of Upper Shear Anchor Test Rig

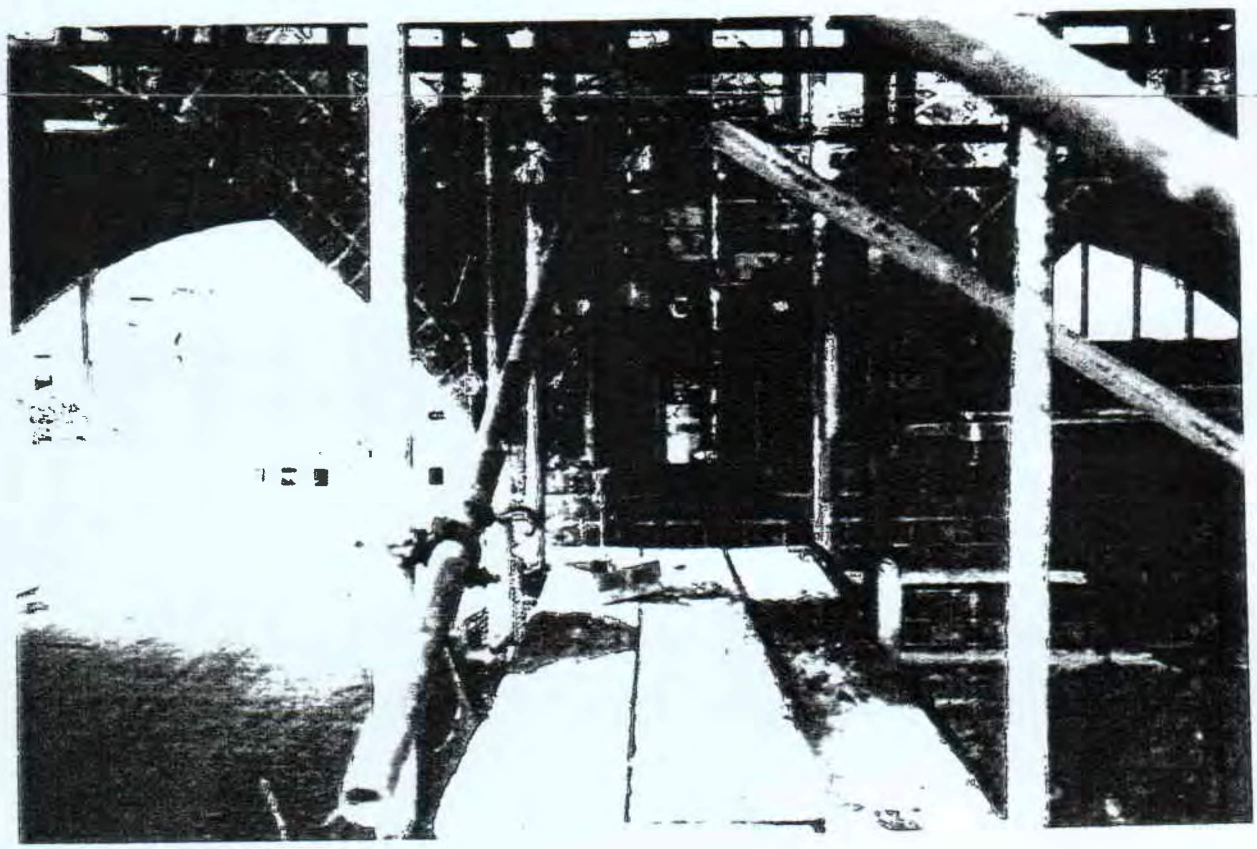


Plate 6. Arrangement of reaction beam for Upper Shear

Parapet Walls

Parapet strengthening and testing for London Underground. UK testing by Gifford and Partners (April, 1999)

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Our Ref: PDJ/B2056A/001

29 April 1999

London Underground Ltd
30 The South Colonnade
Canary Wharf
LONDON
E14 5EU

For the attention of Dr Weng Lau

Dear Sirs

EAST PUTNEY PARAPET STRENGTHENING & TESTING

Following the successful test of the parapet strengthening at East Putney Station on 27 April 1999, we have pleasure in providing you with a copy of our report. Should you wish to comment on the report or discuss the test further please do not hesitate to contact us through Cintec.

Yours faithfully

P D Jansen
Senior Engineer

Enc

cc: I Dimmick, Cintec

Commercial-in-Confidence

Report No. B2056A/R01
April 1999

LOAD TEST ON POST-TENSIONED PARAPET WALL

LUL EAST PUTNEY

Cintec International Limited
Factory Road
Newport
Gwent
NP9 5FA

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Commercial-in-Confidence

LOAD TEST ON POST-TENSIONED PARAPET WALL

LUL EAST PUTNEY

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Checked:	S Mehrkar Asl		
Gifford Approved:	G P Tilly		
Client Accepted:			

Revision Record					
Rev.	*Date*	*By*	*Summary of Changes*	*Chkd*	*Aprvd*

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LOAD TEST ON POST-TENSIONED PARAPET WALL

LUL EAST PUTNEY

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ABSTRACT

A 2m long section of a masonry panel wall at LUL East Putney Station was strengthened to resist wind forces and dynamic effects. It was post-tensioned using Cintec anchors.

In order to demonstrate the enhanced strength to the satisfaction of LUL, lateral load tests were carried out on 27 April 1999. The loads were taken to 30 per cent above the required level. The measured response of the wall was linear and values of deflection and strain were close to values calculated beforehand. There was no damage to the wall.

It was concluded that the strengthening was entirely satisfactory.

1. INTRODUCTION

Having undertaken assessments of many structures on their network London Underground Limited (LUL) have identified several as being under-strength but which could potentially be strengthened using a post-tensioned anchor system.

Cintec International Limited were approached by London Underground Limited (LUL) to demonstrate the adequacy of the Cintec Anchor System in strengthening existing LUL structures. Amongst these were the masonry parapet walls at East Putney Station which were found to be substandard under wind loading. The parapet strengthening is achieved by installing vertical Cintec anchors through the parapet wall and anchoring them in a mass concrete retaining wall below.

As part of this demonstration, Gifford and Partners (Consulting Engineers to Cintec International Limited) designed a strengthening scheme for the parapets using post-tensioned vertical Cintec anchors and developed a testing arrangement to verify the adequacy of the scheme.

A Method Statement was prepared in October 1998 and agreed with all parties.

The parapet walls at East Putney Station are 1.5 bricks thick at the top and vary in height from 1.5m to 1.8m approximately. For the purposes of the test, a panel has been selected with a height above substrate of 1.7m. The test panel is located on the disused platform on the outside of the westbound line (see Appendix A). At this location, the parapet is 330mm wide at the top (= 1.5 bricks thick) and 380mm wide at its base.

- No bond between the masonry parapet and the concrete retaining wall supporting it; the factor of safety against overturning of the parapet to be greater than 2.

- Bond stress between Cintec anchor sock and concrete 0.4N/mm^2 and between Cintec anchor sock and masonry 0.5N/mm^2.

3. PREPARATION OF STRENGTHENED TEST PANEL

The wall was in a poor state of repair having cracked joints, cracked bricks, bricks eroded by 10 to 15mm at ground level and joints damaged by ivy and other vegetation that had penetrated quite deeply. There had been repairs at different times, some successful, others not. Preparation of the wall was as follows:

3.1 Removal of vegetation from wall and temporary removal of metal railings in vicinity of parapet to be strengthened.

3.2 Removal of top course of bricks local to positions of Cintec Anchors.

3.3 Vertically saw cut 2m section of parapet to separate the testing panel from the adjacent length of wall.

3.4 Diamond drill 2 No. 56 mm diameter vertical holes at 1m centres through the centreline of the masonry parapet and through the supporting mass concrete retaining wall to provide an embedment length of 850mm in the concrete. Each anchor to be located 500mm from the end of the panel (see details of strengthening panel in Appendix A).

3.5 Removal of all cores from the holes and depth checked.

3.6 Installation of 16mm diameter anchors in accordance with Cintec's "Notes for Approved Installers Using the Cintec Designed Anchoring System".

3.7 Anchors grouted into the mass concrete retaining wall.

3.8 When the anchorage into the retaining wall had achieved adequate strength, tie bars tensioned to 57kN.

3.9 Anchors grouted into parapet walls.

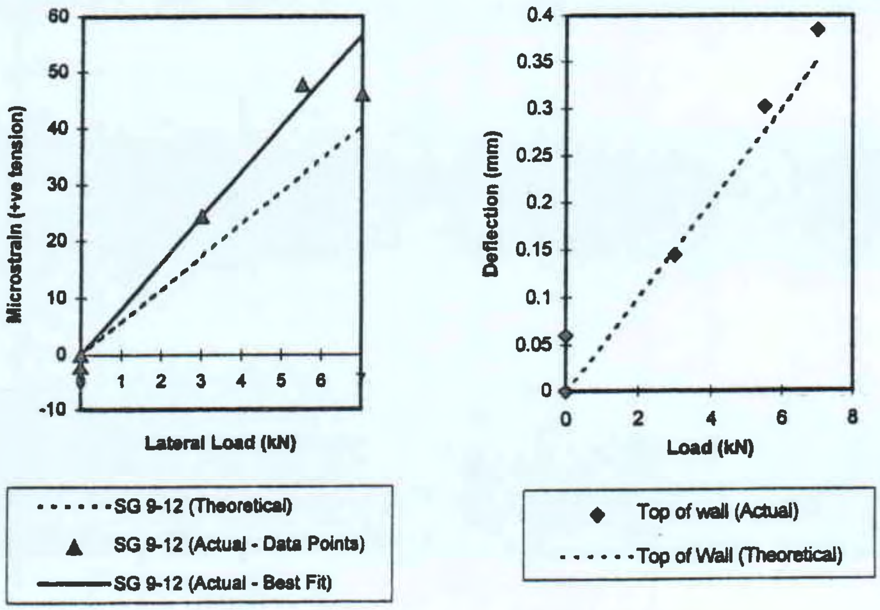
At the end of the test there was no evidence that the loading had caused any damage such as cracking or spalling.

5.2 Test Two

As some of the gauges malfunctioned in Test One, they were re-set and a second load test was carried out immediately after the first, starting at 12.30pm.

A higher maximum load of 7kN was applied, ie approximately 30 per cent higher than required by LUL. The resulting values of deflection and strain are given in Appendix B.

The maximum averaged deflection (at the top of the wall) was 0.38mm and the maximum averaged tensile strain (at the bottom of the wall) was 54.8 micro strain. The relationships with load were linear, see Figure 2. Again the theoretical strains and deflections are plotted for comparison with the test results, and again the correlation is surprisingly good.



Test 2 - Strains

Test 2 - Deflections

Figure 2

As for the first test there was no evidence that the second and higher loading had caused any cracking or spalling to the wall.

6. CONCLUDING REMARKS

The 2m long test-section of parapet wall at East Putney was successfully strengthened against wind and dynamic pressure and suction loading, using two Cintec anchors.

Two tests were carried out. The first was to the required maximum lateral load of 5.5kN, the second to 7kN. Responses, as measured by deflection gauges and vibrating wire strain gauges were linear elastic.

Responses calculated beforehand, using assumed values of the material properties, were within 30 per cent of the measured values. Bearing in mind the wide range of uncertainties in relation to the wall stiffness and strength, this is surprisingly close. It should be noted that on atypical masonry wall, values of strength and stiffness can vary by ±15%. On completion of the tests there was no damage such as cracking or spalling.

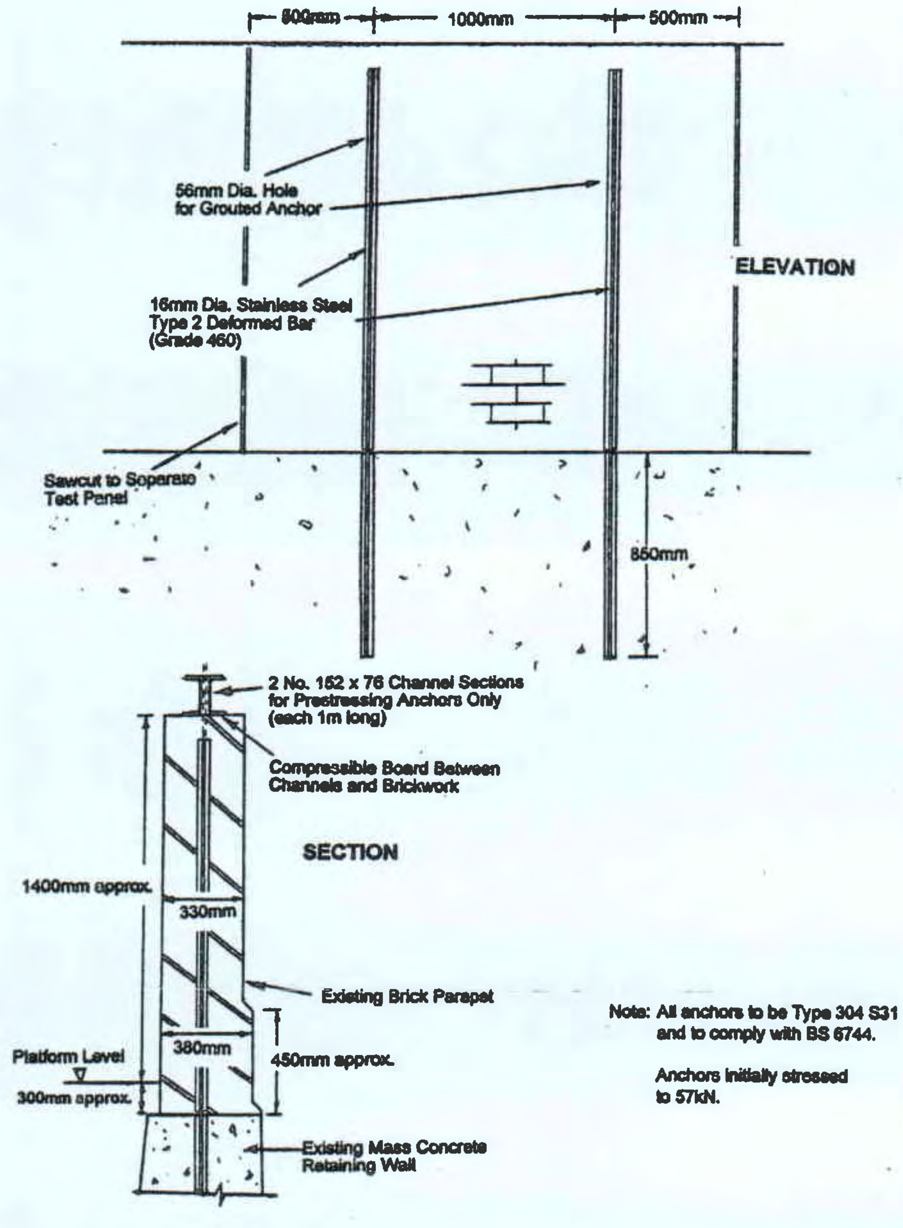
It is concluded that the tests were successful in demonstrating the efficacy of strengthening an old brick wall in a poor state of repair.

7. INTELLECTUAL PROPERTY RIGHTS

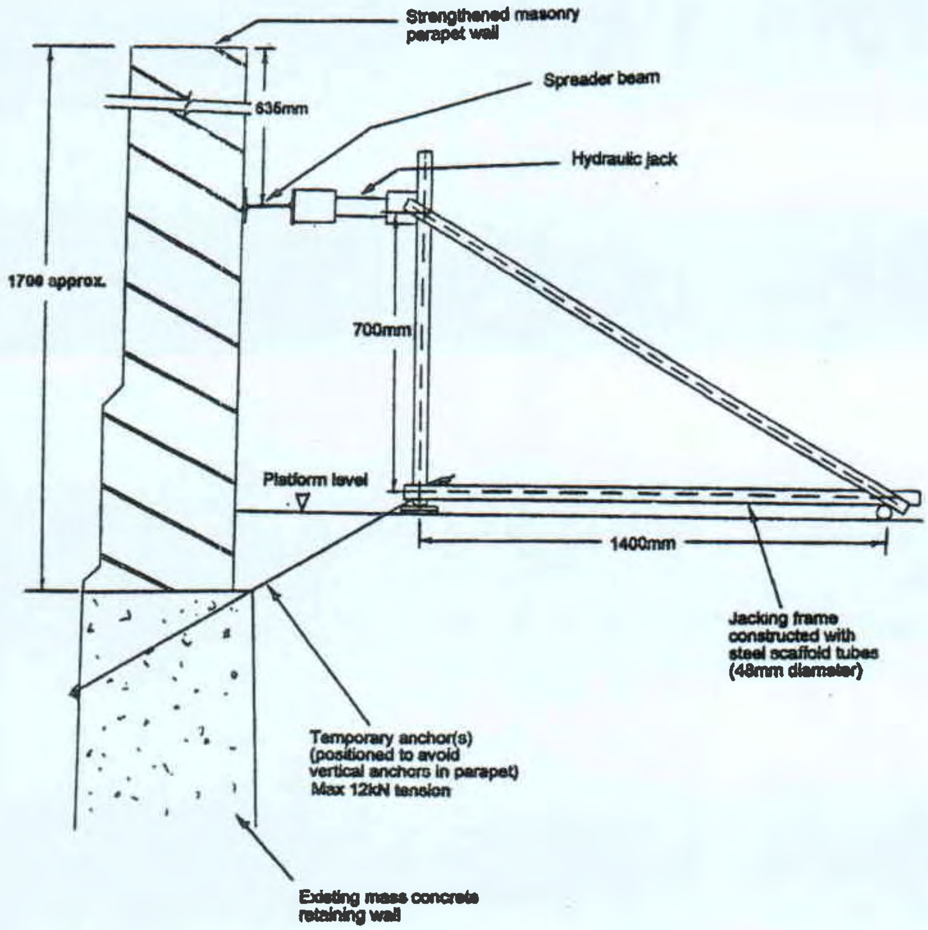
The Intellectual Property Rights for these strengthening and testing works are jointly owned by Gifford and Partners and Cintec International Limited.

APPENDIX A

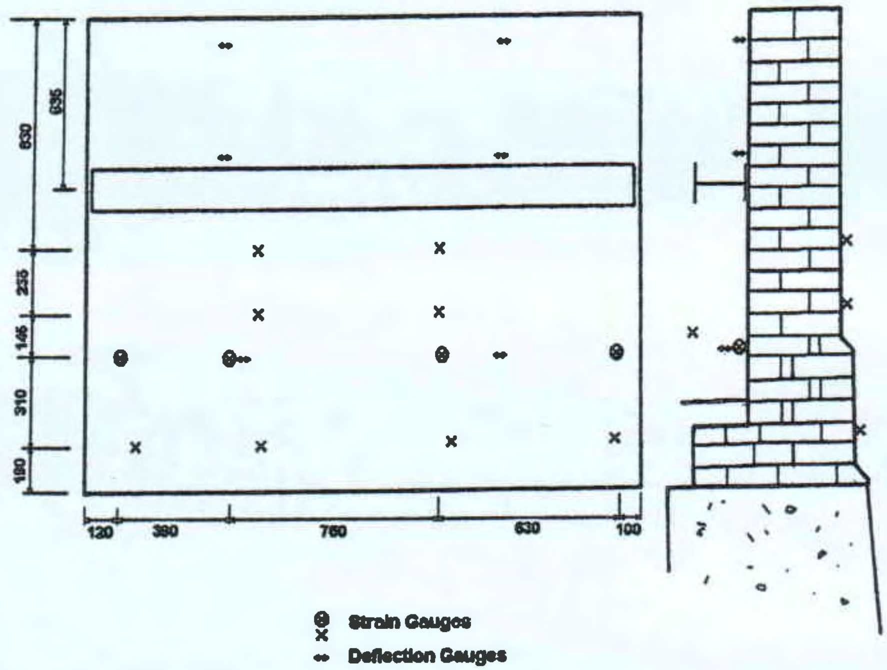
LOCATION AND DETAILS OF TEST PANEL



Details of Strengthening Scheme



Details of Testing Arrangement - Sheet 1



Details of Testing Arrangement - Sheet 2

APPENDIX B

TEST RESULTS

Test 1 - Strain and Deflection Measurements for Lateral Load taken to 5.5kN

Load (kN)	Micro Strains with temperature correction (+ve = compression) Gauge-1	Gauge-2	Gauge-3	Gauge-4	Gauge-5	Gauge-6	Gauge-7	Gauge-8	Gauge-9	Gauge-10	Gauge-11	Gauge-12
0	0	0	0	0	0	0	0	0	0	0	0	0
1	3.43	0.88	2.08	4.12	-2.68	0.06	-2.31	0.59	-1.70	-3.04	-2.60	-3.32
2	6.50	7.90	7.59	5.76	-2.94	-1.15	-5.72	-2.74	-3.40	-9.97	-8.19	-9.81
3	22.91	26.35	22.96	17.25	4.92	-6.75	-3.27	2.50	-5.40	-24.40	-19.23	-24.06
4	38.46	47.40	38.14	28.12	21.13	11.45	8.32	11.45	-3.00	-28.27	-22.87	-27.96
5.5	57.40	73.01	59.93	44.63	33.41	16.27	12.19	14.21	-1.50	-44.11	-35.70	-43.27
3	39.63	54.60	41.66	31.46	25.02	9.16	12.19	5.30	-0.30	-30.86	-25.61	-28.97
0	2.99	6.25	7.38	12.48	3.69	3.46	4.61	5.40	3.58	-8.14	-1.50	2.37

Load (kN)	Deflections (mm) Dial-1	Dial-2	Dial-3	Dial-4	Dial-5	Dial-6
0	0	0	0	0	0	0
1	0.02	0.00	0.00	0.03	0.04	0.00
2	0.04	0.00	0.04	0.06	0.07	0.00
3	0.07	0.00	0.13	0.13	0.15	0.00
4	0.09	0.00	0.14	0.17	0.20	0.00
5.5	0.115	0	0.26	0.235	0.25	0
3	0.195		0.2	0.175	0.2	0
0			0.07	0.075	0.045	

Test 2 - Strain and Deflection Measurements for Lateral Load taken to 7kN

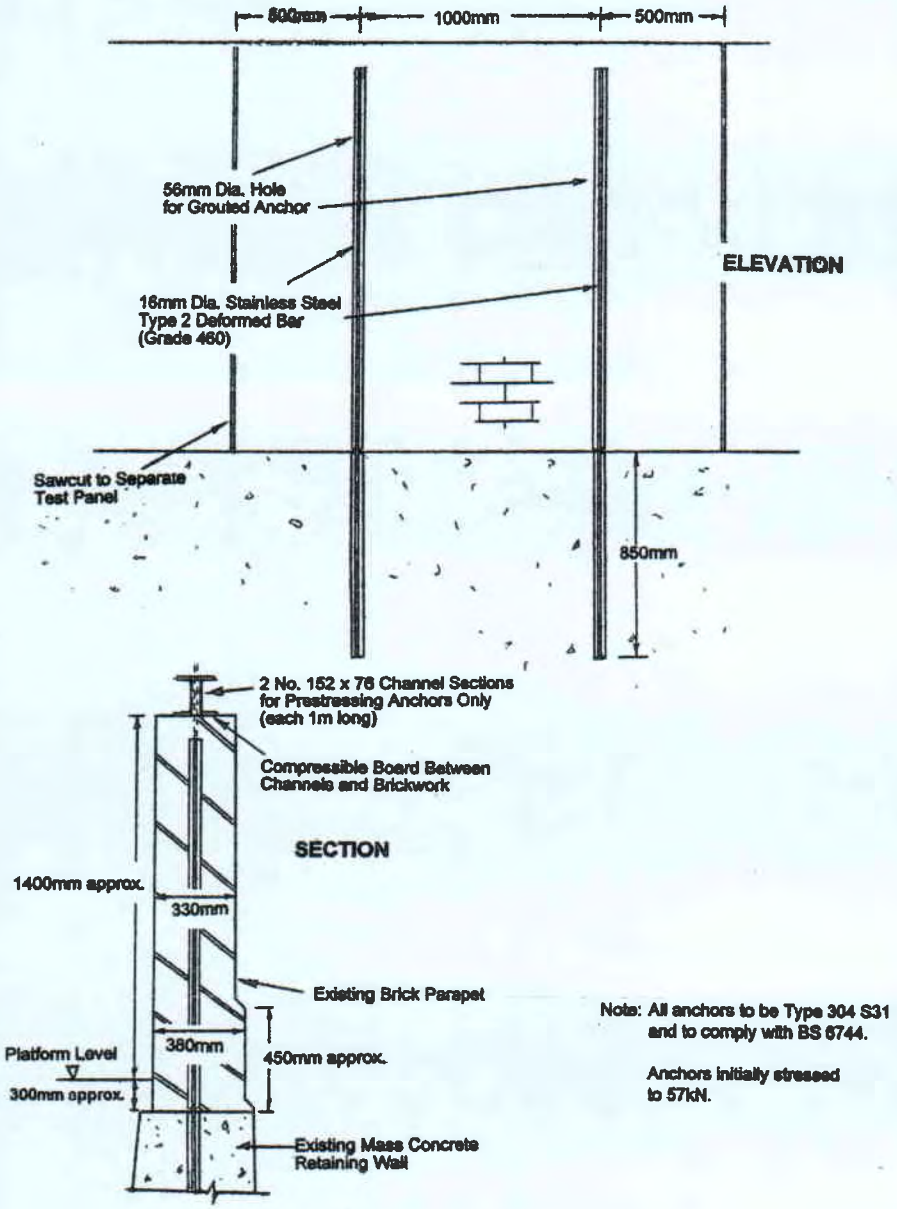
Load (kN)	Micro Strains with temperature correction (+ve = compression) Gauge-1	Gauge-2	Gauge-3	Gauge-4	Gauge-5	Gauge-6	Gauge-7	Gauge-8	Gauge-9	Gauge-10	Gauge-11	Gauge-12
0	0	0	0	0	0	0	0	0	0	0	0	0
3	17.24	25.52	26.60	18.15	5.44	-0.06	-3.14	-2.45	-10.20	-30.44	-23.74	-27.45
5.5	34.28	47.35	43.32	29.42	7.73	-3.03	-8.04	-6.46	-19.20	-57.89	-47.96	-54.45
7	80.52	95.56	81.68	60.40	39.43	18.15	15.55	11.43	-3.01	-59.99	-47.19	-57.07
0	5.96	3.99	-1.73	2.58	-2.37	-8.42	-5.20	-9.44	6.61	-0.19	1.80	2.71

Load (kN)	Deflections (mm) Dial-1	Dial-2	Dial-3	Dial-4	Dial-5	Dial-6
0	0	0	0	0	0	0
3	0.02	0.02	0.12	0.095	0.15	0.14
5.5	0.040	0.020	0.190	0.155	0.245	0.360
7	0.060	0.020	0.275	0.205	0.330	0.440
0	0.060	0.010	0.010	-0.005	0.000	0.120

NOTE: For location of strain gauges and trial gauges refer to Details of Testing Arrangement - Sheet 2 in Appendix A

APPENDIX A

LOCATION AND DETAILS OF TEST PANEL



Details of Strengthening Scheme

Seismic

Introduction to describe how a discrete element formation has been employed to simulate masonry under seismic loading.

Al-Ghuri Mosque, Cairo, Egypt.
Finite Element Analysis Report on Results, Gifford and Partners.

Summary of Results from Horizontal shaking Table Tests. Southampton University (1996)

Earthquake Damage Repair and Strengthening.
Christchurch Cathedral, Newcastle, Australia.
Report July 1977

Seismic Tests to a full-scale model part of the Sao Vicente de For a Monastery, Lisbon. Testing by Elsa Laboratory Joint Research Centre, Italy

Anchor Tests Mission San Juan Capistrano, California. USA Testing Engineers: Twinning Laboratories, 2001

Anchor Tests Wylfa Nuclear Power Station, Wales. Testing by Celtest & Cintec International, December 2000

Numerical modelling of masonry using discrete elements

C L Brookes
Gifford and Partners, UK

S Mehrkar-Asl
Gifford and Partners, UK

ABSTRACT: This paper describes how a discrete element formulation has been employed to simulate masonry under seismic loading. Using non-linear numerical analysis the performance of two typical shear walls with and without additional strengthening has been compared. The first is representative of masonry built dry using accurately sized stone blocks, forms the end of a two storey building and is loaded horizontally. The second is similar but typical of more random brick masonry with high strength mortar. In both cases the benefits of strengthening by the introduction of passively stressed reinforcement are predicted when subjected to a hypothetical earthquake. It is concluded that, although more work is required to verify simulations against tests, the discrete element technique is ideally suited to dynamic modelling of masonry and overcomes many difficulties experienced with traditional finite element analysis.

1 INTRODUCTION

The use of continuum based numerical models to generally simulate discontinuous structures, such as masonry, is fraught with difficulty. The introduction of discontinuities such as cracks during the loading event, or as a result of loading history, resulting from tensile forces has to be wholly or partly predetermined. The use of gap elements allows cracks to open and maintain normal and shear force connection when closed but the crack locations have to be known in advance. Another approach is to avoid explicit representation of discontinuities but instead smear their effect by using a brittle non-linear material model. However, these models fail to predict mechanisms where, for example, initially isolated parts react dynamically together. Continuum methods can give satisfactory results but generally fail to provide a general and cost effective method of analysis for masonry.

As an alternative to the traditional finite element continuum approach, a discrete element formulation has been employed to simulate masonry under seismic loading. So far the results of the analyses have been applied to parts of buildings and used to help determine remedial design philosophies by providing simulations under specific ground excitations. In a separate project, calculated collapse loads of masonry arch bridges have been shown to correlate very closely with full-scale tests (Brookes C L, 1998). As the technique is developed it is hoped that the performance of whole buildings can be checked once strengthening systems have been installed.

2 ANALYTICAL REQUIREMENTS

In order to represent masonry, particularly in seismic engineering where non-linear structural performance defines how ductility and energy absorption characteristics are exhibited, the following types of fundamental behaviour need to be included in the model.

i) Material and geometric properties of the masonry blocks themselves.

ii) Contact/gap/friction effects along joints between the masonry blocks.

iii) Depending on block and joint properties the ability to evolve further joints by fracturing which in turn depends on limiting tensile strength and fracture energy.

iv) Full account of stiffness and derived inertia loads which may occur over very short time intervals.

v) The capability to model post-failure behaviour to help verify simulations against the evidence collected after observed seismic damage and collapse.

vi) To allow stress and initial damage from previous seismic events to be included.

To date, most numerical simulations of masonry have been based on finite element continuum methods in which sophisticated and often complex material models in conjunction with arrays of gap elements are used to account for the requirements listed above. A more intrinsically satisfactory approach is to base the analysis on a series of *discrete elements*. This more natural approach can be used to represent ranges of masonry from completely intact buildings to piles of random rubble.

3 DISCRETE ELEMENT TECHNIQUE

The technique used to perform all the analysis in this study is the discrete element (DE) method. This is a development of the distinct element method (Cundall, 1971) in which the concept of individual elements being separate and reacting with their neighbours by contact through friction/adhesion was first successfully applied to geotechnical and granular flow problems. Here elements were considered rigid but later developments (Munjiza et al, 1995) included the addition of element deformations and fracturing, with some overlap with traditional finite element theory.

In the current investigation the DE formulation available in the explicit dynamic version of ELFEN (Rockfield Software Limited, 1996) has been used. The non-linear analysis of masonry has been separated into two types each requiring a different modelling approach. The first has been termed *Macro Blocks* and is the category where the joints between blocks have predominantly no strength and models the construction generally found in historic structures. The second has been named *Brittle Material* and is where the masonry blocks and joints have predominantly similar strengths, as is more likely in modern forms of construction.

3.1 Macro block

The macro block approach has been achieved by separately modelling each block or group of blocks in the structure and applying permanent static loads and seismic excitation to the base. Individual blocks of elements have defined elastic materials and are arranged to the required bond. All joints and therefore potential discontinuities are predefined and have friction parameters assigned. It is assumed that failure at joints always develops before blocks fail.

3.2 Brittle material

Where masonry includes high strength mortar or where the strength of blocks is low, a brittle non-linear material model has been used. Here the continuum becomes discretised due to evolving fractures in the blocks and possibly through joints. This is achieved in the analysis automatically using adaptive mesh algorithms. Newly generated cracks become contact surfaces requiring friction parameters to be assigned as for the macro block approach. A Rankine material model was used including fracture energy.

4 SHEAR WALL INVESTIGATION

As part of the assessment of the seismic resistivity of a large number of two storey masonry buildings in the Middle East, a detailed analysis of masonry shear walls was undertaken. One of these shear walls is described below. The main objectives of the analysis were to provide some comparison between the performance of the walls with and without strengthening and to explore the potential of the DE technique applied to masonry under dynamic loading. Recent studies using the DE method to predict the ultimate strength of masonry arches, including comparisons with full scale tests, has shown the technique to be very accurate and better than alternative analyses for the case of static loading.

4.1 Description

The masonry shear wall under investigation is shown in Figure 1. The wall is 6.15m wide, 6.43m high and has shallow foundations over rock. It forms the shorter side of the rectangular building shown in Figure 2 and supports two reinforced concrete floor slabs that behave as diaphragms. The longer side walls (not explicitly modelled) partly support the floor slabs, and have little out-of-plane shear resistance. Vertical body forces and imposed loads are supported uniformly by all external walls. Loads developed by horizontal seismic ground accelerations in the transverse direction of the building are resisted by in-plane forces in the side walls. One of these walls is the subject of the current investigation.

4.2 DE Model

Several plane stress DE models of a single side wall were developed incorporating the masonry blocks and reinforced concrete slabs. The vertical loads and masses attributed to the slabs were modified to reflect mass and load transfer from the rest of the building. The wall was constructed from hollow clay blocks, bedded on sand and cement mortar and rendered with plaster.

Without full scale testing of part of the wall, uncertainties exist for most material parameters and indeed the modelling. Hence, both macro block and brittle material masonry models were developed to help characterise the walls behaviour. Macro block models were used to investigate sensitivity to equivalent friction of the mortar joints and strengthening. Brittle material models were developed to investigate fracturing of the mortar and blocks resulting from stronger joints, and the influence of strengthening.

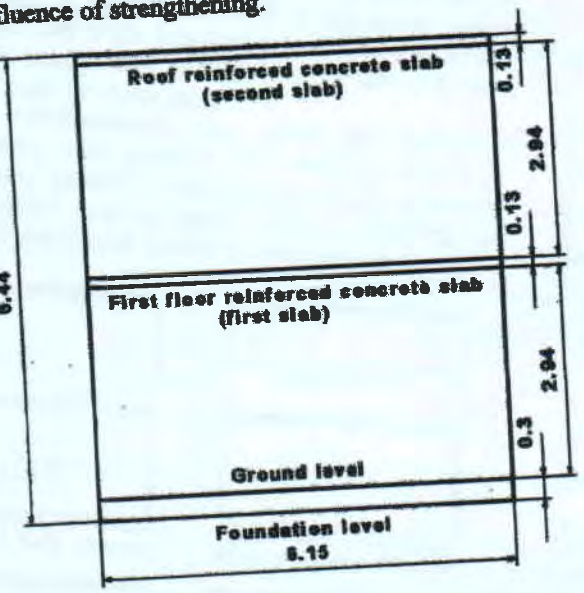


Figure 1. Masonry shear wall details (all dimensions in metres)

Between two and six actual blocks were represented by each macro block. Although it is feasible to include all of the blocks, previous work on masonry arches had shown that there was little advantage in terms of accuracy and a considerable disadvantage in terms of computational efficiency. The reinforced concrete slabs and foundation were defined as separate continuums with similar perimeter frictional properties to the blocks. In both model types the slabs and foundations were linear elastic.

All models were developed within a solid modelling environment using DISPLAY3 (EMRC, 1996) and exported to the ELFEN pre-processor.

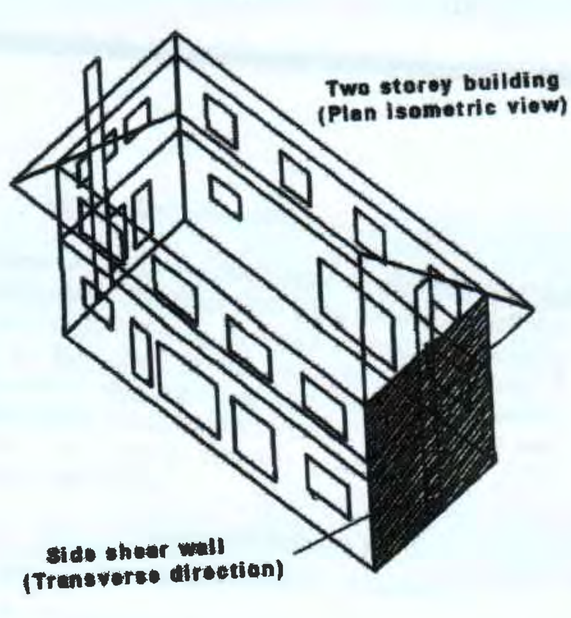


Figure 2. General arrangement of building

4.3 *Loading*

Hypothetical horizontal seismic loading based on a circular frequency of approximately 0.6 Hz and containing four shocks was derived and applied to all of the models as displacement functions at foundation level (Figure 9). Peak acceleration for this simplified motion was 0.15g.

Vertical accelerations were not considered due to the inherent inconsistencies in the distribution of mass that were required to simplify the problem to one of two dimensions. Horizontal motion results in a greater proportion of the effective mass being distributed to the two shear walls than that corresponding to vertical motion. Vertical motion is resisted by all load bearing walls. However, whilst concurrent vertical motion has an influence on the overall behaviour of masonry shear walls, it is generally accepted that horizontal motion is critical.

4.4 *Strengthening*

Both model types were modified to include a strengthening system. The proposed system consists of horizontal reinforcement through individual block courses. In effect a full width lintel capable of carrying tensile force is retrofitted to the wall. The reinforcement is fully bonded along its length. It is not deliberately stressed but attracts load during a seismic event. Experimental results and reported field evidence (H A Moghaddam, 1993) has shown that this type of strengthening forms a barrier in the wall helping to contain the development of cracks into adjacent masonry panels. It also retains some ductility by allowing blocks to move on either side. The practice of installing horizontal timber lintels in stone masonry to help control cracking has been observed in several ancient buildings with varying degrees of success.

Figure 3 shows the locations of reinforcement and resulting lintels that were the subject of the investigation. Here the term lintel has been used to describe the line of blocks bonded together with the reinforcement. Using the macro block representation three arrangements were considered; positioned at the centre of the bottom panel, at the centre of the top panel and at the two sites combined. The brittle material model was modified to include only the bottom panel arrangement.

The reinforcement bars were not explicitly represented with separate line elements but their effect was globally simulated by using a simple linear elastic material model thereby allowing the course of reinforced blocks to resist tensile forces.

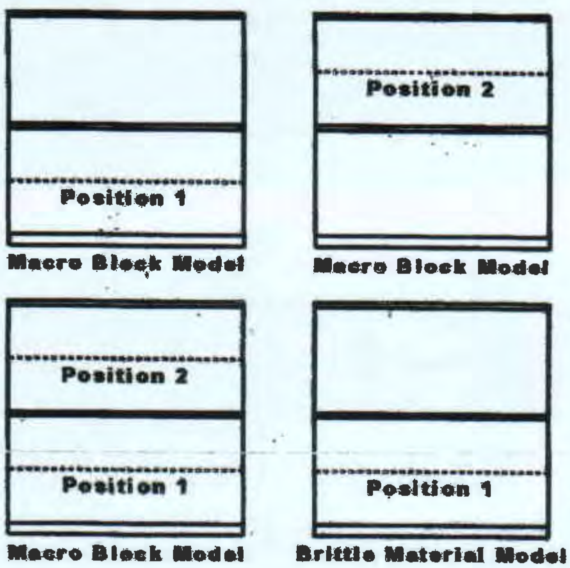


Figure 3. Position of horizontal reinforcement strengthening

5 DISCUSSION OF RESULTS

DE simulations were carried out to show how the strength and ductility of the walls varied with the mortar joint properties and the location of reinforcement. The results have been illustrated by arrays of contour diagrams in which, the coefficient of friction (μ) in the joints, and the position of reinforcement have been varied.

5.1 *Macro block simulations*

Figure 4 shows the horizontal displacement of the foundation, first floor slab (first slab) and roof slab (second slab) during the seismic event for values of μ in the joints of 0.2, 0.6 and 1.0. Figure 5 illustrates displacements and deformed geometry after shock 1 and the final shock (number 4) for each case. The shaded contours range between -0.2m (dark) and 0.05m.

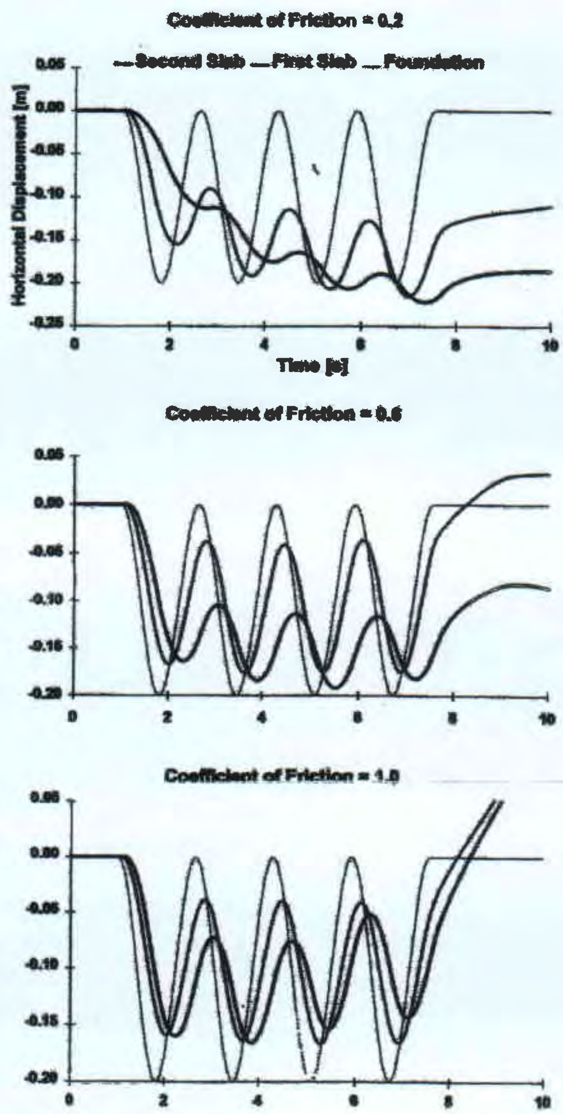


Figure 4. Horizontal displacement of slabs and foundations

The predicted ductility of the walls is highly sensitive to the properties of the joints and the duration of the event. In all cases the inherited damage history from preceding shocks has increased the seismic vulnerability of the wall. With a lower bound value $\mu=0.2$ significant damage is inflicted on the first storey. Steady state conditions return about 2 seconds after the shocks and the slabs displace by up to 0.2m. For a more realistic value of $\mu=0.6$ collapse of the second storey occurs after shock 4 and there is less overall movement of the reinforced concrete slabs. An upper bound value of $\mu=1.0$ causes similar amounts of damage to each storey and does not undergo collapse.

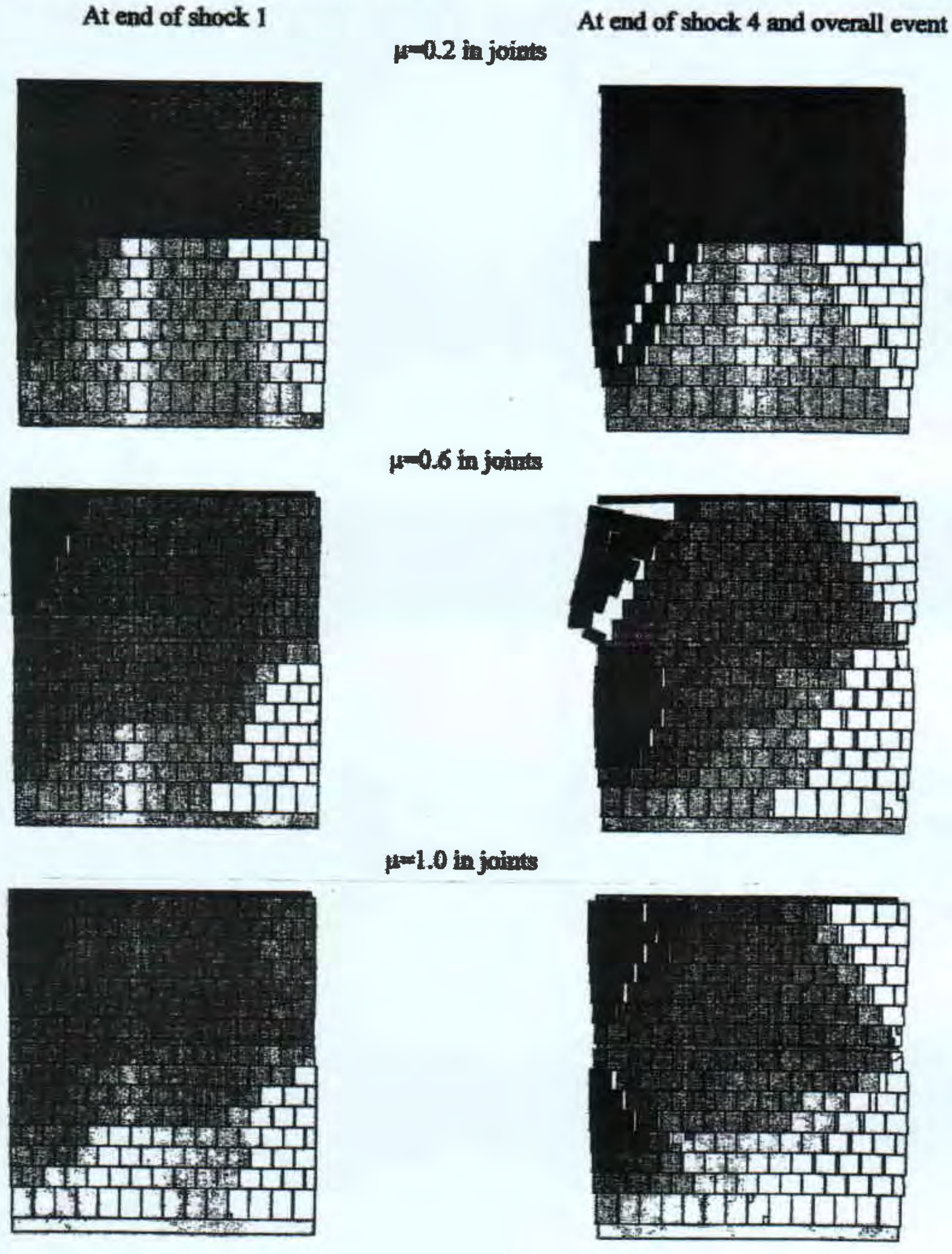
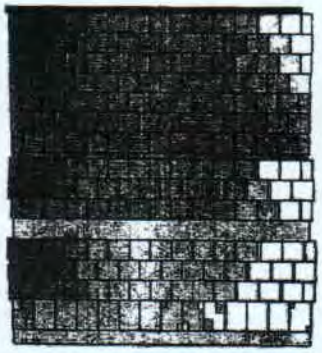


Figure 5. Macro block model simulations with varying joint friction

At end of shock 4 and overall event
with μ=0.6 in joints
Reinforcement in position 1



Reinforcement in position 2

Reinforcement in positions 1 and 2

Figure 6. Comparisons between positions of reinforcement

Predictions when passive reinforcement has been included are shown in Figure 6. Here results have been calculated for μ=0.6, which is considered to be most probable, and for the three reinforcement arrangements. All strengthening schemes improve the seismic resistivity of the wall. Reinforcement in position 2 is least efficient and whilst preventing partial collapse of the second storey causes more damage to be inflicted on the first storey. The best method of resisting the hypothetical loading with least cracking is by strengthening the first storey alone. This scheme does not reduce the ductility of the second storey. Results also show that this method of strengthening has little influence on the displacement of the two slabs which if left unarrested could propagate failure.

5.2 Brittle material simulations

Figure 7 illustrates displacements and deformed geometry after shock 3 and after the final shock where severe fracturing has damaged the lower storey. The results show that the initiation of cracking began just before the end of shock 3 and developed rapidly into local failure mechanisms when subjected to the final part of the seismic event. The predicted failure and local collapse, reflecting modelled ductility and energy absorption, is similar to damage frequently sustained in seismic regions.

At end of shock 3

At end of shock 4 and overall event

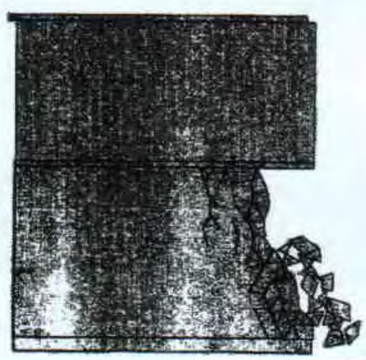


Figure 7. Brittle model simulations showing development of cracking and local failure

Figure 8 shows contoured equivalent principal compressive stresses, immediately after loading, with and without strengthening. Here the reinforcement has been positioned in the lower storey. Equivalent stress contours, prior to adjustment for the hollow blocks, range between 0 and -0.08 N/mm^2 (dark).

The modelling approaches (macro block and brittle material) represent two extremes in masonry construction regarding the arrangement for strengthening. Interestingly, both suggest a degree of independence from masonry properties.

6 CONCLUSIONS

The discrete element technique is ideally suited to dynamic modelling of masonry. The ability to represent a non-homogenised continuum is a more natural way of representing regular and random arrays of blocks and overcomes many difficulties experienced with non-linear contact analysis using the more traditional finite element technique.

At end of shock 4 and overall event

At end of shock 4 and overall event with reinforcement in position 1

Figure 8. Brittle model simulations showing equivalent principal compressive stresses with and without strengthening

By considering a simple plane shear wall and two separate models, it has been possible to illustrate how the discrete element method can be applied to ancient masonry where mortar is often weak, or non-existent, and to modern masonry where mortar is strong. The example has also been used to investigate the sensitivity of seismic resistivity to mortar properties and to explore a simple strengthening scheme involving retrofitted reinforcement. Both exercises would have been extremely difficult and costly using conventional analysis or testing.

Further work is required to verify the simulations against observed behaviour of masonry structures subjected to seismic loading.

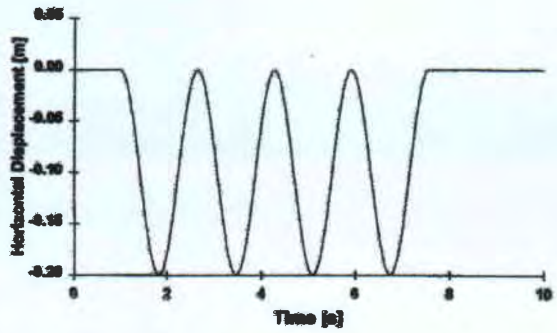


Figure 9. Time history of input motions

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AL-GHURI FINITE ELEMENT ANALYSIS

REPORT ON RESULTS

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1. INTRODUCTION

Gifford and Partners have been appointed to conduct a numerical analysis using the finite element technique on Al-Ghuri mosque in Cairo, Egypt, which suffered severe cracking during the earthquake of October 1992. As part of a general programme of proposed repairs to earthquake-damaged buildings throughout the city, it has been agreed to carry out a computer simulation of one building to substantiate our proposed method of repair.

This report assumes a basic knowledge of dynamic analysis and finite element modelling and their terminologies.

2. BRIEF

The brief was to build a finite element model of Al-Ghuri mosque and to use this to predict the behaviour of the damaged mosque under further earthquake loading. The most significant known cracks in the mosque were to be included in order to reflect as accurately as possible its current state. Because cracks behave differently under tension and compression then a non-linear analysis would be required. This model could then be adapted to assess the design loads in the stitching anchors forming the proposed repair system. These design loads could then be compared with the anchor load capacities to determine a factor of safety. This would require a linear analysis. For now, however, the accuracy of the input data limits the reporting of results to the more behavioural aspects. In tandem with these specific aims, this was also to be an open-ended study of masonry modelling; to begin developing techniques that could be applied to masonry buildings in general.

3. EARTHQUAKES

This report does not give an exhaustive appraisal of seismic activity. Seismology is a complicated subject to which many scientists and engineers have devoted many years. Our research into earthquake behaviour has been limited to determining suitable input ground motions for our model. These inputs are discussed in section 6.2. It appears there is no available recorded data of the Cairo earthquake. The main sources of earthquake information that were used in this project are listed in the references.

4. SOFTWARE

A commercially available, general finite element package by Engineering Mechanics Research Corporation (EMRC) was used. This comprises a whole family of programs of which we used DISPLAY3 for pre- and post-processing (ie building the model and viewing the results) and NISA2 for processing (ie performing the calculations). All DISPLAY3 information pertaining to a model is stored in a database file. DISPLAY3 also writes out the model and analysis data into a NISA file which can be read by NISA2. This file is in a text format which with a little familiarity is quite easy to read and understand, hence sections have been referred to throughout this report. NISA2 produces a text output file containing all input information and selected results and binary output files that contain results for stress contour, deformation and graph plotting in DISPLAY3. A feature of DISPLAY3 that has proved to be particularly valuable for this analysis has been the movie facility. It is possible to capture the stress contour and deformation plot for each load step of an analysis in a sequence of frames and play them back as an animation.

Units in DISPLAY3 and NISA2 are not fixed. This project uses the megametric system. This means all values are input in MN, m, Gg and seconds. For example acceleration is in m/s^2 and stresses are in MN/m^2 which is equivalent to N/mm^2.

5. MODEL

This section describes the thought processes behind the development of the model and refers the reader to the appropriate part of the NISA file, in Appendix A.

5.1 Geometry

From a set of 1:100 scale plan, section and elevation drawings it was possible to construct the geometry of the mosque. The building is quite complex in places and had to be simplified to keep the model a reasonable size. Such simplifications can be justified. From the principle of diminishing returns, increasing the model complexity by 10% will not necessarily yield 10% more accuracy in results. Our knowledge of the mosque was limited to a set of drawings, a series of photographs and a Gifford inspection report. The inspection report mentions the mosque walls being demolished in places to make more room for the shopkeepers to sell their produce. In the light of so many unknown parameters there is little reward in constructing the geometry in fine detail. Vertically the mosque was simplified into layers, representing floor and roof levels. Small returns, voids, mouldings, partitions and any other parts deemed non-structural were ignored. The mosque as shown on the plan drawings was divided into four distinct areas, the main body, the cellular structure (including the northwest tower), the minaret and the shops to the north. See figure 1 in Appendix B.

5.2 Element mesh

Once the geometry was prepared, the finite element mesh could be generated. It was decided to model the first three areas, but omit the shops. These were regarded as providing only a little stability to the mosque and it would be slightly conservative to omit them. For the main body and the minaret, solid elements were used to model the large masses of masonry. The cellular structure was more suited to a shell element to model the walls. The mesh was kept fairly coarse, although fine enough to represent the geometry in the required detail. A finer mesh is not justified for the same reasons as keeping the geometry simple, and it is the overall behaviour of the whole mosque that is of interest. It is also worth noting that structures behave more smoothly under dynamic loading and therefore do not need such a fine mesh as a static model[1]. Mesh data is stored under the headings *ELTYPE, *NODES and *ELEMENTS in the NISA file. An isometric view of the element mesh is shown in figure 2 in Appendix B.

5.3 Cracks

Cracks in the model were simulated with non-linear springs. With this element type it is possible to assign different geometric properties in the tension and compression domains. It is not possible to use zero stiffness and infinite stiffness, as is akin to a real crack. These extreme values can cause problems during the solution. Instead the spring stiffness in tension was given a relatively low value and in compression a relatively high value (eg 750 000 times the low value) which is perfectly adequate. See figure 3 in Appendix B. Each crack was made about 100mm wide. This does not affect the behaviour of the crack. It is still 'closed' at 100mm width. However it does make it easier to see the cracks in the model. The non-linear springs span horizontally between and normal to the faces of each crack. Linear springs span vertically and diagonally between the faces. These with an arrangement of coupled displacement data between the nodes model the shear resistance between the two crack faces, the resistance to sliding movement. See figures 4 and 5 in Appendix B. The linear spring constant is defined in the *RCTABLE section of the NISA file. The non-linear spring stiffness curve, described by four points, is given in the *NLSPRING section which is referred to in *RCTABLE. Cracks modelled were all vertical and in the following locations:-

- between minaret and south elevation wall
- between minaret and east elevation wall
- in roof ring beam south side (1 no.)
- in roof ring beam north side (2 no.)
- between interval cross walls and facade in cellular structure (several)

These are annotated in figure 2 in Appendix B.

5.4 Foundations

No quantitative information was provided regarding foundations and soil conditions so an 'exact' model of the ground-structure interaction was not possible. It is true to say however that no foundation is entirely rigid. They all to some extent allow some displacement and rotation. To model this compliance, an extra layer of elements was added to the bottom of the structure to which a low value of Young's modulus was applied, ie a soft cushion-like structure which would deform vertically (squash) under load and allow rotation by having different amounts of vertical deformation. See figure 6 in Appendix B. This enables the base of the walls to rotate freely, which is conservative. Horizontal rigidity was maintained however, because the ground motion is applied directly to the structure. See section 6.3.

5.5 Properties

Geometric and material properties were assigned to each element. Geometric properties are given under the heading *RCTABLE and material properties under *MATERIAL in the NISA file.

5.5.1 Geometric Properties

Four geometric properties have been used as follows.

Property id = 1 gives the thickness of all the shell elements. The shell elements form the walls in the cellular structure. Measuring from the drawings shows the walls to be on average 0.5m thick.

Property id = 2 refers to the id number for the non-linear spring stiffness curve given under the heading *NLSPRING in the NISA file. It also gives the spring constant to be used in linear analysis. This value is used to represent the spring constant for a stitching anchor (10 MN/m^2).

Property id = 3 gives the spring constant for the vertical linear springs used throughout the model. This is very stiff (10 000 MN/m^2) in order to restrain vertical movement (for example, the cracks in the ring beams are actually cracks in arches and thus are restrained from vertical movement by the arch compressive force).

Property id = 4 gives the spring constant for the horizontal linear springs used throughout the model. This is a low value (0.01 MN/m^2) and was used to maintain a relatively low shearing resistance across the cracks in the direction in which ground movements were applied.

5.5.2 Material Properties

Three material properties have been used as follows.

Material id = 1 represents stone. It has a value of Young's modulus of 20 000 MN/m^2 and Poisson's ratio of 0.1. Because this is an isotropic material, these parameters will apply in all three directions. The shear modulus was obtained from the isotropic elasticity relationship. The density value used was approximately 50% that of solid stone. This weighted value was used to take into account the fact that areas modelled as solid volumes do in reality contain voids.

Material id = 2 represents a material with stiffness but zero density. In order to make a suitably rigid connection between shell and solid elements, extra shell elements have to be created. These elements need stiffness, hence a stiffness value is assigned to them, however density is not required and so is made zero so as not to add unnecessary extra mass to that part of the model.

Material id = 3 represents the material that the soft foundation is made from. It is not necessary to have values for a particular soil. The important feature is that it does allow some rotation at the base of the walls.

5.6 Floors and roofs

Floors were omitted originally during the development of this model. It was intended to eventually put them in, but it has been so useful to be able to look inside the model, a different approach was adopted. Floors and roofs with low mass that appeared to be of little structural significance on the drawings and in the photographs have been omitted entirely. This will have negligible effect on the accuracy of the model and if anything, will yield slightly conservative results. Important floors and roofs, for example those in the cellular area which are acting as horizontal diaphragms tying the internal walls together, have instead been modelled with mathematical equations that link the displacements of the walls together. These equations are stored under the heading *CPDISP in the NISA file.

5.7 Miscellaneous data

Sets of numbers relating to node and element id numbers are listed also in the NISA file under *SETS. This enables selective printouts for these nodes and elements only that are of particular interest. An example of this is the request for time history data under *HISTOUT at the end of the file. The variations of displacements and reactions for two sets of nodes are stored in a separate part of the output files and are available for graph plotting in DISPLAY3. The nodes requested are annotated on figure 2 in Appendix B.

Damping (attenuation in amplitude of response over time due to dissipation of energy) is another aspect to consider in transient analysis. It is not important in this case what its exact value is because it is the maximum stresses that are of most interest. Neither is it possible to assign an exact value to the mosque. Energy can be dissipated within it in many ways; as friction between joints, as kinetic energy in flexure etc. However, with some textbook guidance[1], values for the damping coefficients have been estimated as $\alpha = 0$ and $\beta = 0.0032$. These could be included in the NISA file under *PDAMPING in order that the vibrations will eventually die down at the end of the analysis to obtain a fuller picture of the overall vibrational behaviour of the mosque.

6. ANALYSIS

This section describes the thought processes behind the analysis and refers the reader to the appropriate parts of the NISA file.

6.1 Type of analysis

At the beginning of the project there were two options regarding analysis type, modal and direct transient. They are summarised in Table 6.1.

Table 6.1 Dynamic Analysis Types

MODAL		DIRECT TRANSIENT	
Advantages	Disadvantages	Advantages	Disadvantages
Efficient use of CPU time because it calculates forces at resonance points only	Two stage analysis - eigenvalue analysis followed by shock spectrum analysis	Non-linear model possible	Longer time to process model because it solves the equations of motion directly
Easy to input envelope of design frequencies and amplitudes	Not possible to use non-linear model		
	Possible to miss failure points		

Modal dynamic analysis is based on the superposition of responses calculated for a selection of natural frequencies for a given structure. Using this technique the accurate prediction of dynamic responses is only possible if the appropriate natural modes of vibration have been selected. It is also important that the modes of vibration have similar frequencies to that of the load. A modal dynamic analysis is therefore a twofold process. First an eigenvalue analysis is conducted to determine the eigenvalues (natural frequencies) and eigenvectors (modal shapes) of the structure. Then a shock spectrum analysis is performed using a frequency spectrum as the ground motion input. A frequency spectrum is a graph of amplitude of response versus frequency. This amplitude can be displacement, velocity or acceleration. Some typical shock spectra for an earthquake are given in figure 7 in Appendix B. The model is excited at each natural frequency by the corresponding amplitude response. Structural responses for each mode are calculated separately before combining components with one of several well verified combination procedures.

It is an efficient method of dynamic analysis because it concentrates on the vulnerable aspects of the structure. Such a method was invaluable when computer processing time was at a premium. However, the drawbacks with this type of selective analysis is that some failures not associated with natural modes and frequencies of vibration may be missed. It is also not possible to model non-linear behaviour, which is a key feature in this project.

Preliminary eigenvalue analyses produced relatively high natural frequencies for the main body of the mosque and moderate frequencies for the minaret and cellular structure. These frequencies are rare in a 'typical' earthquake, therefore most of the movement of the mosque is rigid body rather than modal deformation. The forces generated in the model are more from its inertia being given an acceleration rather than those induced by deformation. The decision was then taken to concentrate on direct transient analysis. The modal approach has revealed much useful information regarding the dynamic behaviour of the mosque. It located areas most vulnerable to vibration and by animating the modal shapes using DISPLAY3 it was possible to see how the structure was deflecting.

6.2 Ground motion input

Real earthquakes are three-dimensional phenomena; they have translations in X, Y and Z directions and rotations about the X, Y and Z axes. All earthquakes have different magnitudes of these components and at different points in time. Some typical amplitude time strong motion recordings are given in figure 8 in Appendix B. A simplified ground input was needed which would cover any reasonably expected magnitude of shock wave, from any direction; in effect an envelope of ground motions. It has been seen from recorded data that vertical components of motion are small in comparison with the horizontal components. It was therefore deemed acceptable to limit our input to horizontal motion by translations in the X and Y directions. These orthogonal directions correspond approximately to the principal axes of the building and are likely to yield the most severe responses. The same displacement-time curve was applied in each direction. See figure 10 in Appendix B. This curve is defined as a series of coordinates in the *TIMEAMP section of the NISA file and was based on the strong motion recording in figure 8. The first peak of the wave corresponds to an acceleration of approximately 0.5g which is much higher than most earthquake design code requirements.

6.3 Support conditions

Defining the right boundary conditions is essential to the analysis. The way the mosque is supported by the ground needs to be simplified and broken down into its constituent components. Support from adjacent buildings was ignored. For all analyses, the whole building was supported vertically (UZ = 0) at each node at the base of the soft layer. The X and Y boundary conditions however were dependent on the direction of ground motion. For Y input, nodes at the top and bottom of the soft layer were given specified displacements of UY = 1.0 and UX = 0.0, and conversely for input in the X direction. See figure 11 in Appendix B. This data is stored in the NISA file under *SPDISP. The specified displacement of 1.0 in the Y direction is also tagged with a time-amplitude curve id = 101, described under *TIMEAMP in the NISA file, which means that at any particular time t in the analysis, displacement Y will have a value equal to 1.0 x the amplitude of the time-amplitude curve at time t. Such a varying of Y displacement with time in effect 'shakes' the base of the mosque.

6.4 Gravity

Gravity is included in the analysis to pre-load the model before the horizontal movement was applied to it, which is what happens in reality. Gravity is defined in the NISA file as a pressure in the negative Z direction applied to the bottom face of each element in the *PRESSURE section of the NISA file. Gravity is a static load in this case, hence the load is applied as a ramp over a ten second time interval, a long enough time step to avoid any unwanted dynamic effects. This ramp is defined in timeamp curve id = 100. See figure 9 in Appendix B.

6.5 Stages of analysis

Because two different types of load are being applied to the model, the analysis has to be broken down into two stages called events. Each event is then broken down into one or more steps depending on the type of load being applied and the nature of the model. Event 1 is gravity loading. It was applied in only one step as it is a simple load and it will not have too much effect on the model i.e. no huge stresses or strains. Event 2 is the horizontal dynamic load produced by the earthquake. This was applied in 200 steps for two reasons. Firstly, the number of steps is related to the rate of sampling. It is necessary to sample a waveform at twice its frequency to obtain even its approximate behaviour. Text books[1] on this subject recommend that the load step size is less than or equal to 1/20 the period of the highest frequency to be sampled. Secondly, the model is more sensitive to this input and therefore needs to deal with the motion in small amounts to get an economic convergence of solution for non-linear effects.

6.6 Summary of analyses

A summary of the analyses is given in Table 6.2. To reiterate the brief:-

1. To reasonably predict the behaviour of the currently unrepaired mosque under further earthquake loading, this necessitates a cracked model with non-linear springs, hence a non-linear analysis. The NISA file will contain the instructions ANALYSIS = NLTRAN and NLTYPE = GEOMETRY.

2. To assess the behaviour of the repaired mosque and the loads in the stitching anchors forming the repair system, this uses the same cracked model, but with linear springs, hence linear analysis. The NISA file will be altered to show NLTYPE = LINEAR.

Table 6.2 Summary of Analysis

	Ground motion Y direction	Ground motion X direction
Unrepaired model - NLTRAN, GEOMETRY	AL29	AL30
Repaired model - NLTRAN, LINEAR	AL31	AL32

7. RESULTS

The discussion of results will be limited to the first 75 load steps i.e. up to and just past the first peak of the earthquake input. Envelopes of the maximum tensile stress at each node that occurred over this time period were processed and displayed as stress contour plots in Appendix B.

7.1 Unrepaired model

Figures 12 to 14 in Appendix B are stress plots of various parts of AL29.

7.2 Repaired model

Figures 15 to 17 in Appendix B are stress plots of various parts of AL31, for comparison with the plots of AL29.

8. CONCLUSIONS

This analysis can only be as accurate as the input data given. Many parameters such as material properties and earthquake loads are a result of engineering judgement rather than physical measurement or established facts. Despite these limitations the model is exhibiting understandable behaviour and much qualitative information can be obtained.

The following conclusions have been drawn from the finite element analysis carried out on Al-Ghuri mosque:-

1. The analysis confirms the general arrangement and locations of the stitch anchors.

2. The simulation of the structure under lateral earthquake loading has been used to successfully predict its structural behaviour in both its 'current' condition and 'repaired' condition.

3. Two distinct types of behaviour have been identified:-

   (i) Flexing of tower at frequencies similar to those in earthquakes but not resonant.

   (ii) Almost rigid body movement of all low rise components.

4. The axial forces required to secure the tower to the rest of the building could not be realistically resisted by any horizontal anchor system. Even if they could, the probable result would be to move the failure zone to another part of the building. Hence, the analysis confers with the designed system to not attempt to join the two parts together.

5. The cracks between the minaret and mosque should not be rigidly tied because the vibrational behaviours of the two parts are so different, there would be a detrimental pounding action. Whether the minaret is tied or not, the mosque will still suffer the huge compressive forces as the minaret knocks against it. However the tensile forces induced in the mosque as the minaret then rebounds will be eliminated.

6. Loads in the proposed anchors excluding those associated with the tower have been predicted and are similar in magnitude to those used for design.

7. Simulation of the minaret/building interplane in the 'current' condition suggests that there could be some benefit in providing horizontal shear connection. This will help support relatively narrow walls which have no other support than lightweight timber roofs.

9. FURTHER WORK

This analysis has been worthwhile in its own right, as summarised in the conclusions to this report. But in addition to this, it has provided a basis for further work in this field. There has been a distinct learning curve, but much knowledge has been acquired along the way in the fields of modelling cracked masonry buildings using finite elements and seismic loading. Analyses of further buildings could certainly benefit from this project.

Representing masonry as a continuous medium is an adequate approach, but there is scope for improvement if the real nature of masonry could be modelled more accurately. Masonry units such as blocks, bricks and stones are bonded together with mortar. The mortar is usually weak in comparison to the masonry units so ultimately the masonry acts as a stack of discrete elements rather than finite elements in a continuous medium which is the only element type currently possible with NISA software.

The software company Rockfield, within their general modelling package ELFEN, have developed a program DEFT which can model and analyse a problem using discrete elements. Although commercially available, it is still in its infancy and offers only 2D elements. Collaboration in this field between Gifford and Rockfield is a possibility in order to develop the 3D analysis.

The shaking table tests carried out at Southampton University in August this year can also assist in these studies. In the long term, analysis by computer is more cost effective than real life tests. In the short term, simple tests can be carried out and the results used to calibrate the software. Some of the tests conducted recently may be suitable for calibrating NISA or DEFT.

10. REFERENCES

1. Smith J W 'Vibration of Structures, Applications in civil engineering design' Chapman and Hall

2. Newmark/Rosenbluth 'Fundamentals of Earthquake Engineering' Prentice-Hall

3. 'The Northridge (California) Earthquake of 17 January 1994: Observations, Strong Motion and Correlative Response Analysis' ESEE Research Report No. 94/4 June 1994

entities such a:

APPENDIX A

NISA file

This is a printout of AL29.NIS. Files for AL30-32 are similar. Repetitive entities such as node coordinates and element connectivities have been partly omitted.

```
ANALYSIS = NLTRAN
FILE = AL29
SAVE=26,27,37
MAXCPU TIME = 0,0
NLTYPE = GEOMETRY
*TITLE
 NON-LINEAR DIRECT TRANSIENT ANALYSIS OF CRACKED MOSQUE WITH SOFT FOUNDS
 EVENT 1 - GRAVITY LOADING SIMULATED BY PRESSURE AND CFORCE LOADING
 EVENT 2 - HORIZONTAL HALF SINE PULSE DYNAMIC DISPLACEMENT IN Y-Y DIRECTION
*ELTYPE
  1,   4,      1
  2,   4,     10
  3,  20,      1
  4,   4,     20
  5,  17,      1
*RCTABLE
     1,      4,1,      0
5.0000000E-01, 5.0000000E-01, 5.0000000E-01, 5.0000000E-01,
     2,      7,1,      0
1.0000000E+01, 0.0000000E+00, 0.0000000E+00, 0.0000000E+00,
0.0000000E+00, 1.0200000E+02, 0.0000000E+00,
     3,      7,1,      0
1.0000000E+04, 0.0000000E+00, 0.0000000E+00, 0.0000000E+00,
0.0000000E+00, 0.0000000E+00, 0.0000000E+00,
     4,      7,1,      0
9.9999998E-03, 0.0000000E+00, 0.0000000E+00, 0.0000000E+00,
0.0000000E+00, 0.0000000E+00, 0.0000000E+00,
*NODES
     1,,,, 1.16000E+01, 3.80000E+00, 1.44000E+01,      0
     2,,,, 1.36500E+01, 3.80000E+00, 1.44000E+01,      0
     .
     .
     .
  6885,,,, 1.34333E+01, 1.97000E+01, 2.16000E+01,      0
  6886,,,, 1.19098E+01, 1.97000E+01, 2.16000E+01,      0
*ELEMENTS
     2,      1,      1,      1,      0
     2,      3,     10,      9,     16,     17,     24,     23,
     3,      1,      1,      1,      0
     3,      4,     11,     10,     17,     18,     25,     24,
     .
     .
     .
  2285,      1,      5,      2,      0
  5581,   6886,
  2286,      1,      5,      2,      0
  5580,   6885,
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```
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EZ   ,      1,0, 2.00000E+04,
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NUXZ,       1,0, 1.00000E-01,
NUYZ,       1,0, 1.00000E-01,
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GXZ ,       1,0, 9.09100E+03,
GYZ ,       1,0, 9.09100E+03,
DENS,       1,0, 1.00000E-03,
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EY   ,      2,0, 2.00000E+04,
EZ   ,      2,0, 2.00000E+04,
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NUXZ,       2,0, 1.00000E-01,
NUYZ,       2,0, 1.00000E-01,
GXY ,       2,0, 9.09100E+03,
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APPENDIX B

Figures

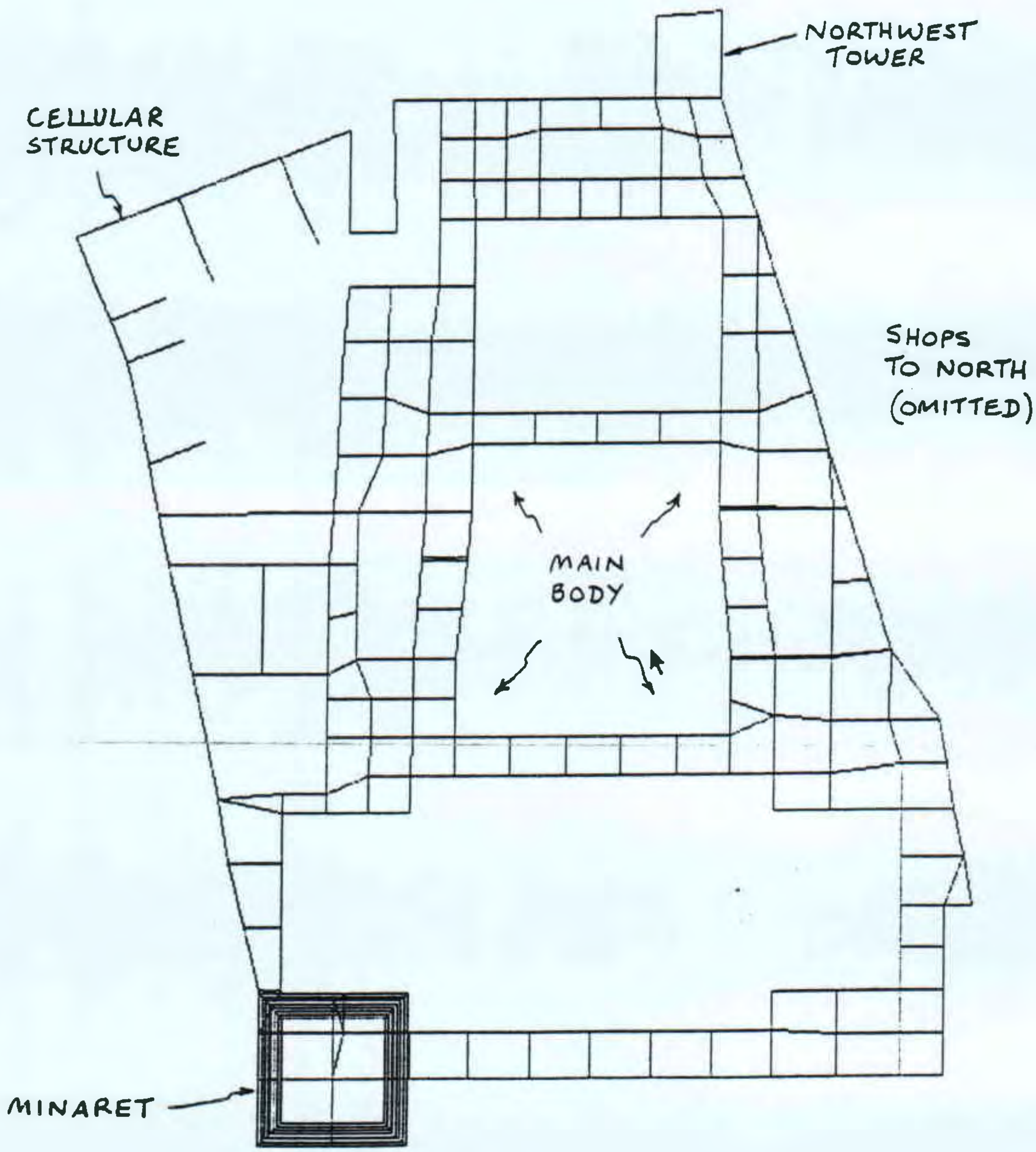

Figure 1 Plan geometry of mosque

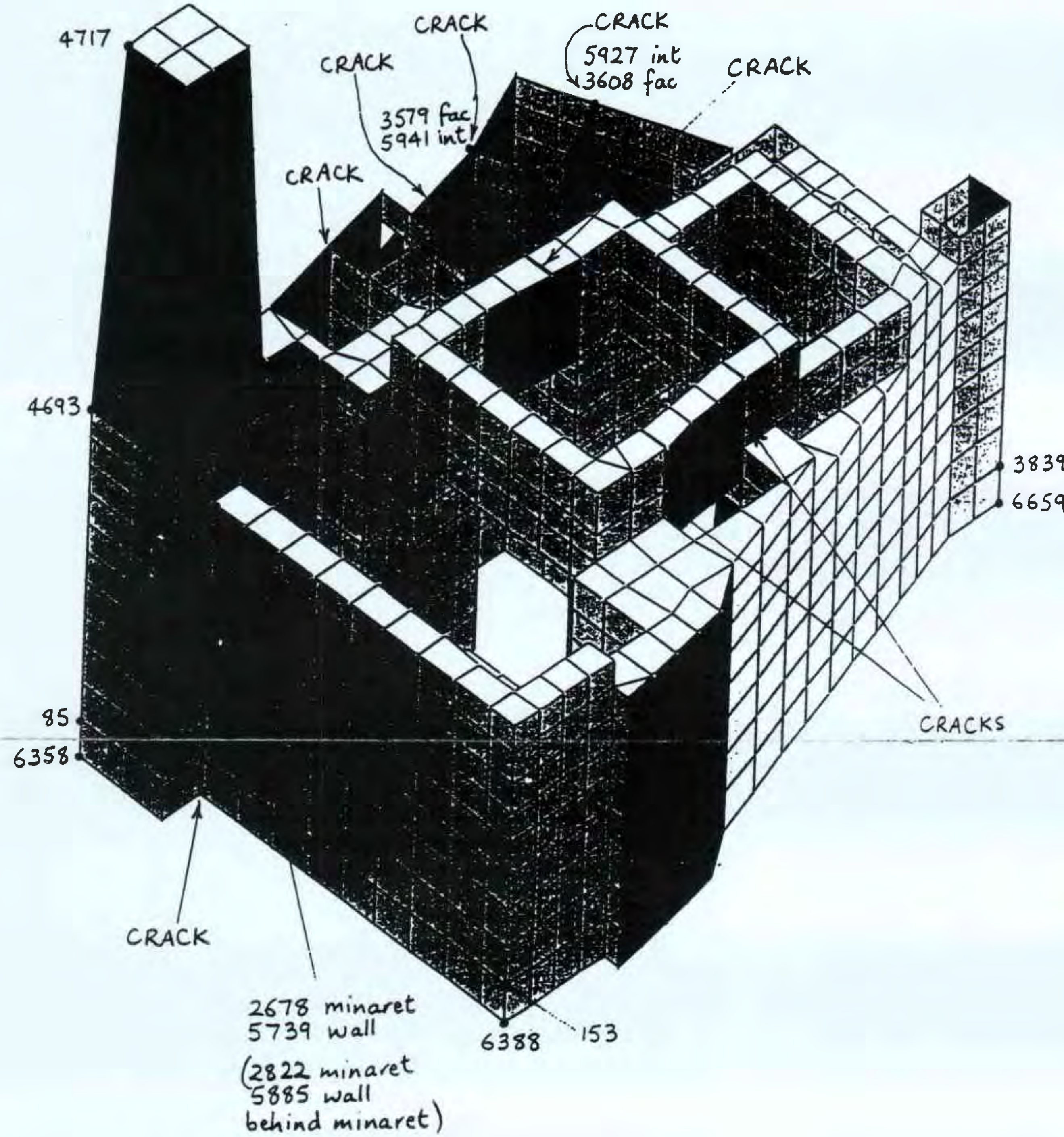

Figure 2 Isometric view of element mesh

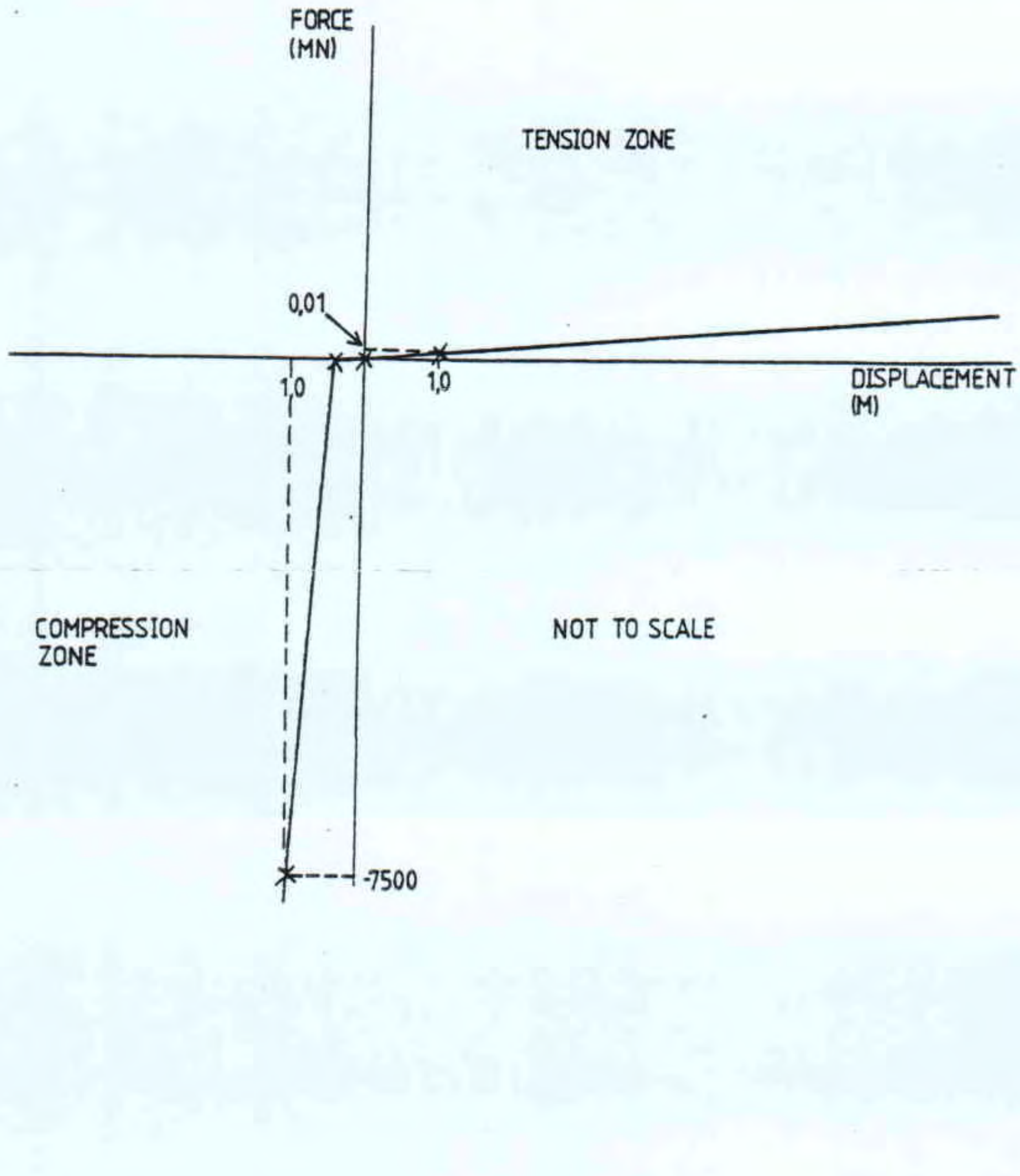

Figure 3 Loading curve for non-linear spring elements

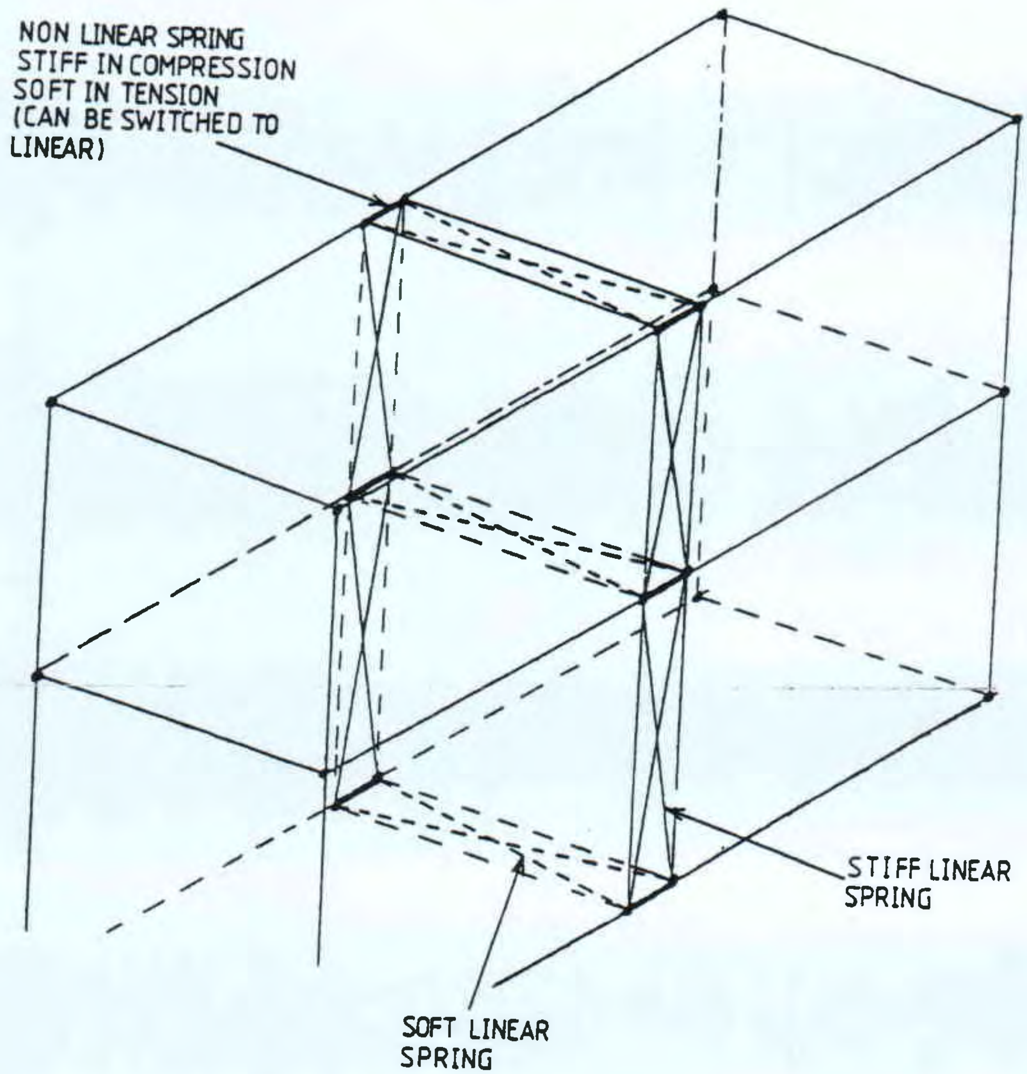

Figure 4 Arrangement of springs across gap to model crack behaviour

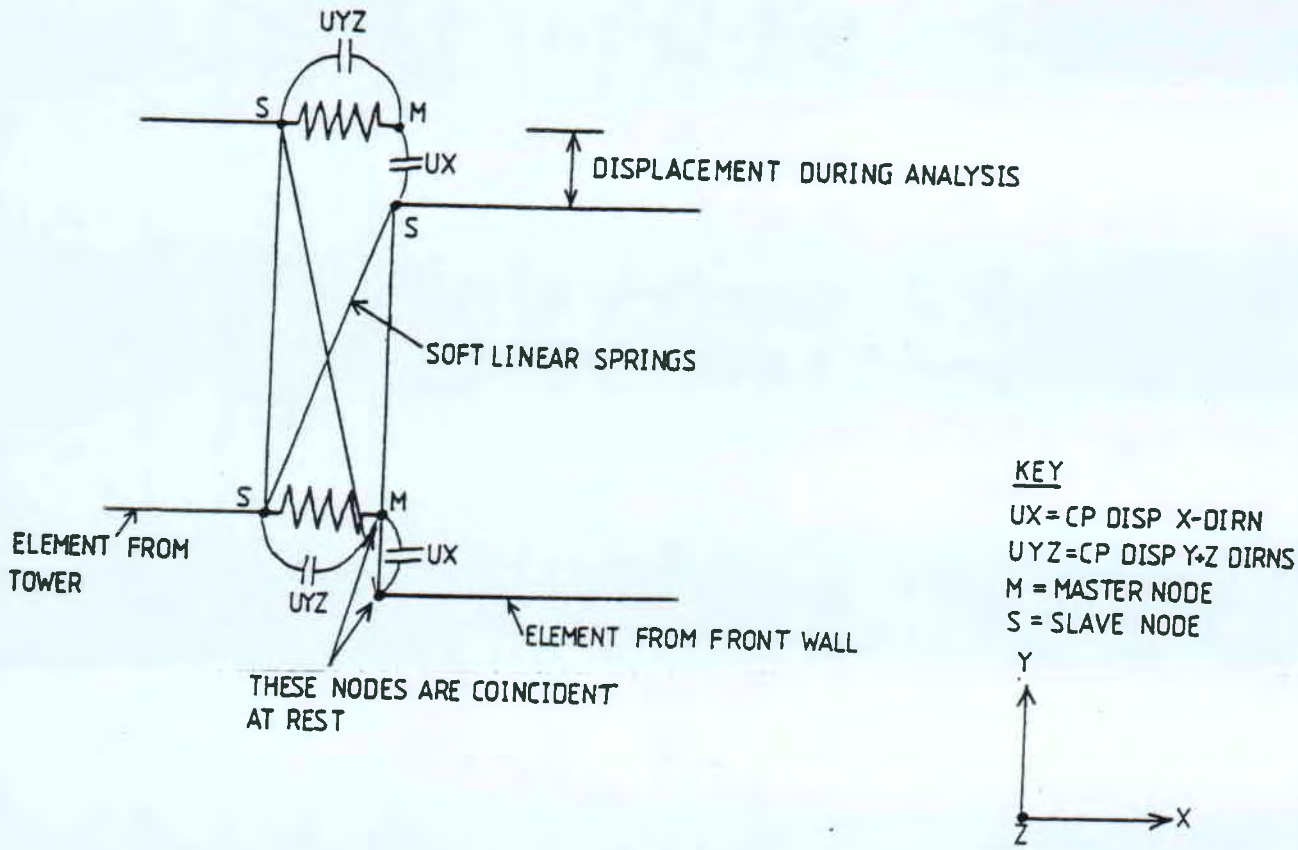

Figure 5 Arrangement of coupled displacements to improve crack behaviour

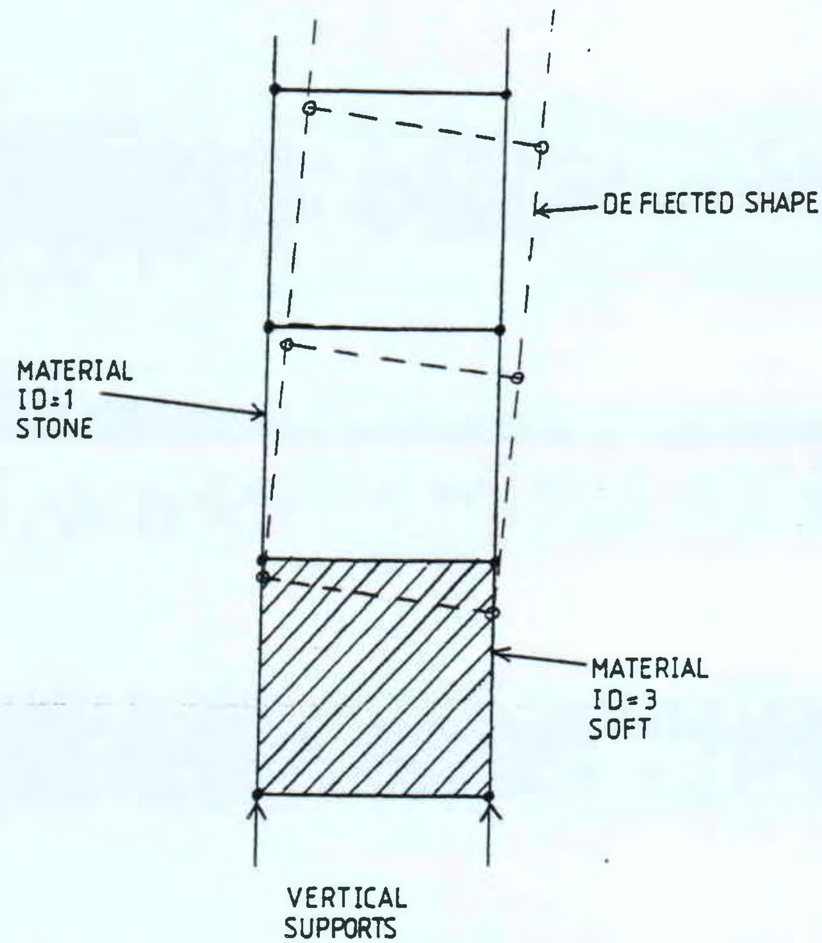

Figure 6 Rotation at base of wall due to soft foundation

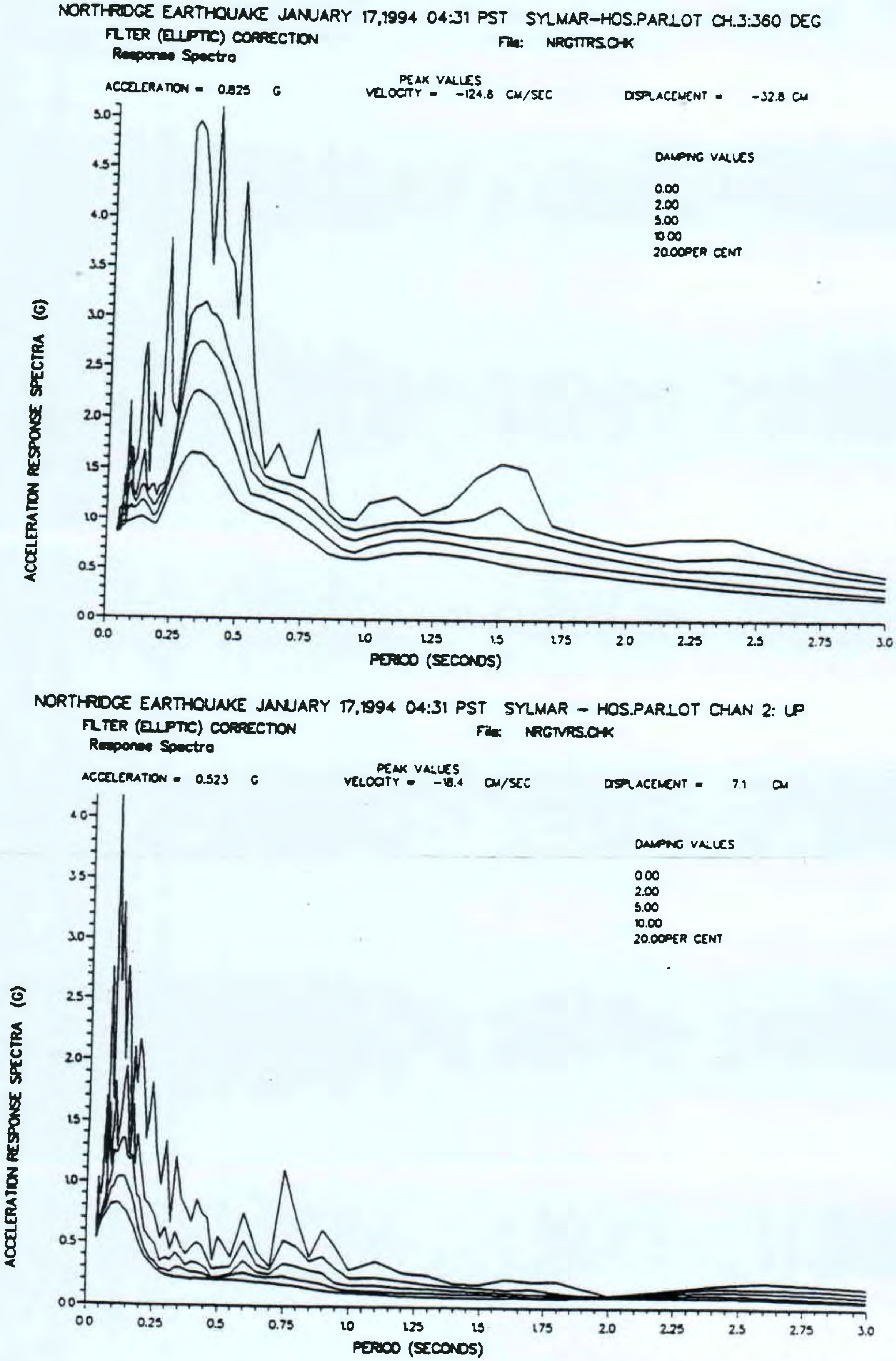

Figure 2.5: Horizontal and Vertical Acceleration Response Spectra - Sylmar

Figure 7 Typical shock spectra for an earthquake

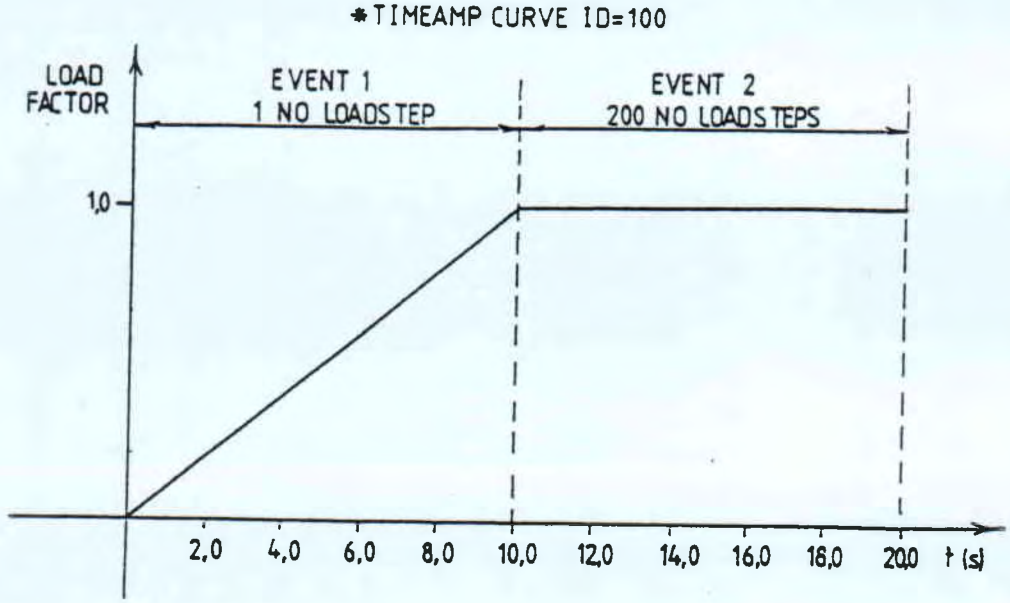

Figure 9 Amplitude-time curve for gravity loading

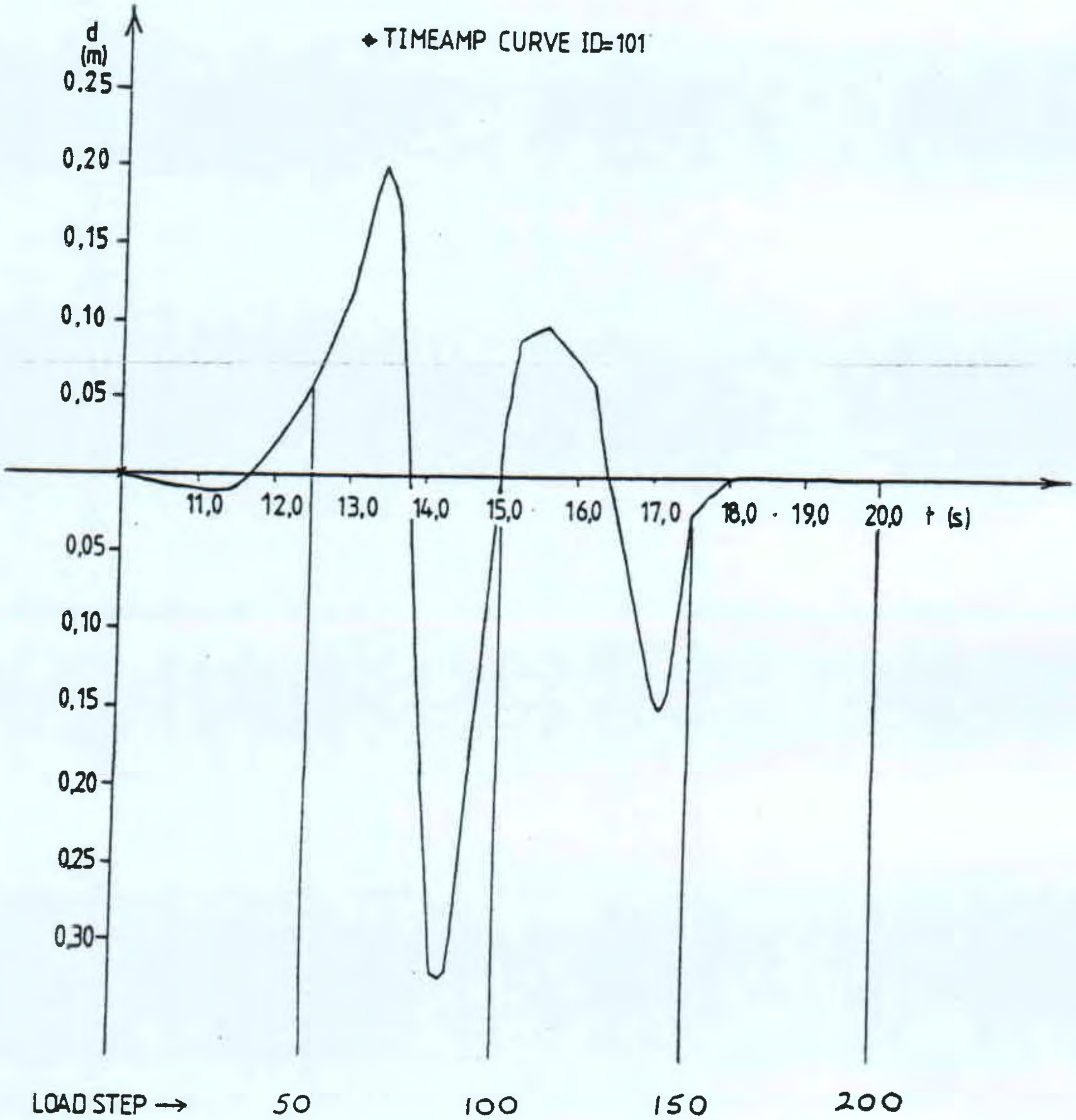

Figure 10 Amplitude-time curve for earthquake loading

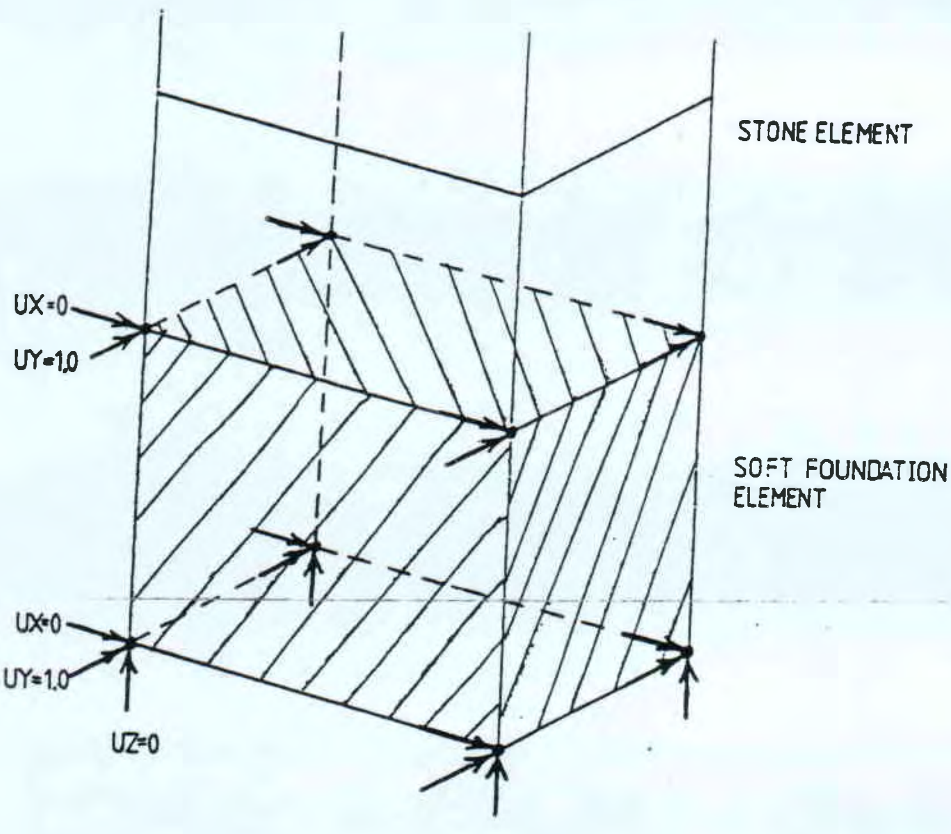

Figure 11 Arrangement of vertical and horizontal specified displacement data

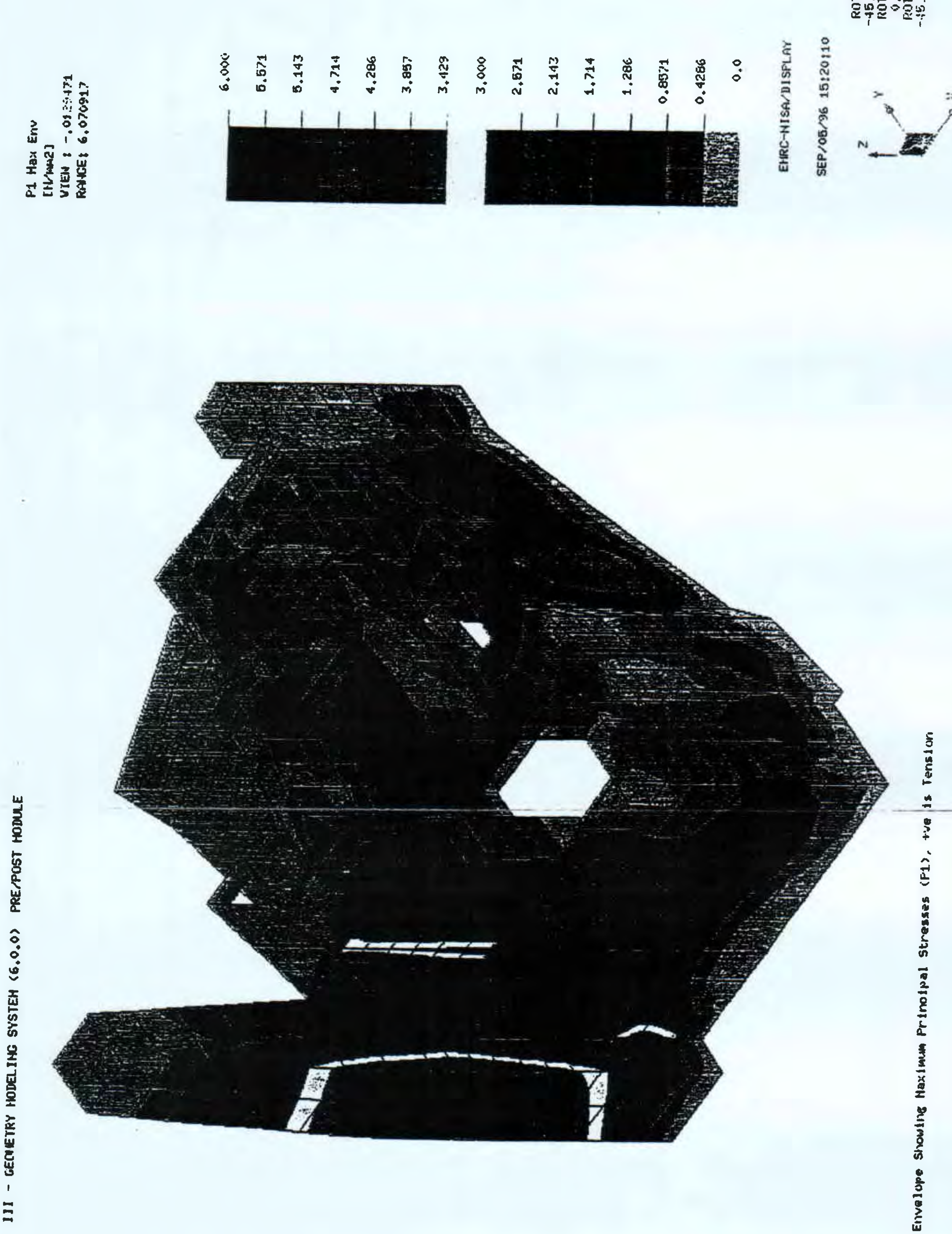

Figure 12 AL29 - view of whole mosque

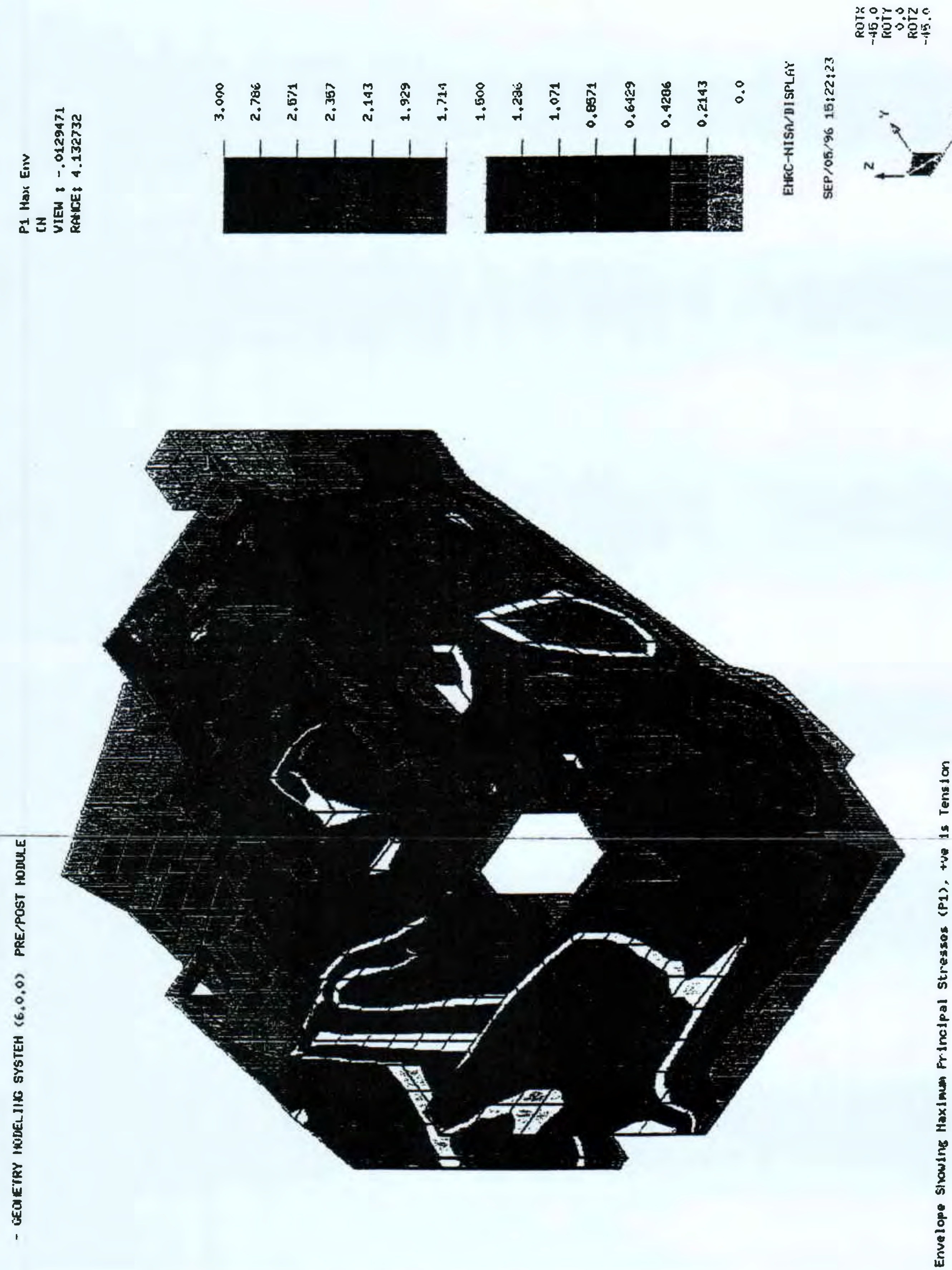

Figure 13 AL29 - view of mosque minus minaret

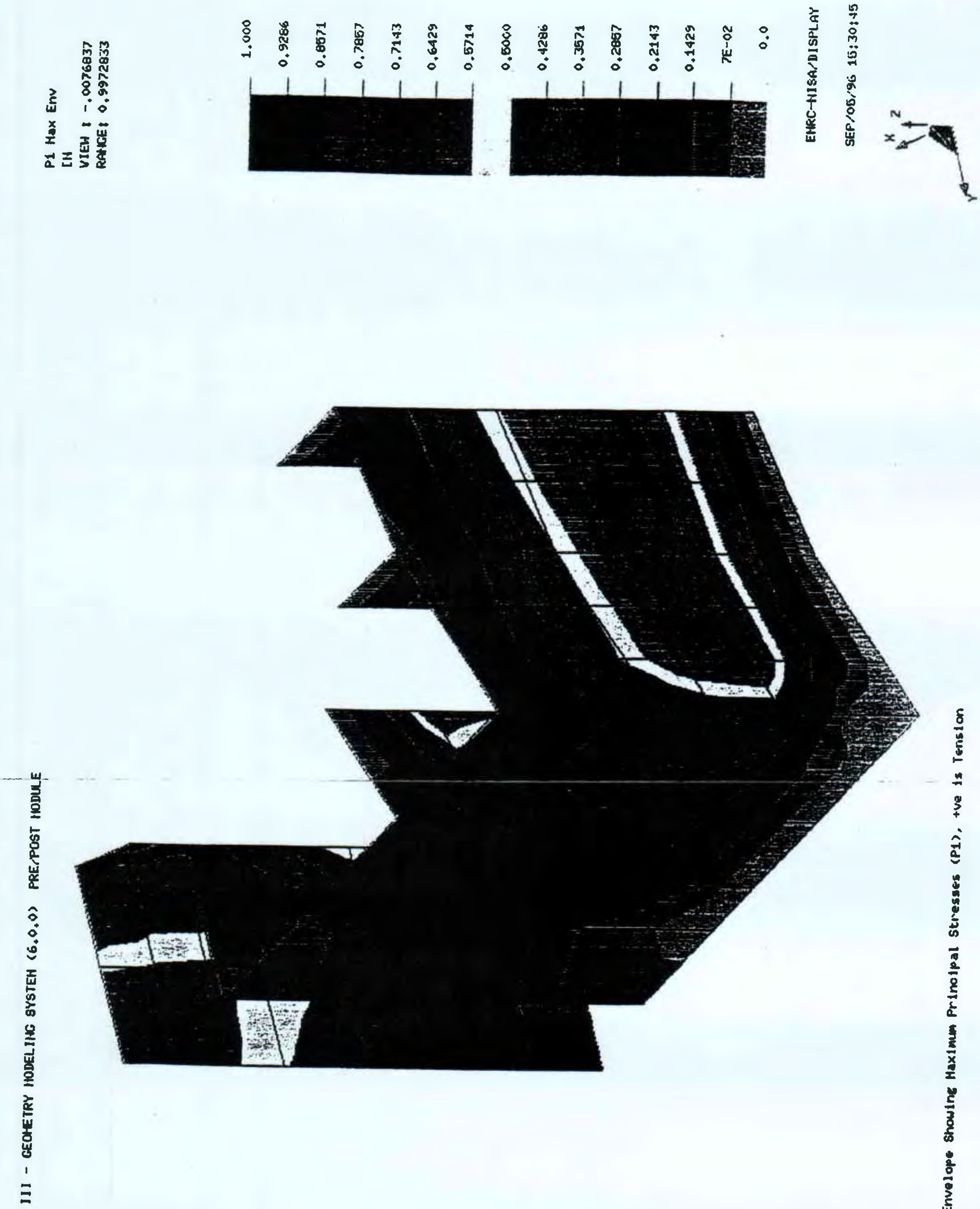

Figure 14 AL29 - view of cellular structure only

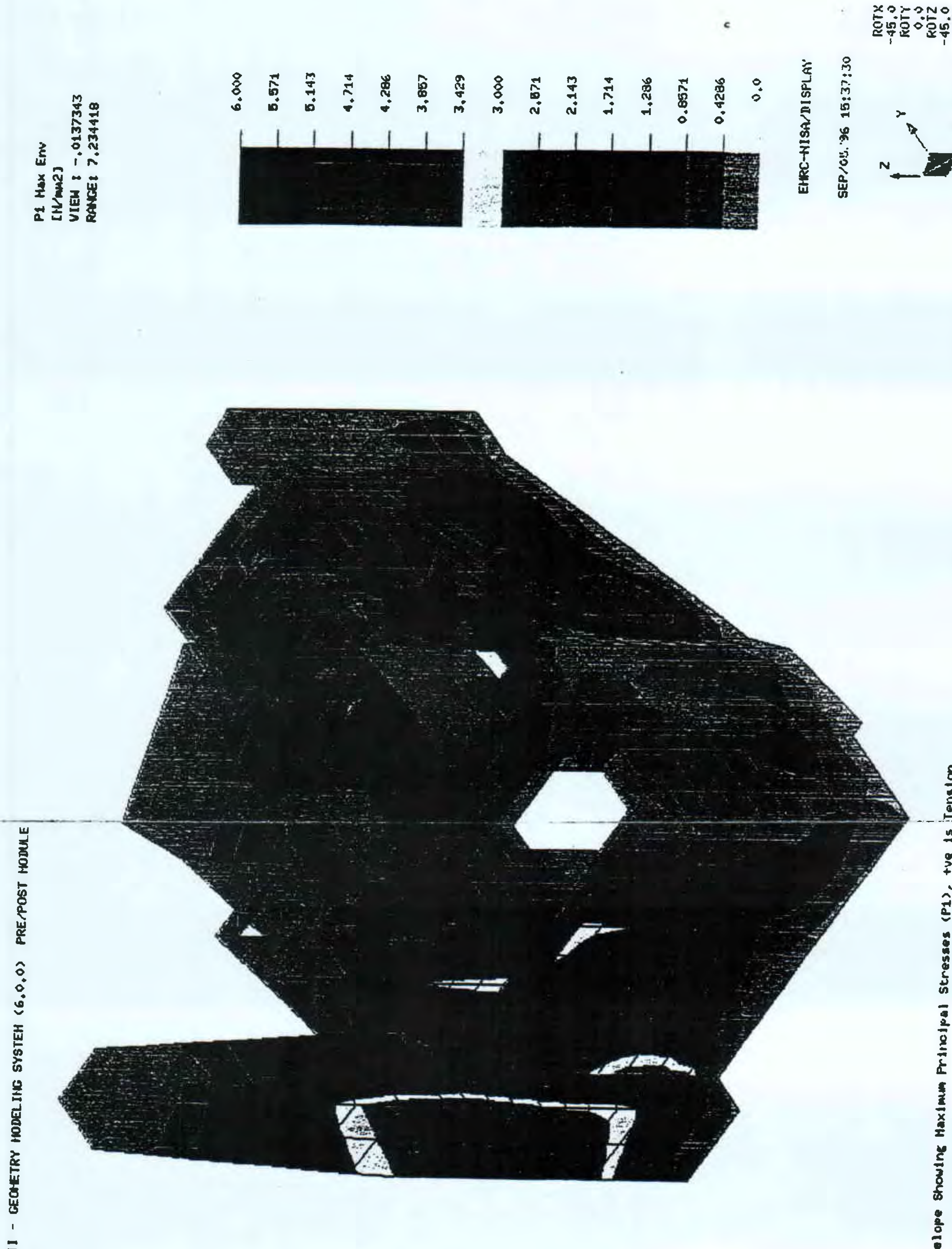

Figure 15 AL31 - view of whole mosque

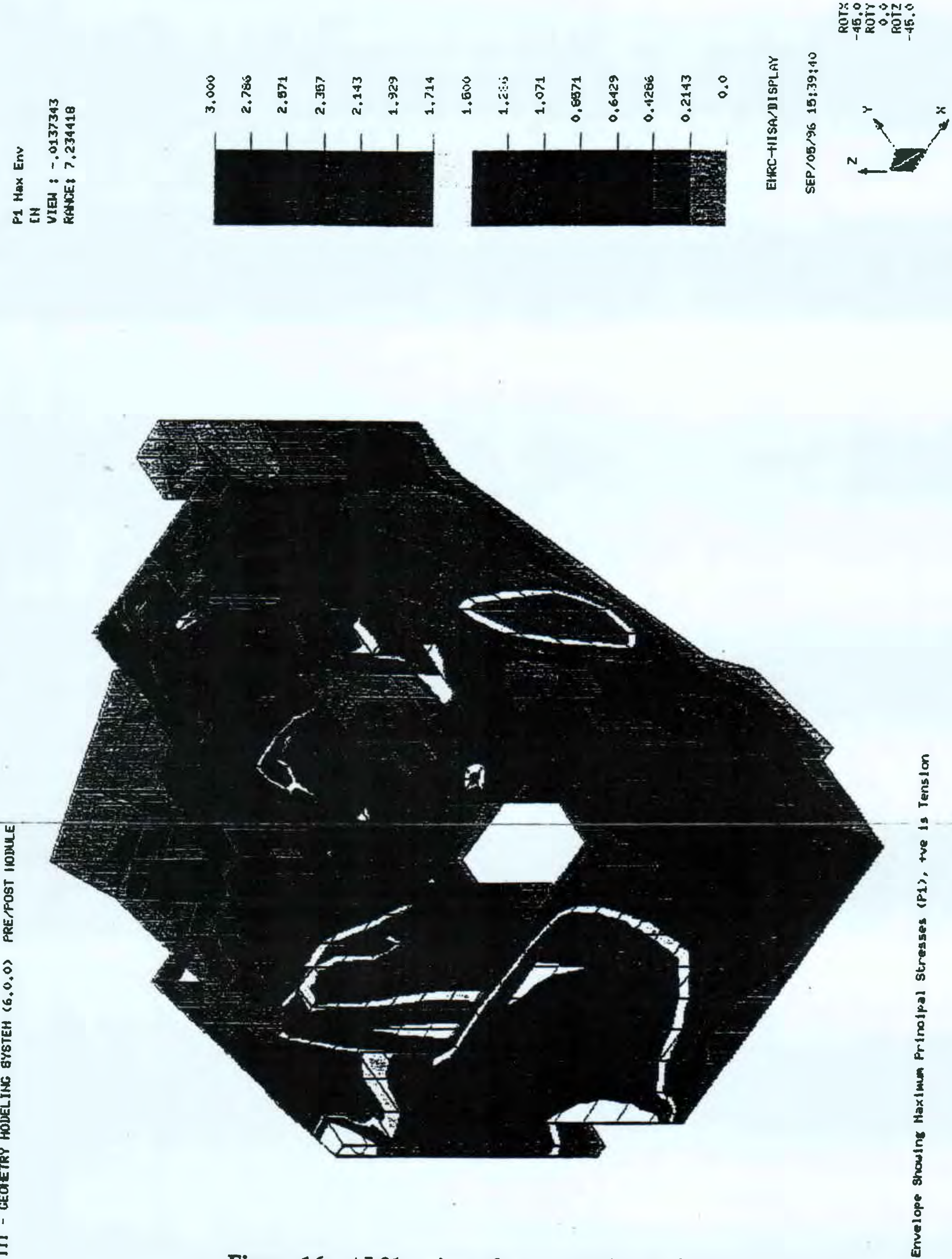

Figure 16 AL31 - view of mosque minus minaret

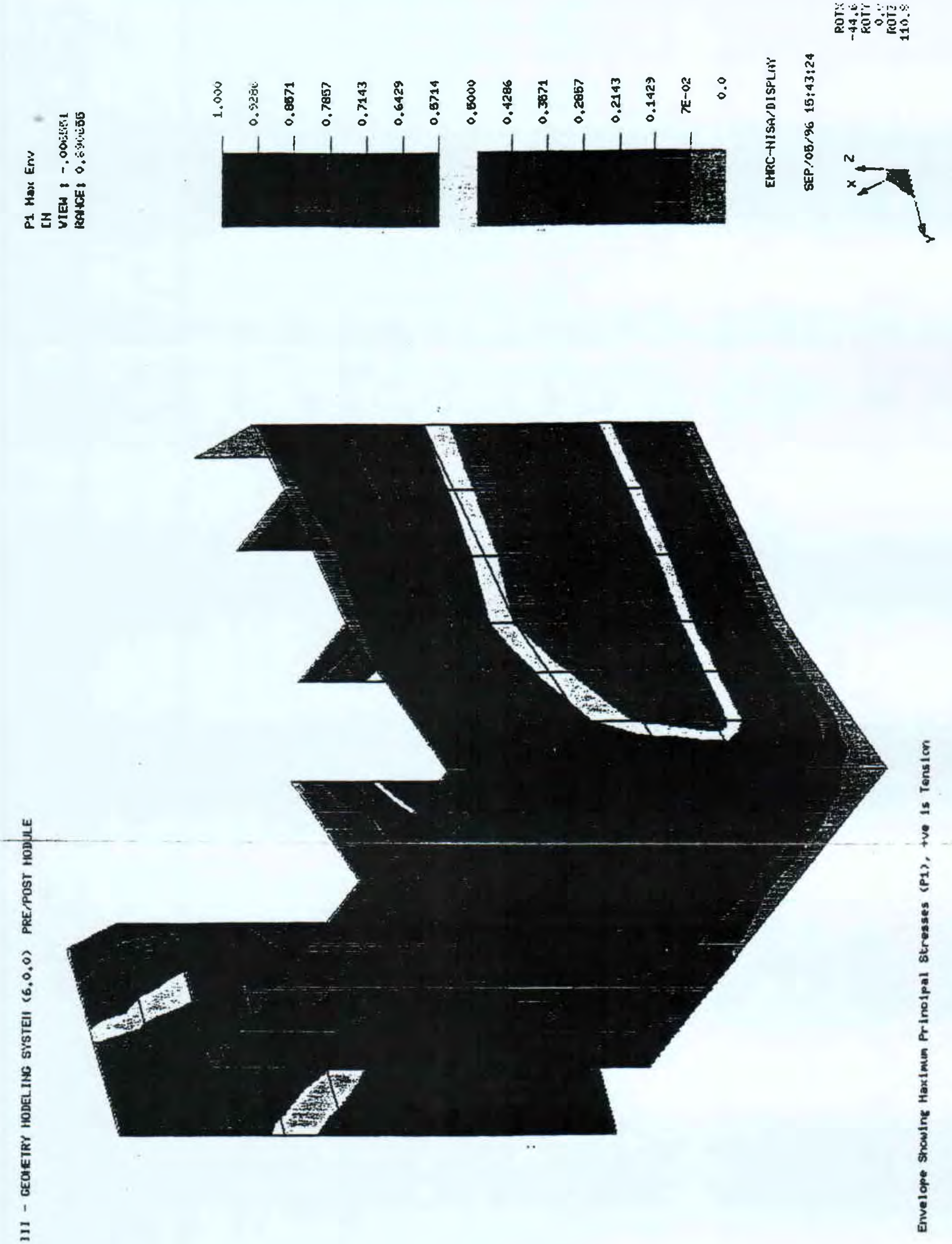

Figure 17 AL31 - view of cellular structure only

QUESTIONS AND ANSWERS

1. **Why have you simplified the structure of the mosque so much?**

   Please refer to sections 5.1 and 5.2 in the report.

2. **Why have you not used 'accurate' input information?**

   Please refer to sections 5.1 and 5.2 in the report.

3. **Why have you not considered foundations in more detail?**

   An exact model of the ground-structure interaction is not necessary because this is a behavioural and comparative analysis. The mosque is sitting on a soft layer of elements to enable some rotation at the base of the walls to avoid locked-in stresses. However, all translational movement is applied directly to the mosque, not through the foundation. See figure 11 in Appendix B of the report. This is what would occur if the foundations were very rigid (which may be the case now the mosque has been underpinned) and is a conservative approach. Please refer to sections 5.4 and 6.3 in the report.

4. **Why did you not use the Cairo earthquake?**

   There is no guarantee that a similar earthquake will occur again. Earthquakes are so variable that when you are designing for them it is better to use a standard design load. For example in design codes, structures are expected to withstand statically a certain value of acceleration. For now however, because we are only looking at behaviour, it is sufficient to use a section of another earthquake's displacement-time curve. Please refer to sections 3.0 and 6.2 in the report.

University
of Southampton

HFRU 96/39

Faculty of Engineering and Applied Science

Institute of Sound and Vibration Research

SUMMARY OF RESULTS FROM HORIZONTAL SHAKING TABLE TESTS ON WALLS

Report submitted to:

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Ringwood Road
Woodlands
Southampton SO40 7HT

Report Prepared by:

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Human Factors Research Unit
Institute of Sound and Vibration Research
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Southampton SO17 1BJ

August 1996

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Table 2 Description of test walls.

Wall	Description
1	No ties, straight joint
2	Ties, straight joint
3	No ties, bonded joint
4	Ties, bonded joint
5	Dry laid, parallel to motion
6	Dry laid, perpendicular to motion
7	Box test, Thermalite 24mm tie
8	Box test, Thermalite 40mm tie
9	Box test, Thermalite 55mm tie
10	Box test, dense brick 24mm tie

4. Results

4.1 Wall 1 - No ties, straight joint

The transmissibility of wall 1 is shown in Figure 2. The transmissibility resonance frequency was 21.9 Hz with a magnitude of 7.8. Acceleration time histories at the base and top of the wall for the final condition are shown in Figure 3. The tests are described in Table 3.

Table 3 Tests carried out on wall 1.

Test	Stimulus	r.m.s. acceleration (ms^{-2})	Peak acceleration (ms^{-2})	Observation
1	1	0.08	0.31	-
2	2	0.06	0.14	-
3	2	0.10	0.29	-
4	2	0.21	0.51	-
5	2	0.42	1.00	-
6	2	0.86	2.21	-
7	2	1.10	2.83	-
8	2	1.10	2.85	-
9	2	2.19	5.36	-
10	2	2.23	10.41	Failure at wall interface

4.5 Wall 5 - Dry laid, parallel to motion

Acceleration time histories at the base and top of wall 5 for the final condition are shown in Figure 10. The tests are described in Table 7.

Table 7 Tests carried out on wall 5.

Test	Stimulus	r.m.s. acceleration (ms^{-2})	Peak acceleration (ms^{-2})	Observation
1	5	0.32	0.65	-
2	5	0.66	1.23	-
3	5	1.21	2.32	-
4	5	1.69	3.03	Wall shearing
5	5	2.14	3.90	Wall shearing
6	5	2.52	4.66	Wall shearing
7	5	2.85	5.52	Wall shearing
8	5	3.09	5.97	Wall shearing
9	5	3.35	6.43	Wall shearing
10	5	3.21	6.68	Wall shearing
11	5	3.22	7.46	Total failure

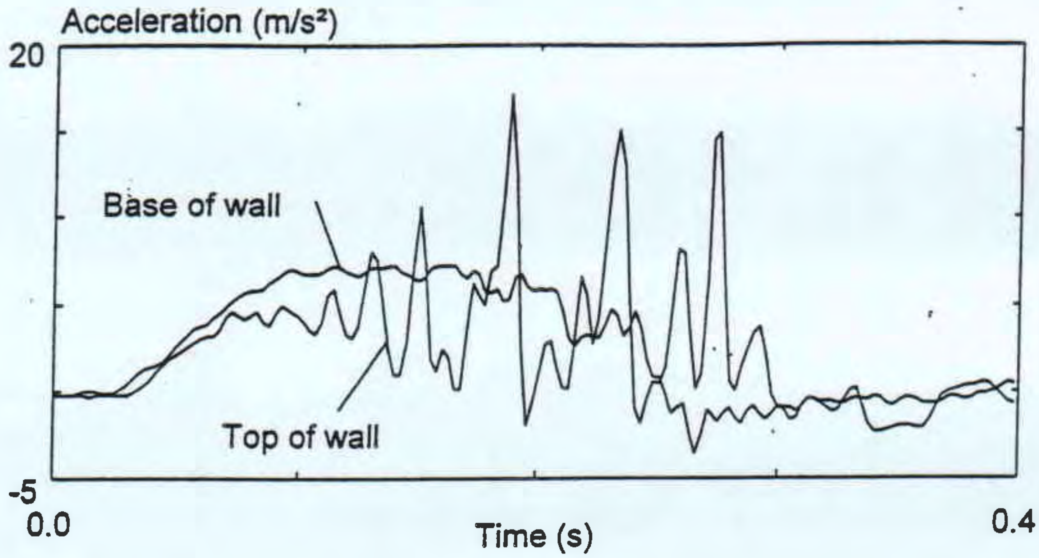

Figure 10 Acceleration time history measured at base and top of wall 5 for greatest excitation magnitude.

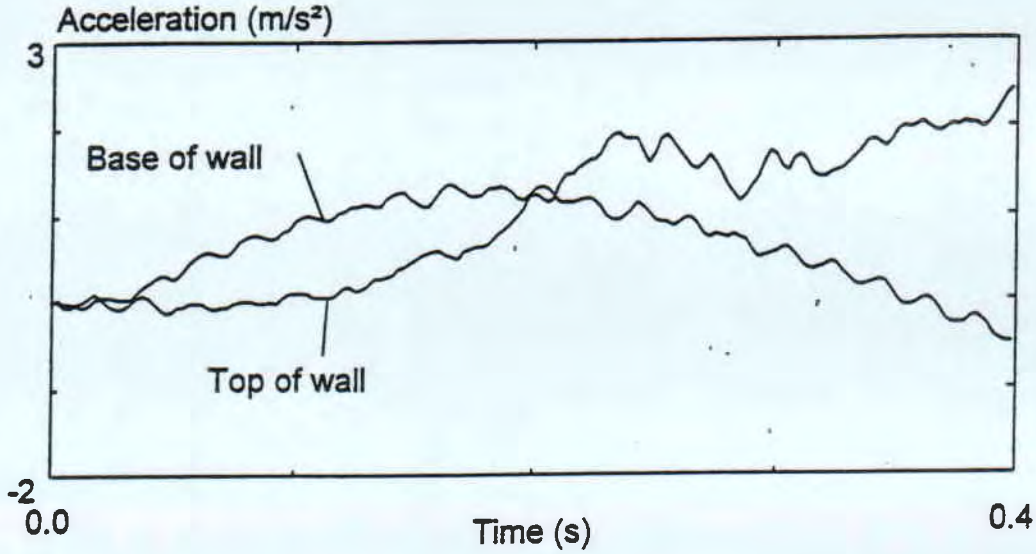

Figure 11 Acceleration time history measured at base and top of wall 6 for greatest excitation magnitude.

4.7 Box test 1 - Thermalite 24mm tie

Acceleration time histories at the base and top of the box containing the 24mm Thermalite tie for the final condition are shown in Figure 12. The tests are described in Table 9.

Table 9 Tests carried out on 24mm tied Thermalite brick.

Test	Stimulus	r.m.s. acceleration (ms^{-2})	Peak acceleration (ms^{-2})	Observation
1	1	0.06	1.21	-
2	3	0.22	0.56	-
3	3	0.44	1.00	-
4	3	0.85	1.96	-
5	3	1.16	2.74	-
6	3	2.12	5.10	-
7	3	2.83	7.02	-
8	3	4.02	11.14	Initial cracks
9	3	5.19	14.19	Initial cracks
10	3	6.08	15.09	Total failure

4.8 Box test 2 - Thermalite 40mm tie

Acceleration time histories at the base and top of the box containing the 40mm Thermalite tie for the final condition are shown in Figure 13. The tests are described in Table 10.

Table 10 Tests carried out on 40mm tied Thermalite brick.

Test	Stimulus	r.m.s. acceleration (ms^{-2})	Peak acceleration (ms^{-2})	Observation
1	1	0.04	3.14	-
2	3	1.16	2.74	-
3	3	2.15	5.38	-
4	3	3.10	7.59	-
5	3	3.99	9.79	Initial cracks
6	3	5.09	12.32	Initial cracks
7	3	6.19	16.14	Total failure

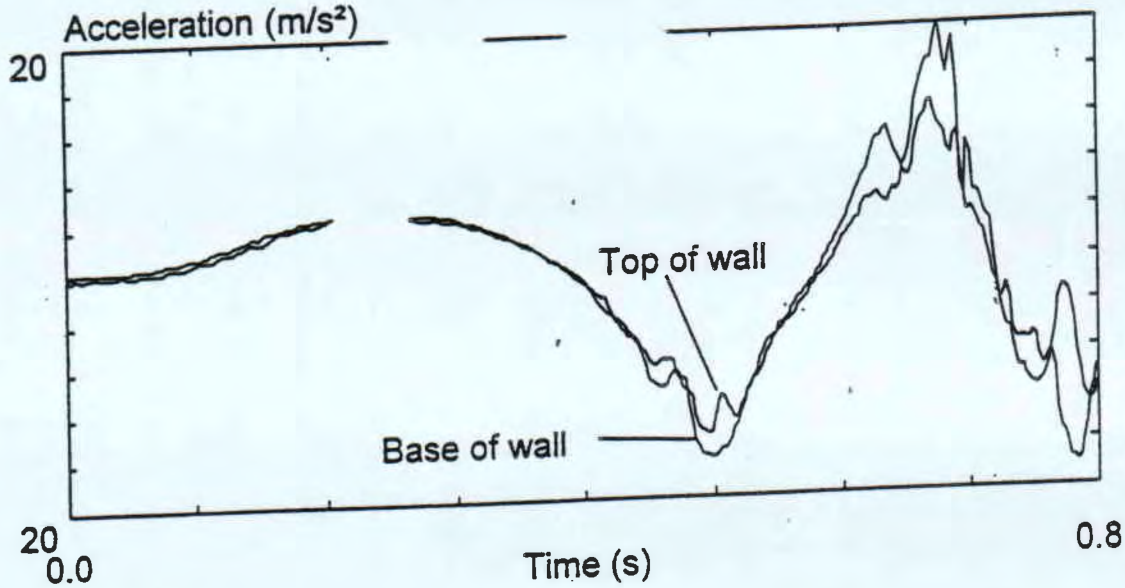

Figure 13 Acceleration time history measured at base and top of box containing the 40mm Thermalite tie for the greatest excitation magnitude.

4.9 Box test 3 - Thermalite 55mm tie

Acceleration time histories for the fifth and thirtieth second of motion at the base and top of the box containing the 55mm Thermalite tie for the final condition are shown in Figure 14. The tests are described in Table 11.

Table 11 Tests carried out on 55mm tied Thermalite brick.

Test	Stimulus	r.m.s. acceleration (ms^{-2})	Peak acceleration (ms^{-2})	Observation
1	1	0.06	0.83	-
2	4	1.35	2.37	-
3	4	3.99	7.03	Initial cracks
4	4	5.42	12.75	Initial cracks
5	4	6.90	17.08	Total failure

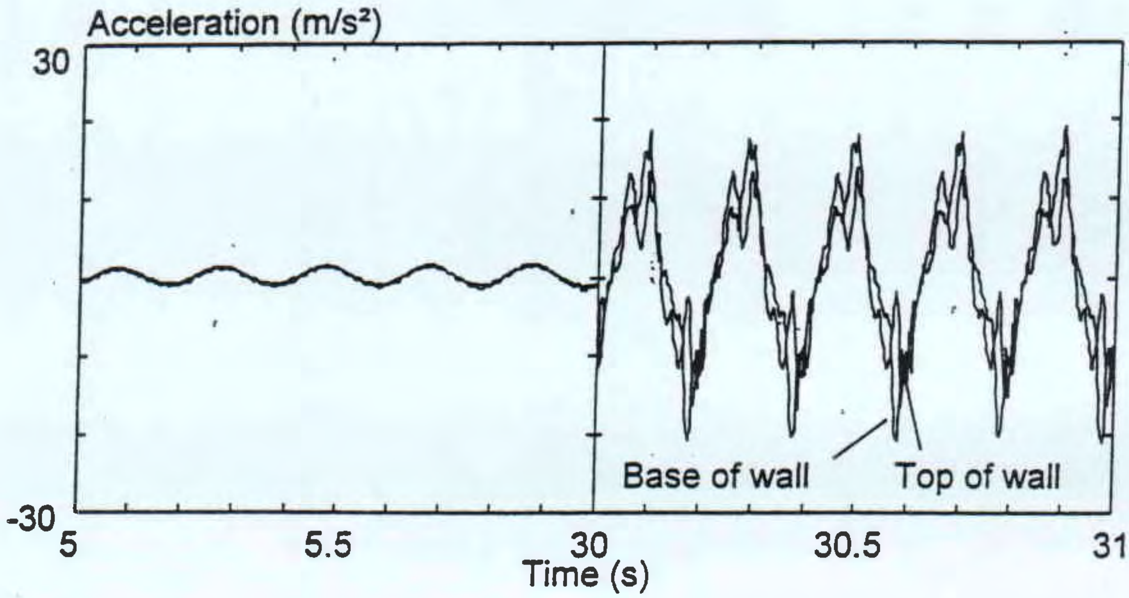

Figure 14 Acceleration time history measured at base and top of box containing the 55mm Thermalite tie for the greatest excitation magnitude. Figure shows fifth and thirtieth seconds of motion.

4.10 Box test 4 - Dense brick 24mm tie

Acceleration time histories for the fifth and thirtieth second of motion at the base and top of the box containing the 24mm dense brick tie for the final condition are shown in Figure 15. The tests are described in Table 12.

Table 12 Tests carried out on 24mm tied dense brick.

Test	Stimulus	r.m.s. acceleration (ms^{-2})	Peak acceleration (ms^{-2})	Observation
1	1	0.07	0.86	-
2	3	1.08	2.55	-
3	3	2.02	4.88	-
4	3	2.83	8.42	-
5	3	4.12	11.44	-
6	3	5.16	15.52	-
7	3	6.30	18.26	-
8	3	6.98	20.03	-
9	3	7.93	22.07	-
10	3	9.29	25.25	-
11	3	10.17	27.33	-
12	3	10.22	28.94	Pin lock nut came loose
13	4	6.53	15.52	Bending of pin

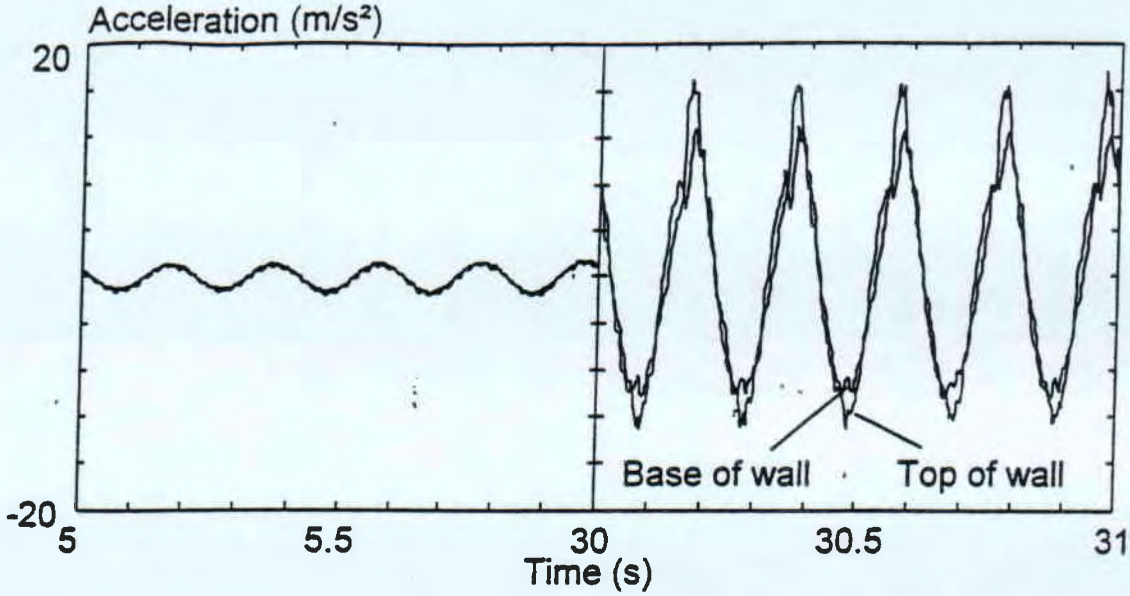

Figure 15 Acceleration time history measured at base and top of box containing the 24mm dense brick tie for the greatest excitation magnitude. Figure shows fifth and thirtieth seconds of motion.

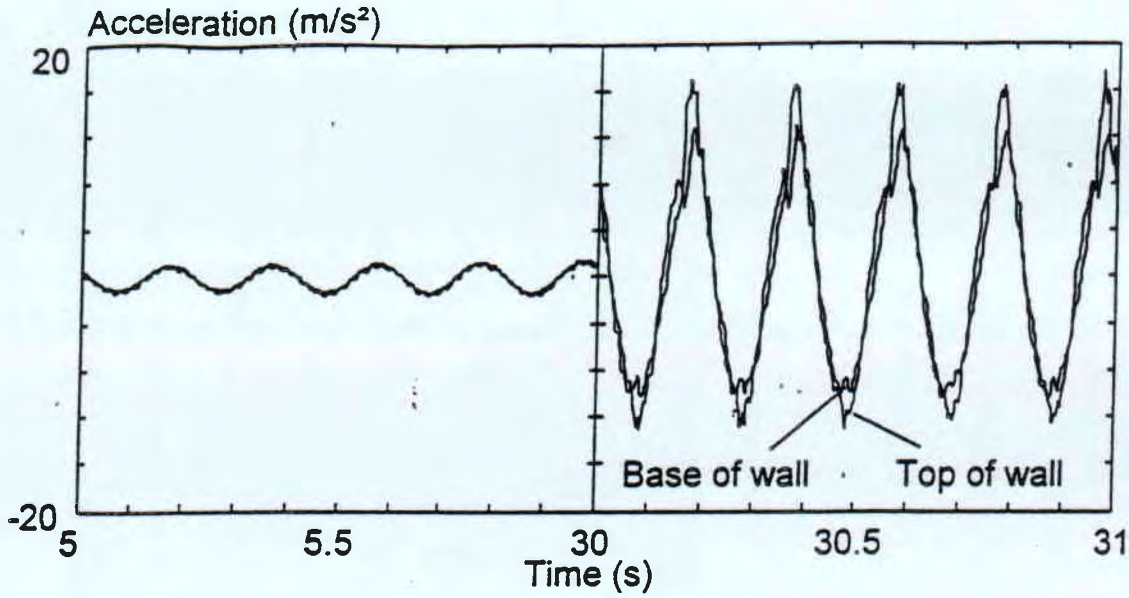

Figure 15 Acceleration time history measured at base and top of box containing the 24mm dense brick tie for the greatest excitation magnitude. Figure shows fifth and thirtieth seconds of motion.

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EARTHQUAKE DAMAGE REPAIR AND STRENGTHENING OF CHRIST CHURCH CATHEDRAL, NEWCASTLE NSW

presented at

STRUCTURAL FAULTS + REPAIR-97

(The Seventh International Conference on
Structural Faults and Repair)
EDINBURGH, SCOTLAND
8 - 10 July 1997

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EARTHQUAKE DAMAGE REPAIR AND STRENGTHENING OF CHRIST CHURCH CATHEDRAL, NEWCASTLE NSW

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INTRODUCTION

In December 1989, Newcastle NSW experienced the first earthquake to affect significantly an urban area in Australia, resulting in the death of 13 people and extensive damage to masonry buildings. The most important building to be severely damaged was Christ Church Cathedral which dominates the skyline of the city.

Australia lies completely within a continental plate and has not been considered as a "seismic" zone as are locations like California and Japan which are near plate boundaries. Academic research has concentrated on earthquakes at plate boundaries leading to a sense of complacency in locations away from these regions. Certainly, the size of earthquakes at plate boundaries ("interplate earthquakes") is potentially much greater than those elsewhere ("intraplate earthquakes"), but the different characteristics of intraplate earthquakes can make them just as damaging as their better known counterparts. It is often conveniently forgotten that the most devastating earthquake in the USA occurred well away from California (New Madrid, Missouri, 1811-12).

Intraplate earthquakes had previously occurred in Australia in sparsely populated regions and, according to the applicable Australian Standard in 1989, Newcastle was located in a "zero" seismic zone, as was most of the populated eastern seaboard. In consequence most practising structural engineers and building authorities in Australia knew little, if anything, about earthquake design requirements. That situation has changed dramatically, and the history of the damage and repairs to Christ Church Cathedral is a good illustration of the evolution of Australian practice.

The structural engineering design of a project such as the Cathedral can be undertaken in a number of ways and, as is usually the case with structural design, different engineers will come up with different valid solutions. Working on a heritage building introduces its own discipline which, in Australia, is governed by the Burra Charter, a document of Australia ICOMOS derived from the world body's Venice Charter. Establishment of heritage significance is the first step in the process of conservation under the Burra Charter.

The Heritage significance of Christ Church Cathedral is embodied in its material fabric (including its structural systems), its architecture, its setting, its contents and what it represents to people.

To best understand this significance, a methodical process of collecting and analysing all of the information, both physical and documentary, was required particularly prior to the major decision-making processes that come after extreme environmental events such as the 1989 Newcastle Earthquake.

The Christ Church experiences genuinely reinforces these principles. For six years following the Newcastle earthquake, the former Consultants, the client (The Anglican Diocese of Newcastle), the Insurance Company (NZI Insurance) and Statutory Authorities struggled with decision-making at every level.

THE 1989 NEWCASTLE EARTHQUAKE

At 10.27 am, Australian Eastern Summer Time, on Thursday, 28th December 1989, Newcastle was subjected to an intraplate earthquake. The earthquake had an epicentre approximately 14 km south west of Newcastle's city centre and was recorded by some distant seismographic stations as having a Richter magnitude of ML5.5 or ML5.6. The earthquake had only about a 10 second duration and there were no major aftershocks.

The Richter magnitude is the one usually quoted in any earthquake because of its relative ease of calculation from distant observations. It is a measure of the total energy released by the earthquake and has only a loose relationship with the intensity felt at the surface or, more importantly, in a building structure.

Most people would be familiar with reports on major earthquakes, particularly those that occur on tectonic plate boundaries. An earthquake of "only" ML5.6 in, say, Los Angeles or Tokyo would probably cause very little damage, so the people from

1. Introduction

Dynamic testing of six walls and four bricks has been carried out at the facilities of the Human Factors Research Unit, University of Southampton on behalf of Gifford and Partners Ltd., Southampton, England. This report presents the transmissibilities of the walls and acceleration at failure.

2. Apparatus

2.1 Shaker

The tests were performed using a horizontal electro-hydraulic vibrator capable of producing a displacement of 1 metre. The table of the shaker consisted of a flat aluminium plate with dimensions of 1.57 x 1.00 metres.

2.2 Accelerometers

Motion of the vibrator plate was measured using an Entran EGCSY-240*-10 accelerometer. Motion of the wall was measured using Setra 141A accelerometers.

2.3 Data acquisition and analysis

An 8-channel *HVLab* data acquisition and analysis system (Acer 9, *HVLab* version 3.80) was used to control the vibrator and to acquire the signals from the accelerometers. The system used *HVLab* data acquisition boards incorporating signal conditioning and anti-aliasing filters.

3. Method

3.1 Stimuli

Five stimuli were used to test the walls as listed in Table 1.

Table 1 Description of test stimuli.

Stimulus	Duration (s)	Description
1	60.0	0.1 to 100 Hz frequency sweep
2	4.0	0.1 to 40 Hz frequency sweep
3	4.0	0.1 to 25 Hz frequency sweep
4	60.0	5.0 Hz tapered sinusoid
5	0.5	Half sine pulse

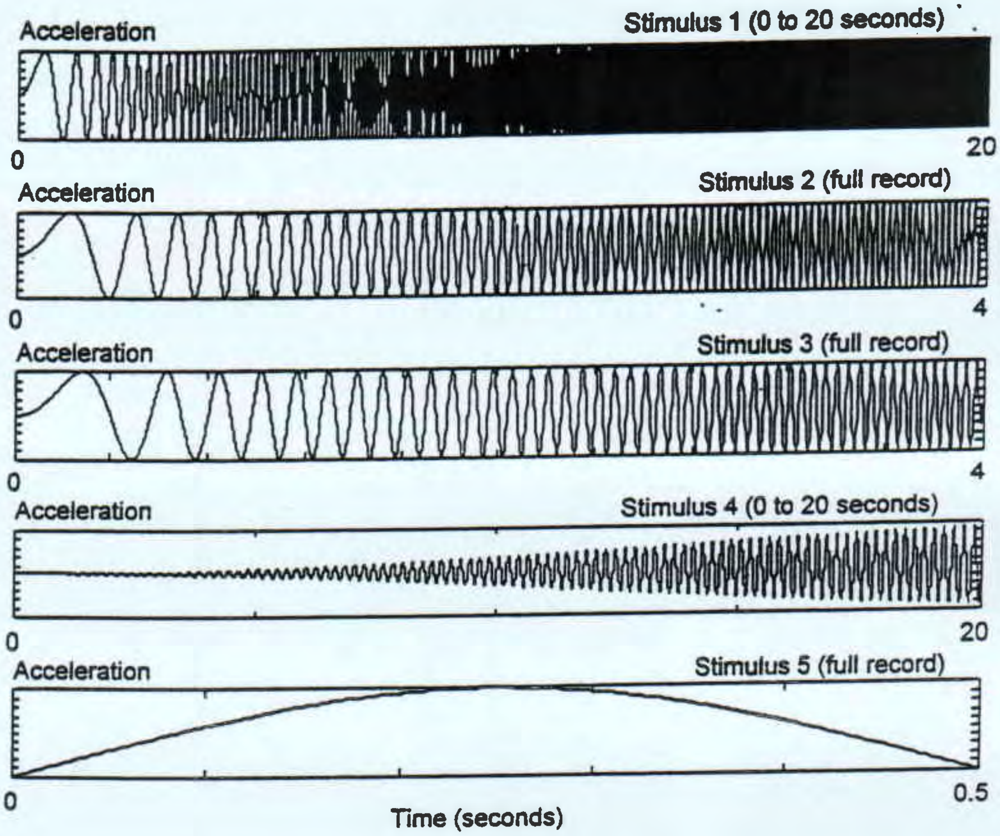


Figure 1 Five test stimuli used to test walls.

The five test stimuli are shown in Figure 1.

3.2 Walls

The ten test walls are described in Table 2.

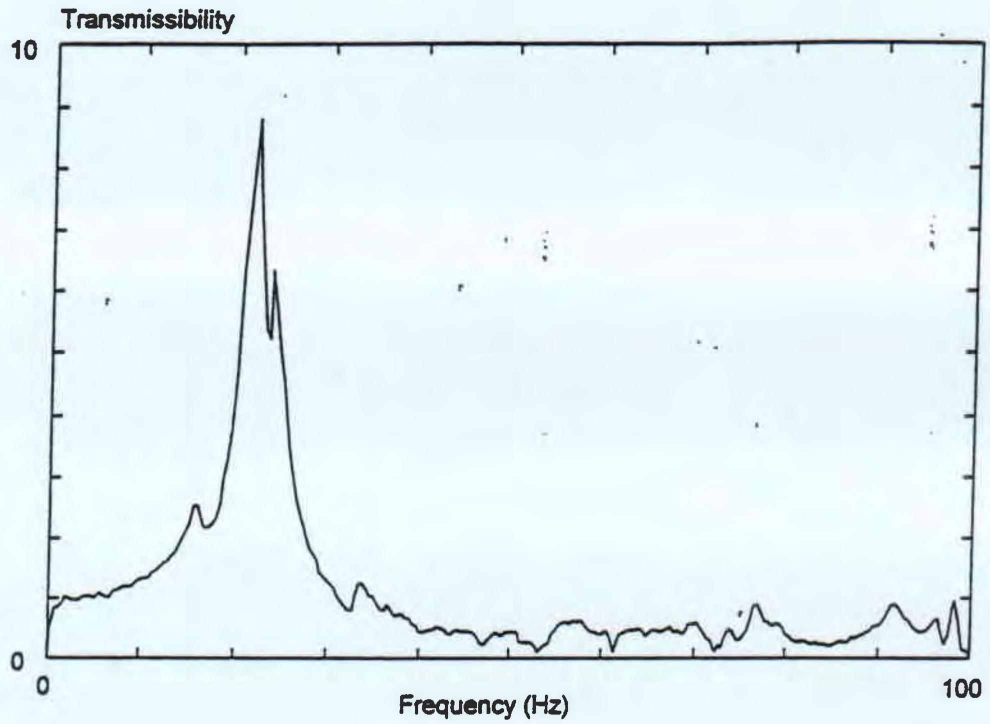


Figure 2 Transmissibility from base to top of wall 1.

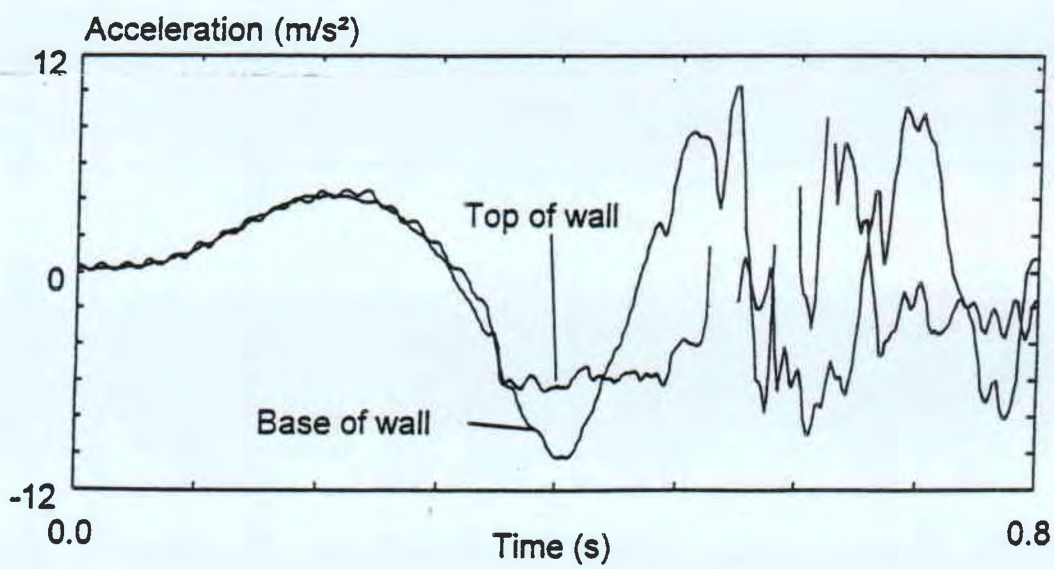


Figure 3 Acceleration time history measured at base and top of wall 1 for greatest excitation magnitude.

4.2 Wall 2 - Ties, straight joint

The transmissibility of wall 2 is shown in Figure 4. The transmissibility resonance frequency was 21.1 Hz with a magnitude of 9.2. Acceleration time histories at the base and top of the wall for the final condition are shown in Figure 5. The tests are described in Table 4.

Table 4: Tests carried out on wall 2.

Test	Stimulus	r.m.s. acceleration (ms^{-2})	Peak acceleration (ms^{-2})	Observation
1	1	0.05	3.79	-
2	2	0.15	3.20	-
3	2	0.20	2.13	-
4	2	0.11	0.38	-
5	2	0.19	0.68	-
6	2	0.35	1.26	-
7	2	0.83	3.05	-
8	2	0.92	3.62	-
9	2	1.89	5.48	-
10	2	1.73	10.97	Wall lifted from pallet
11	2	2.13	8.15	-
12	2	3.69	11.21	Initial cracks
13	2	4.71	15.27	Initial cracks
14	2	5.91	14.73	Forward shear
15	2	6.96	17.25	Total failure

4.3 Wall 3 - No ties, bonded joint

The transmissibility of wall 3 is shown in Figure 6. The transmissibility resonance frequency was 18.3 Hz with a magnitude of 9.6. Acceleration time histories at the base and top of the wall for the final condition are shown in Figure 7. The tests are described in Table 5.

Table 5 Tests carried out on wall 3.

Test	Stimulus	r.m.s. acceleration (ms^{-2})	Peak acceleration (ms^{-2})	Observation
1	1	0.07	0.33	-
2	2	0.52	1.23	-
3	2	1.00	2.47	-
4	2	2.02	5.09	-
5	2	2.99	7.46	-
6	2	3.98	9.90	-
7	2	5.00	12.24	Wall movement on pallet
8	2	5.88	14.48	-
9	2	5.96	16.22	-
10	2	5.91	14.87	Shearing of wall
11	2	6.82	17.28	Shearing of wall
12	2	7.94	19.14	Total failure

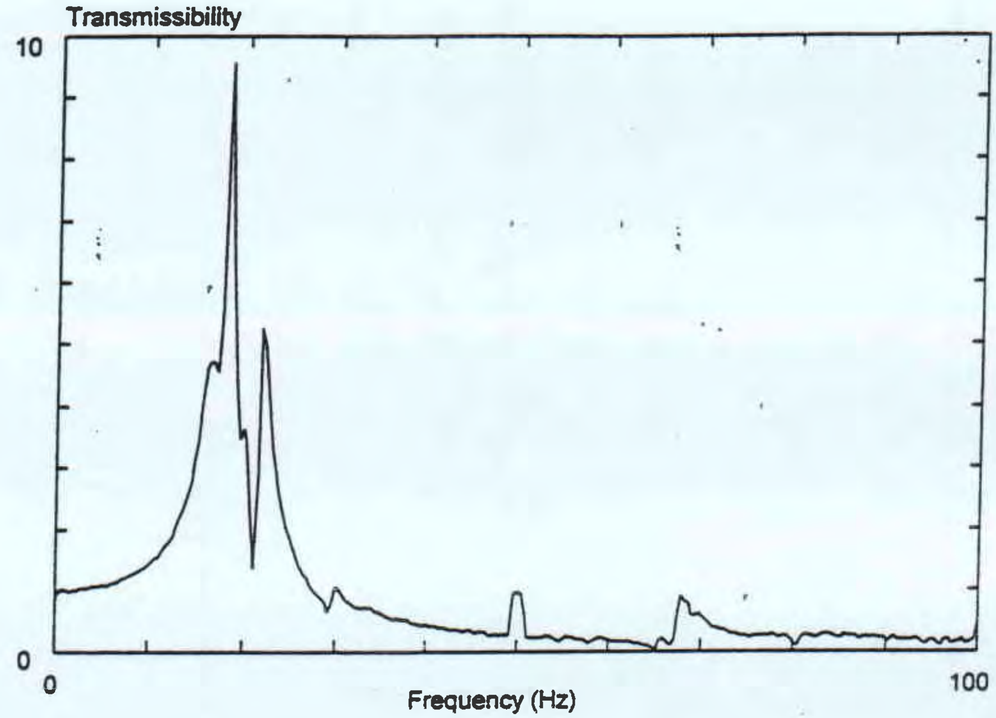


Figure 6 Transmissibility from base to top of wall 3.

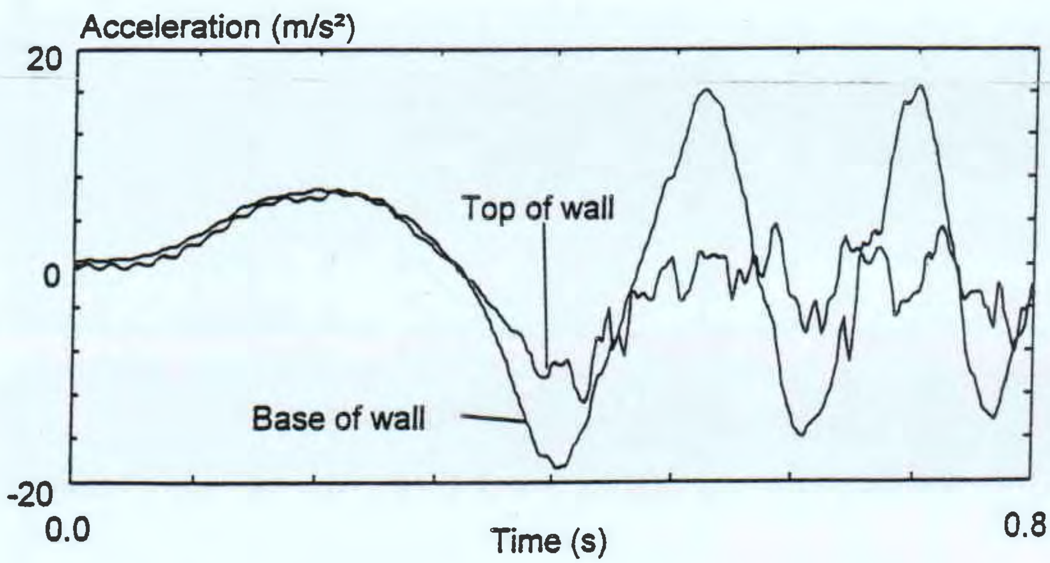


Figure 7 Acceleration time history measured at base and top of wall 3 for greatest excitation magnitude.

4.4 Wall 4 - Ties, bonded joint

The transmissibility of wall 4 is shown in Figure 8. The transmissibility resonance frequency was 25.4 Hz with a magnitude of 8.3. Acceleration time histories at the base and top of the wall for the final condition are shown in Figure 9. The tests are described in Table 6.

Table 6 Tests carried out on wall 4.

Test	Stimulus	r.m.s. acceleration (ms^{-2})	Peak acceleration (ms^{-2})	Observation
1	1	0.07	0.35	-
2	2	1.02	2.54	-
3	2	2.07	5.07	-
4	2	3.05	7.43	-
5	2	4.16	9.82	-
6	2	5.17	12.60	-
7	2	6.06	15.21	Initial cracks
8	2	6.97	16.55	Cracks
9	2	7.99	22.17	Cracks
10	2	9.05	22.27	Total failure

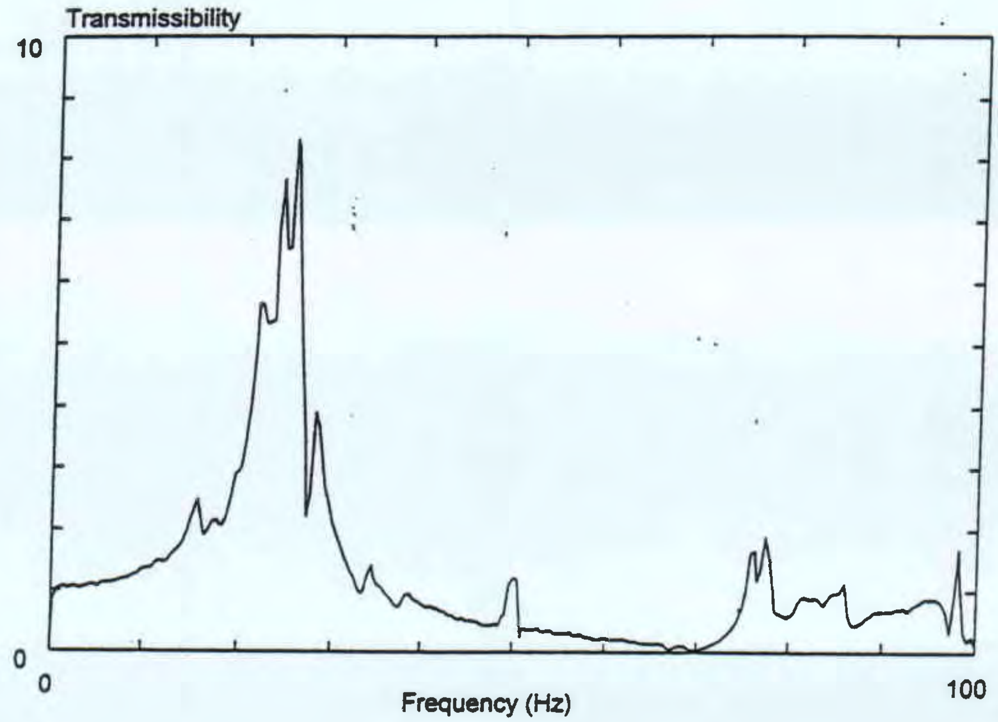


Figure 8 Transmissibility from base to top of wall 4.

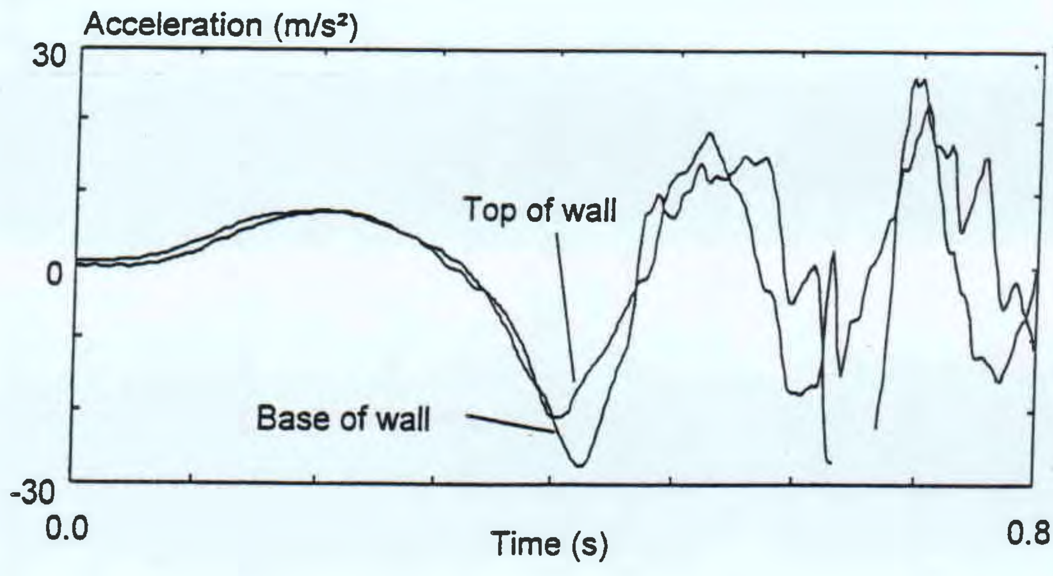


Figure 9 Acceleration time history measured at base and top of wall 4 for greatest excitation magnitude.

earthquake prone countries (that is prone to interplate as opposed to intraplate events) wonder what all the fuss was about. One of the big differences, of course, is that in Newcastle the energy was released at a depth of about 14 km beneath the surface: typical interplate earthquakes occur at depths of the order of 100 km, so there is a lot more between them and the surface to absorb and deflect the energy.

To overcome the problems associated with the Richter scale, the scale known as the Modified Mercalli index is used in most western countries. This is a partly subjective scale which attempts to classify damage at the surface and hence give a measure of intensity. In Newcastle the Modified Mercalli index varied throughout the area, subject to local geological conditions, but it ranged up to MMVIII on an areal basis with some pockets of damage possibly being classified as MMIX: this is not much different from some of the intensities experienced in the well-known earthquake areas on a scale where total destruction is the highest at MMXII. At the Cathedral site the damage is consistent with MMVII, although most of the surrounding area was noted as MMVIII on the published studies: at the bottom of the hill on which the Cathedral stands the soils are alluvial with a greater amplification factor, which would explain the difference.

The Newcastle area has been mined for coal since 1797 and the whole area is riddled with old mine workings. No conclusive evidence has been put forward to suggest that the workings affected the intensity of the earthquake in any general way and miners working underground quite close to the epicentre are reported not to have felt the tremor.

A fuller account of the earthquake and damage caused to older buildings can be found in a previous paper by one of the authors (Jordan, Trueman & Ludlow, 1992).

DAMAGE TO THE CATHEDRAL

Christ Church Cathedral, which dominates the skyline of Newcastle, is the largest provincial Anglican cathedral in Australia and is of "Federation Gothic" style (Apperly, Irving & Reynolds, 1989) in brick masonry construction; building of the walls commenced in 1893 and continued until the 1970s when the tower was added. The building is of cruciform shape and 67 metres long. A tower is located at the crossing of the transepts, with small towers at the western end. The internal span of the main roof is 9 metres and a single clerestory is supported by flying buttresses.

The effect of the earthquake on the building was largely as would be expected: high-set stone crosses and other decorations were dislodged and fell to the ground or lower roofs; flying buttresses were dislodged, but none fell completely away; shear cracking occurred in the nave walls which lie roughly parallel to the direction of the seismic wave and out-of-plane movements occurred in the east wall and dislodged windows. The degree of damage varied with the state of the brickwork with very little occurring in the tower structure, completed in 1979.

Figure 1: Christ Church Cathedral, Newcastle Australia, 1989

As well as the familiar "p-" and "s-" waves associated with vibrations in solids, earthquake waves (or any vibrational wave in a solid with a boundary plane) can be shown to have two other types of wave motion associated with them: Love waves oscillate in the boundary plane, or "half space", and Rayleigh waves form alternate circulation patterns perpendicular to the boundary. Of particular interest seismically was the behaviour of the brick finials topping off the columns of the main walls of the nave. The finials could be seen to have rotated alternately along the length of the nave. This gave an interesting example of Rayleigh waves with a horizontal component. The cathedral is situated towards the top of a steep hill, with steeply dipping strata under it, and the circulating patterns of the Rayleigh waves would have produced horizontal circulating components in the building.

STRENGTHENING CRITERIA

History following the earthquake

In the period immediately following the earthquake an attempt was made by the responsible building authority, Newcastle City Council, to define a standard for new building works and, by implication, for repair of older buildings. Naturally, such general standards could not cope with a building such as the Cathedral, and so started years of argument among engineers and architects acting for the insurers, the church authorities and the City Council. Estimates of repair costs varied by factors of up to four because of the differing interpretations of what needed to be done.

Firstly, the emergency decisions involving immediate demolition of sections of the Cathedral without adequate recording meant reconstruction to exact detail was more difficult, requiring additional research and documentation which, in some cases, would not have been required if conservation policies had been in place to direct decision-making.

Immediately following the earthquake, the first priority was to make the damaged structure safe for the public. This was achieved through various methods of scaffolding, shoring, propping, strapping and unrecorded removal of various stone and brick elements. During this period, various proposals were considered which involved demolition of large sections of the building, including the removal of the flying buttress and parapet elements which are significant aesthetic and structural components of the Cathedral.

These aggressive approaches led to many disagreements, and ultimately confusion, between the various consultants, authorities, the Insurer and the Diocese, on the best way to approach the insurance claim and ultimate repair and reinforcing of the building. Legal argument followed and continued for a period of five years.

Australian Standard Earthquake Loading Code

The Newcastle earthquake precipitated a rapid reappraisal of the Australian design code for earthquake loading which had been under review for some time but with little sense of urgency. As a result in 1993 the new code was issued as part of the structure loading code, being designated AS 1170.4—1993, "Minimum design loads on structures, Part 4: Earthquake loads". The new Code became obligatory for structural design of the project.

However, as in so many structural design exercises, there was more than one way to interpret the Code and apply it to the structure. The different philosophies of the firms of structural engineers could not be resolved.

Appointment of Engineer Mediator

Following an agreement between the Insurer and the Diocese, a Working Party was formed to resolve the disputes. An eminent Engineer, with structural and heritage conservation credentials, Mr Harry Trueman, was appointed to assist the working party in resolving the differences, both analytical and philosophical, between the various consultants working for the Insurer and the Diocese (See Appendix).

Selection of the structural design parameters was not a clear choice, even with the help of the new Code, because the building does not fall into a readily identifiable class, nor does such a building have a design life within the boundaries considered by the Code (a "design earthquake" is based on an estimated 90% probability of the ground motions not being exceeded in a 50-year period). The difference in cost estimates due to the earthquake design parameters was A$1.6 million in a total difference of A$12.6 million.

Resolving the design differences

Of more importance in the difference in estimates was the approach to the work. On one side was a proposal to demolish large sections of the building and to reinstate it to an "as new" condition, together with drilling and reinforcing of apparently undamaged sections of masonry; damage in a design earthquake was to be minimal. At the other extreme was a proposal to only reinforce sufficiently to prevent collapse in the design earthquake, with repair rather than replacement of damaged brickwork.

After reaching agreement on structural design parameters including earthquake loading requirements, it was then proposed by Mr Trueman that the best way to approach the vast philosophical differences was to prepare a Conservation Plan for Christ Church Cathedral. The Conservation Plan in the Australian context is structured following a specific form devised under the guidance of Dr J S Kerr (Kerr, 1985), in accordance with the principles the "Burra Charter" (Australia ICOMOS, 1988). Those principles originated from the Venice Charter. Similar documents elsewhere in the world are known by names such as Historic Structure Report or "HSRs" (APT, 1997).

In the Christ Church Cathedral case, the brief from the Working Party explicitly stated "The Conservation Plan is to advise the most appropriate method of reinstatement and rectification of the Cathedral". Following expressions of interest in January 1995 and a response to the Working Party brief, EJE Architecture was selected as the Conservation Consultant to prepare a Conservation Plan to guide the decision-making processes.

CONSERVATION PLAN

The Conservation Plan for the Christ Church Cathedral established the Historic, Aesthetic, Social and Scientific basis upon which decisions and implementation of the reinstatement and rectification of the Cathedral could proceed (EJE Architecture, 1995).

The Conservation Plan process, through documentary research and physical analysis, provided:

1. A chronological history of the 11 different construction phases of development on the Cathedral site.
2. Identification of the important fabric of each construction phase.
3. The particular and differing construction methods, techniques and materials of each construction phase.
4. Time and environmental factors that contributed to the latent conditions of the materials and structure.
5. Information on previous repair and maintenance methods.
6. Data and descriptions of other extreme structural failures in the building due to mine subsidence.

Figure 2: The original Christ Church, 1817

The chronological history of construction on the Cathedral site began in 1817 with the erection by convict labour of a small 'T' shaped Church built of brick and stone with a steeple 30.48m (100 feet) tall. From plans, artists impressions and photographs, the exact location of any probable archaeological deposit was able to be determined. The protection and management of the remnant footing from 1817 was given equal precedence with the rest of the building. During removal of destabilised floors in the nave area, portions of the original stone footings were uncovered, recorded and retained without disturbance. The knowledge of the existence and location of the archaeological deposits greatly influenced the design and construction methods of the new reinforced concrete floors, ensuring the protection of the deposit.

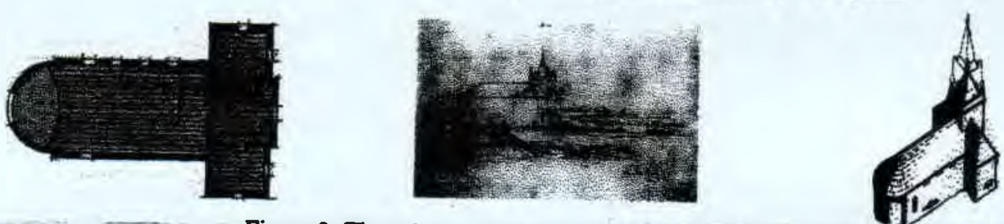


Figure 3: The spire and portion of the tower removed in 1825

From its inception in 1817, the small brick and stone church was gradually demolished with the tower being reduced in two separate stages due to instability. Firstly, in 1825 the steeple, upper tower and 3 m (10 feet) of the lower tower were removed

Figure 4: The remaining tower removed and bellcote added, circa 1868

and some time after 1868 the remaining portion of the tower was removed and a small bellcote installed. In 1883 the installation of the mass concrete footings for the new Cathedral led to the demolition of the 1817 Church in two stages, firstly

Figure 5: The footings of the new Christ Church Cathedral laid 1883

the East End and then the West End. This demolition even caused consternation between the Design Architect John Horbury Hunt and the Dean of the Cathedral. This argument centred around the structural integrity of the footing and the desire of the Dean to retain the building in use for as long as possible. Horbury Hunt demanded that the tower footings must be laid at the same time as the wall footings so as to negate the possibility of differential settlement. The construction of the Cathedral continued at a slow pace with the face brick masonry walls only reaching half their intended height by 1893. Disputes over

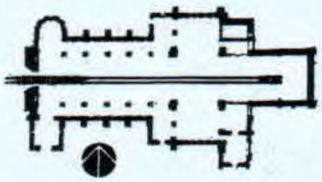


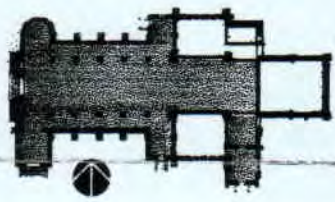
Figure 6: The lower external walls as they stood for a period of eight years from 1893

types and quality of bricks persisted between the Architect, the Contractor and the Cathedral Chapter. The brick construction consisted of solid brickwork up to 1.2 m thick in English bond, finished internally and externally as face with joints as small as 3 mm and the brick detailing providing the decoration. Externally the bricks were hard double glazed, liver coloured, internally a soft cream, intended to maximise the light quality.

In 1893, following a dispute over certified payments to the Contractor by the Architect and a bitter legal battle, John Horbury Hunt and the Contractor were dismissed. The brick walls were left unfinished and exposed to the extreme elements of a coastal climate and environmental pollution of a rapidly growing industrial town.

The building suffered damage and the mortars deteriorated due to constant wetting and drying, particularly to the interior of the brickwork from the exposed bed. This situation remained until 1901.

In 1901 a new Architect was appointed to temporarily roof the nave, the crossing, the baptistry and the porch at half the final height and prepare it for occupation. In the process, brick pilasters to the main nave columns were removed and sandstone



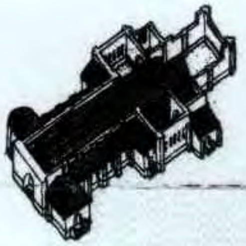
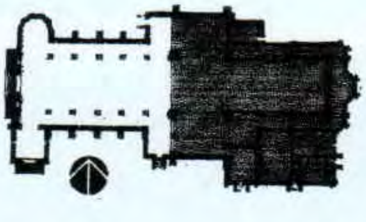


Figure 7: The temporary roof to the nave installed in 1901

facings to the crossing arch supports were introduced. At this time, the deteriorated brickwork was repointed with a hard cement-based mortar to all internal faces. This work was completed and the Cathedral opened for service in 1902. Within five years the Cathedral suffered severe structural damage to its western end with severe mine subsidence which occurred again two years later.

The next construction phase began in 1909 with the temporary roofing of the chancel and the construction of the vestries to a



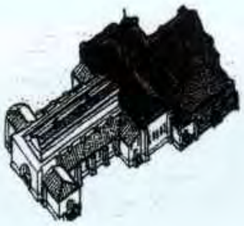


Figure 8: Temporary roof to the chancel installed 1909

new design by Newcastle Architect, F.G. Castleden. With this construction the brick type changed and the joint sizing increased to approximately 10 mm. The quality of the bricklaying also decreased and little attention was given to the filling of perpend joints in the interior of the brickwork.

1912 saw the Cathedral begin to be raised to its full height with the construction of the upper walls of the chancel. Significant in this construction was the use of hard refractory bricks as commons, which proved to be one of the many

Figure 9: The chancel raised to its full height, 1912

difficulties encountered in the drilling for the 1996 reinforcing. 1912 also saw the construction of the Great East Window using sandstone and stained glass imported from England.

World War 1 curtailed the construction programme on the Cathedral. However, as early as 1915 steps were taken to erect a war memorial "if we win the war" (Murray, 1991). The Warriors Chapel was completed in 1924, adding a significant aesthetic and social component to the Cathedral. The Chapel was constructed in a separate structural element on the

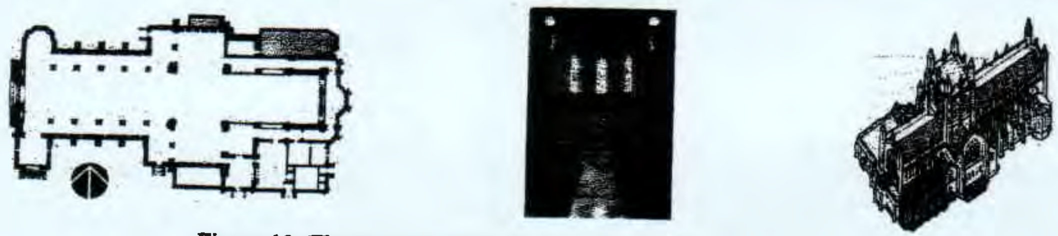


Figure 10: The Warriors Chapel constructed 1924

northern side of the Cathedral and included a finely-detailed Australian sandstone and marble interior, none of which are currently quarried and as such, are unavailable for use.

1926 saw the Nave raised to its full height with flying buttresses and the introduction of a cavity within the brick wall. The base of the Tower was completed and the Western Rose Window installed at this time.

Figure 11: The nave and tower base raised 1926

The final construction phase was the raising of the transepts and the completion of the massive bell tower using cavity work,

Figure 12: The tower and transepts completed 1979

cement mortar and a relatively soft clay face brick compared to the lower portion of the building. This work was designed by Architect John Sara, of the Newcastle firm of Castleden & Sara, and completed in 1979.

Following the documentary and physical research, a Statement of Significance for the Cathedral and site was established following the four criteria of Historic, Aesthetic, Social and Scientific Significance.

Christ Church Cathedral, Newcastle and its site is rare in Australia for its association with the early convict history of the Colony and its paralleling of the historic development of Newcastle, Australia's sixth largest city. As a focus for many historic events the Cathedral is unsurpassed in the region. Its place in Australian Anglican Church history and its association with many Bishops and political dignitaries is also of importance. The impact of the 1989 Newcastle earthquake and its effect on the physical fabric and social history of the Cathedral enhances the historic nature of the Cathedral as a centrepiece of cultural sentiment for the people of Newcastle, the region, the State and the nation.

Aesthetically Christ Church Cathedral is an extraordinary piece of architecture in a dramatic setting. The building displays the innovative skill and ability of John Horbury Hunt and the detailed design ability of F.G. Castleden. The building's stock of stain glass, craftsmanship and artwork heightens the aesthetic value. Stylistically the building expresses the significant changes from the Victorian period of architecture with its reliance on academic correctness to the freer realisations of the Federation period and its influence by the Arts and Crafts movement in Australian architecture.

Socially the Cathedral has been and will remain a focus for the lives of the people of Newcastle, the region and in many respects the State and nation, in terms of tourism and the perception of Newcastle as a city. At a regional and local level the Cathedral is the premier location for the expression of Anglican religious practice and is a key element of cultural activity for the community.

Scientifically the site and Cathedral are of some significance, particularly in regard to European historic archaeology, the understanding of J. Horbury Hunt's work and the effect of mine subsidence and earthquake on large masonry buildings in Australia.

CONSERVATION POLICIES

In order to retain the specific significance of the Cathedral, a detailed conservation policy was developed in conjunction with an implementation strategy. The policy had as its basis the requirements of:

- the Anglican Diocese of Newcastle
- the Insurer (NZI Insurance)
- Local, State and Federal Government Legislation
- Structural Engineering (including earthquake loading)
- Architectural considerations
- Physical condition
- Retention of significance

Nineteen separate broad Policy Statements evolved. The policy statements that specifically influenced the philosophical and technical approach to the structural repair and strengthening were:

Policy No. 2 The conservation issues to be closely and creatively linked to the overall repair strategy for the Cathedral. Specifically respect for the various stages of the Cathedral's construction be taken into account in terms of differing construction techniques and material variations.

> *Reason: So that all periods of the building's history are recognised and the various associations with Clergy, Architects and Builders are not obliterated.*

Policy No. 3 Maintain a philosophy of cultural continuity through the retention of the physical effects of historic changes to the fabric both intentional and accidental (e.g. evidence of the intentional removal of brick pilasters to the nave arcades by J H Buckeridge and evidence of the accidental cracking of masonry due to mine subsidence and earthquake).

> *Reason: So that significant historic events in the Cathedral's history can be interpreted to the broader community, particularly the dramatic events of the 1908 mine subsidence and the 1989 Newcastle earthquake.*

Policy No. 5 That the future conservation and development of the Cathedral and its surroundings be carried out in accordance with the principles of the Australia ICOMOS Charter for the conservation of places of cultural significance (the Burra Charter) as adapted by Australia ICOMOS on 14 April 1984 and revised on 23 April 1988 together with its associated guidelines as published in the illustrated Burra Charter, October 1992.

> *Reason: To ensure that management, conservation architects, engineers, builders and others involved in work on the Cathedral become sufficiently familiar with the Burra Charter for its practicality and flexibility to be fully understood.*

Policy No. 6 That a clear structure showing responsibility for the specific care of the fabric and contents of the Cathedral be set out and made available to all persons involved in work associated with the Cathedral and that this practice be continued in the ongoing management of the Cathedral and its contents.

Reason: To set out the need for a clear understanding of responsibility for decision, the execution of work and the relationship of all involved. This will help avoid problems of communication and demarcation which may reduce efficiency, increase costs and result in damage to the fabric.

Policy No. 7 Modern techniques and advanced technology to achieve code requirements for structure and stability should only be used where traditional techniques cannot achieve a satisfactory level of compliance. Repair should take into account both the elemental basis as well as the overall integrity of the structure. Such methods and techniques must be proven and suitable to the extant materials of the Cathedral.

Reason: To retain the overall historic character of the building and its individual elements while complying with contemporary code requirements but not at the expense of historic, architectural and cultural values.

Policy No. 8 That the protection of the potential archaeological deposit likely to be found in the area of the nave be of a high priority and that all investigation and conservation of the deposit follow the guidelines for Historical Archaeological sites produced by the NSW Department of Planning Heritage Council of NSW, March 1993.

Reason: So than an irreplaceable resource for interpreting Australian history and culture is not inadvertently damaged or compromised.

Policy No. 9 Retain the existing configuration of the Cathedral and maintain the elemental fabric by repairing that fabric which no longer performs its original function or promotes further damage to the fabric.

Reason: To preserve the appreciation of the work of Horbury Hunt and the subsequent Cathedral architects, to ensure the retention of as much original fabric as possible.

Policy No. 10 The form of the Cathedral while appearing complete may be changed following the adaptive program of its history. However, design resolution compatible with the existing form must be the highest priority. An understanding of the intricacy of the Cathedral's design which is summarised in this document must be mandatory for any future architects, engineers and builders.

Reason: To allow future development of the Cathedral in parallel with the desires and aspirations of the community and to ensure sensitive appreciation of the Cathedral by professionals and craftsmen.

Policy No. 12 Conservation work should initially focus on identification of the stages of construction and the analysis of materials with the view that new materials used in the repair should closely match original or adjacent materials while displaying evidence of change.

Reason: To ensure a sound and considered technological basis, in terms of traditional and modern methods while ensuring the ability to identify the historic chronology of the building fabric

Policy No. 13 A patina of age, evidence of use and evidence of residual damage is desirable.

Reason: To assist in the interpretation of the fabric and its different stages of construction. To ensure the perceived cultural continuity is maintained and that evidence of significant events is appreciated.

Policy No. 14 Structural reinforcement installations should minimise its impact on the building's fabric with a minimalist approach to achieving the code requirement, but the long term serviceability of the building and fabric ensured.

Reason: To ensure minimal damage to the historic fabric and maximum benefit in terms of future maintenance and preservation of the fabric.

Policy No. 15 Existing facade modulation, window and door fenestration both internally and externally should be maintained.

Reason: To preserve the significant design intent of architects Hunt, Castleden and Sara and to maintain the aesthetic image of the building for the broader community.

PROCUREMENT AND MANAGEMENT OF THE PROJECT

Repair practices in Newcastle following the earthquake

As indicated previously, the 1989 Newcastle earthquake was the first destructive earthquake to occur in modern times in a major urban area of Australia. This meant that Australian engineers were not experienced in the design or reinforcement of buildings for earthquake loadings and, particularly in the major population centres of the east coast, most had no concept of the requirements, as the existing Australian Standard for earthquake loading classified those centres as being in Zone 0, where no structural consideration was necessary.

Most engineers hurriedly became familiar with the requirements as experience was imported from overseas (mainly New Zealand and California), but little help was available immediately for dealing with masonry buildings of heritage importance, particularly the Cathedral.

The co-authors of this paper were both appointed by the NSW Department of Planning, Heritage Council, to a panel comprising of three Architects and three Structural Engineers who were charged with giving guidance on the conservation of heritage properties; it was during this time that we became aware of the limitations of available strengthening systems. Typical of the systems being used were internal and external steel frameworks, which would be quite inappropriate for a building such as the Cathedral due to visual and environmental considerations.

Some buildings were reinforced by the drilling and reinforcing of masonry, but this often caused many secondary problems, among them being:

- grout was introduced to bond and encapsulate the reinforcing but it flowed out through cracks and flooded cavities, causing considerable problems on site in cleaning it up and leading to damage to other elements of the building;
- with horizontal drill holes, there was no assurance that the grout had bonded the bar for the full length;
- low strength grouts used did not develop the bar strength close enough to the ends of reinforcing bars, leading to designs using epoxy end anchorages — the use of epoxy formulations in heritage buildings was of considerable concern to the authors and many others, but alternatives did not seem to be available.

Other practices which were common, but undesirable, included the repair of masonry by epoxy crack injection. This introduced a material into the building fabric which usually did not have compatible physical properties across the temperature range likely in the masonry and, in many cases, the fire rating of the wall was adversely affected. The performance of epoxy-type materials in buildings, such as a cathedral, with a likely future life measured in hundreds of years was also uncertain.

Finding a better way

Before work on the Cathedral had commenced, one of the authors (Jordan) became aware of the German developed and British marketed "Cintec" masonry anchoring system. Not being directly concerned with the project it was a large commercial risk for a sole-practitioner engineering consultant to introduce the appointed design consultants to the system, find a contractor with the necessary experience to train as an installer and to be instrumental in setting up an Australian branch for Cintec and import the product for the Cathedral project. This was especially so as the project turned out to be the largest installation of Cintec anchors in a single building that the company had tackled and included the longest continuously grouted anchors. Fortuitously, Cintec products were marketed by a private company based in Wales and owned by a man willing to take on such a large project with unknown people at such a distance. The risks taken on by Mr Peter James are acknowledged and played a large part in the final success of the project.

The credentials of the Cintec system are probably well known to European audiences, but a long period ensued before the system was accepted by the Engineers and Architects as being the only viable system for reinforcing the Cathedral. Trial installations were carried out, tests were undertaken and, in the end, a contract was signed with the trained Cintec installer, Australasian Concrete Services Pty Ltd, for the whole of the work on an accelerated programme.

Structural design

The prime aim of the structural design was to turn the building into a ductile structure, lack of ductility being the main cause of catastrophic collapse of buildings during earthquakes. It was accepted that different parts of the building would end up with different degrees of ductility, and so would be likely to suffer different degrees of cosmetic damage in a future earthquake. However this had to be accepted in order to maintain the aesthetic significance of the building, but with the overriding criterion that risk of personal injury to occupants of the building during a future earthquake would be reduced to a minimum.

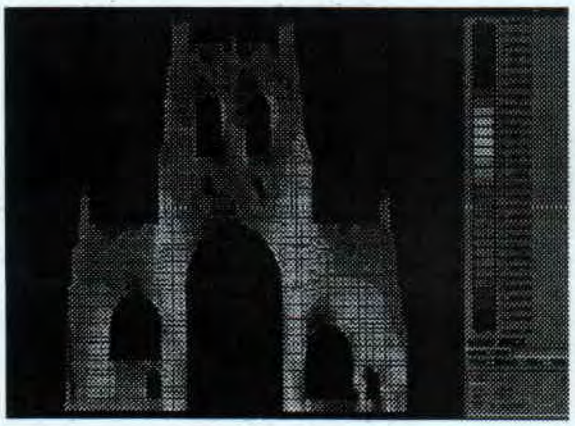


Figure 13: Typical FEA output

Ductility could be introduced into the building either by the addition of ductile frames, with the obvious visually intrusive consequences or by fully reinforcing the brickwork, which had cost and practicality drawbacks. In the end, a combined system was chosen which used some frames where they are hidden from view: behind the parapets, in the roof space and inside the upper section of the tower.

With the design parameters set (see Appendix) a two-dimensional finite element analysis was carried out using a readily available computer package ("Strand 6"). Different combinations of reinforcement strength and spacing were considered and costed and, in the end, a high strength Grade 316 stainless steel deformed bar was used in sizes from 16 mm to 32 mm diameter. In order to limit the amount of drilling that had to be carried out "Hi-Proof" bar with UTS in range 790 to 920 MPa [N/mm^2], depending on size, was specified. The total length of reinforcing installed was 3770 metres.

Anchor (Reinforcement) Design

The Cintec anchor system has three basic elements: the anchor body which carries the load, the cementitious grout formulated using micro-cement technology and the woven fabric sock which controls the grout. Details of the sock and grout can be found in company literature (Cavity Lock Systems Ltd, current).

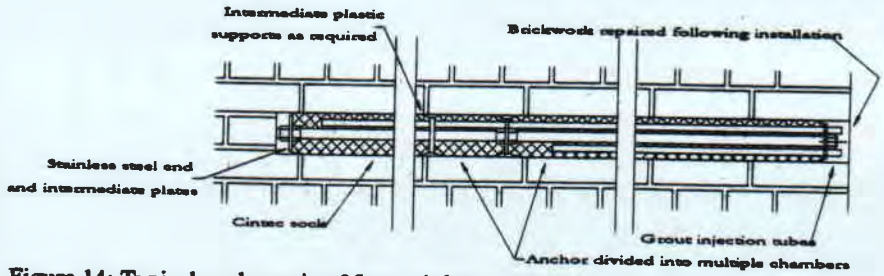


Figure 14: Typical anchor using 25 mm deformed bar in 85 mm diameter hole

The sock design for the Cintec system allows the sock to expand in the drilled hole, and to its maximum in voids, without the openings in the weave allowing more than the finest fraction of grout to pass through. This "grout milk" contains the inorganic adhesion agents of the grout together with the excess moisture not required for cement hydration. The final expulsion of the excess moisture does not occur until the full sock is pressurized, when the inflated sock becomes hard. This process ensures that the filling of the sock can be monitored by observing a small section, either at the injection end or by way of a small observation hole for remote chambers. For shorter anchors, the Cintec system uses return grout tubes for the monitoring of blind chamber inflation. This was not possible for most of the very long holes on the Cathedral without an increase in hole size, but small monitoring holes at joints could be drilled to the side of anchors and repaired later.

Hole sizes were determined by the bond requirements at the ends of the bars, the grout cover requirements to the bar and the requirements for coupling the bars and achieving grout flow. In general, 60 mm diameter holes were adopted for the 16 mm anchor bodies up to 6 m long, 85 mm holes for longer 16 mm , 20 mm and 25 mm anchor bodies, and 120 mm diameter holes for the 32 mm anchor bodies. It would have been possible to have a greater range of hole sizes to meet the design parameters and to have saved some drilling and grout. However , the installation contractor chose to limit the range for better quality control.

Drilling and installation

Masonry drilling is usually carried out by diamond coring using water for cooling and cuttings removal. The use of water or other drilling fluids was not desirable in the Cathedral for two reasons. The more obvious was the potential damage that could be done by escaping drilling fluid: many of the more important "treasures" of the Cathedral could not be removed

Figure 15: Horizontal anchors assembled during installation

Figure 16: Vertical anchor installation by crane

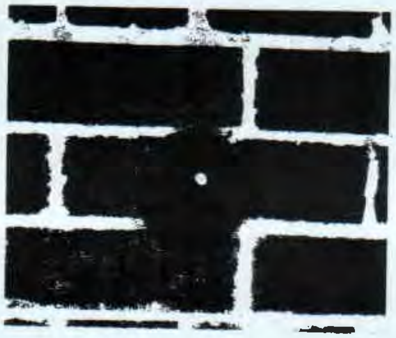


Figure 17: Finished installation prior to brickwork repair

without damage. Also, it was calculated by the structural engineers that a saturation of the brickwork by water could so increase its mass as to risk foundation failure.

Hydraulic drills were set up by securing them rigidly to the masonry of the building. Guidance of the drills depended on the accuracy of the initial set-up in all but the 32 m long holes, where inspection panels were able to be opened along the line of the holes from where the drill rods could be redirected if necessary. Dry drilling by non-coring, polycrystalline diamond bits, using air for cooling and cuttings removal was successfully used, after trials of different bit types and drilling sequences.

Whilst Cintec anchors are normally factory assembled, the size of most of the anchors on this project would not have allowed easy transport and an assembly facility was established on site. Vertical anchors of all lengths were preassembled at ground level and generally placed by crane. Horizontal anchors greater than about 6 m long had to be assembled on the scaffolding platform as they were being placed in the hole. Power, compressed air and clean water were available at all levels of the scaffolding and grout was mixed and injected at the site of the anchor hole.

Contractor procurement, Management and Cost control

The commencement of the physical works occurred immediately after the appointment of the consultant team. A cost plan that reflected the fast track nature of the project procurement was prepared with preliminary design investigation based on the indicative scope of works identified by EJE Architecture and HTL Reinhold, with provisions for undefined earthquake restoration works that would be identified as detailed investigations were made available.

The works were separated into "Trade Packages" at the initial documentation stage, the purpose being twofold:

- to enable documentation efforts of the project to be concentrated on the major structural repairs and to provide for their early on-site commencement;
- to provide control over the tendering and selection of suitable contractors having appropriate specialist expertise, experience and capacity in all required trade works.

Specialist contracts were tendered and let as Lump Sum Contracts with Schedules of Rates used for the control of project variations. Specifically within the drilling contract:

- locations for drilling were provided by survey directly under the control of the site engineer;
- maintenance of accuracy of alignment of the drill holes was documented as the contractors responsibility to be verified by progressive "as executed" survey documentation;
- down-the-hole video was used to verify the integrity of every drill hole;
- the documents provided for drilling to be undertaken in masonry having a compressive strength of up to 70MPa generally and included the onus of proof on the Contractor to provide for test data confirming any higher compressive strength;
- an issue which proved to be an item of considerable dispute and cost variation, was the internal state of some brickwork — internal mortar jointing proved in some areas to be of extremely poor quality both in terms of its low strength and the non-existence of internal joint perpends resulting in internal brickwork collapse at the drill, drill blockages and difficulties in clearing of the drilling spoil — these problems were eventually overcome by down-the-hole grout injection to stabilise the masonry.


Figure 18: Drill hole survey

The significant role implemented by the quantity surveyors was the cost management of the project budget during the construction phase using trade schedules for the major trades of brickwork, reinforcement and brickwork restoration, preparing estimates for various trade packages and monitoring of the cost plus categories of the work.

Monthly Forecast Final Cost Reports and preparation of progress payment recommendation reports for all subcontractors ensured that the client was provided with constant monitoring of the project budget and audit procedures for contract payments.

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- Mr Peter Campbell, EJE Architecture
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APPENDIX

SETTING THE EARTHQUAKE DESIGN PARAMETERS

(The following extract from Mr Trueman's progress report is important in establishing the philosophy of the building repair and is reproduced with thanks. The report was written at the time when Mr Trueman was acting in the capacity of Mediator, and had not yet been appointed to carry out the design.)

The Philosophy Behind the Earthquake Code

The Earthquake Code (AS 1170.4—1993) is unusual amongst the Codes used by structural engineers. Most Codes are concerned with safety and serviceability. The Earthquake Code is solely concerned with safety. Design to the Code does not necessarily prevent structural and non-structural damage in the event of an earthquake.

The Code is also unusual in that overseas experience was generally not relevant to Australian conditions. Most other loads and materials are similar or the same as those in other countries, and the data available is enormously increased. With little guidance on earthquakes, considerable innovation was necessary during the development of the Code. Several revisions were necessary as the resulting economic and practical effects became apparent. The final result is something of a compromise, and must be interpreted using the designer's experience and skill.

Like all Codes it is applicable to "normal" buildings. Special structures are specifically excluded, and it is only possible to use the Code recommendations as a direction or guide on non-standard structures.

Lastly it is intended for new structures. There is an informative Appendix for alterations, but little mention of repair. A new Code is under preparation for existing buildings.

Christ Church Cathedral is not a normal building in size, shape, importance or use.

The Code can only be applied as a guide, and results considered for practicability and economy. Designers will look behind the raw figures and statements for intent, rather than applying them in a rigid manner.

Regardless of the actual numbers, common engineering sense suggests that these conditions be considered whether mathematically required or not — provided the economic impact is not too onerous.

The major requirement of the Code at this level of risk for new structures is "Ductility". The ductility of a structure or element is a measure of its ability to undergo repeated and reversing inelastic deflections beyond its point of yield (elastic failure), while maintaining a substantial part of its initial load carrying capacity. In effect the ductility of a structure or element is a measure of the energy absorption capability of the system.

Ductility is highly desirable. The Earthquake Code is unashamedly intended for the protection of life by minimising the likelihood of collapse of structures. This does not necessarily prevent damage to the building structure or its attachments, or its contents. With ductility a building may be damaged by cracking but will remain standing even in an earthquake somewhat in excess of the design earthquake. As well as protecting the occupants, a building will remain that could possibly be repaired, in lieu of a possible pile of rubble.

A normal masonry building such as the Cathedral is extremely non-ductile i.e. brittle. While it is highly desirable to include at least some measure ductility in repairs, this is both difficult and costly. Yet at least in some parts of the building it is essential.

All parties to the discussion accepted the desirability of incorporating ductility with the reservation expressed by [the structural engineer representing the insurer] as to the cost involved and the responsibility of meeting this cost.

However it was decided to test the increase in costing by a rough redesign.

To convert an existing masonry building to a ductile structure is not simple. Two main alternatives exist — either providing ductile frames to support the masonry, or providing steel reinforcing to the masonry.

The first alternative — additional frames — is unacceptable in a building such as the Cathedral, both for the effect on appearance and perhaps the heritage significance of the building.

Installing reinforcing is the logical answer and was the original method proposed by both consultants, although based on different loading parameters and a slightly different philosophy.

Design parameters

For a rough redesign it was necessary to establish uniform design parameters to allow a direct comparison of the resulting effect on the original proposals.

It is not proposed to detail here the parameters used because, as previously discussed, they remain only guidelines to engineering judgement.

It is not proposed to detail here the parameters used because, as previously discussed, they remain only guidelines to engineering judgement.

It is sufficient to say that the parameters discussed, and eventually used, were considered relative to one another and their individual level of risk. For instance where one factor was considered conservative it was reasonable to be less conservative on another.

Additionally the effect on economics of increased loads was considered. There are strong reasons for being conservative for a building of the importance of the Cathedral, with an anticipated long life span under normal loads. The question of the responsibility for the payment of the increased cost must be considered. It is accepted in the conservation of very significant heritage buildings which have deteriorated from age, that the cost of rehabilitation should be at the expense of the community rather than the owner. It is felt that this should apply to this Cathedral were it to be upgraded to a higher standard befitting its anticipated life span. Normal buildings are designed for a 50 year life span and hence the Earthquake Code is based on a seismic event that has a 90% probability of not being exceeded in 50 years. If the same probability were extended to a longer life, there is a resulting cost to someone.

If this cost is to fall on the general community, the effect of accepting such a principle must be compared to the effect on other similarly important buildings and structures. On this basis, the suggested Cathedral design loads were limited to those applicable to a building that has an important post-disaster function.

At the same time consideration was given to other factors that influence the design process. These included the factors for loads and materials in various Codes to cover the intangible variations that occur. Examples are uncertainty of material quality, of workmanship and even the accuracy of the measurement, or prediction, of loadings. There were reservations about the final resulting parameters by [both the engineering consultants]. Selection of criteria being an art, there will always be different opinions.

In searching for guidance from overseas on selection of parameters for earthquake repair of important buildings such as the Cathedral, some New Zealand examples were considered relevant. The established standard for strengthening of old buildings in New Zealand accepts a relatively higher risk due to the costs involved. The value of earthquake load suggested to be used is equal to one half to two-thirds of those prescribed. However for important long life buildings such as the High Court Building in Auckland it was felt that an attempt should be made to comply if possible with the "spirit and intent" (i.e. philosophy) of their Earthquake Code. This has therefore been the aim with Christ Church Cathedral.

Repairs to heritage buildings are often difficult to achieve without affecting the quality of the building. As the standard of workmanship and the quality of materials are known, it is usual to reduce considerably or remove the risk factors associated with these elements. A similar philosophy was applied in this case.

It was agreed to undertake the rough design using the chosen parameters. From the results it would be possible to see two things - the effective variation of cost incurred by the change from the preferred criteria of each consultant, and any variation in construction methods proposed.

Despite adopting uniform design parameters the application of parameters was left to the discretion of the designer. As previously discussed mathematical analysis is only a guide and the resulting stresses must be examined and interpreted using experience and understanding of structural mechanics. For instance, there may be areas where reinforcing is theoretically not required, but should be installed due to the likelihood of stress concentrations, or because of the risk associated with failure.

The resulting rough designs were found to be quite compatible, and not extraordinarily different to the original layouts.

There were, however, differences in approach that may be affecting estimates.

ADOPTED EARTHQUAKE CODE PARAMETERS

Acceleration Coefficient	A	=	0.11
Importance Factor	I	=	1.25
	or		a=0.11 x 1.25
(The latter has the same result but a different philosophy)			
Site Factor	S	=	1.5
Load Factor (for limit state design)	F	=	1.0
"Materials" Factor or Strength Reduction Factor	C_r or O	=	1.0

Seismic Tests to a Full Scale Monument Model of part of the São Vicente de Fora Monastery, in Lisbon.

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SEISMIC TESTS ON A FULL SCALE MONUMENT MODEL

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ABSTRACT

Pseudo-dynamic and cyclic tests on a full-scale model of part of the cloisters of the São Vicente de Fora Monastery, in Lisbon, are reported. After a first set of pseudo-dynamic and quasi-static cyclic tests performed on the original model, where local damages were observed, the model was retrofitted with four internal continuous bond anchors. A final campaign of quasi-static cyclic tests was carried out on the retrofitted model in order to investigate the effects of this retrofitting technique. The tests were carried out at the ELSA laboratory, JRC, and aimed at the characterization of the non-linear behavior of limestone-block structures under earthquake loading. Moreover, the assessment of the effectiveness of the retrofitting system was also a major objective of the test campaign. Local and overall stability of the stone-blocks, including columns and arches were assessed for large displacement amplitudes. Comparisons between the tests results before and after retrofitting allowed investigating the applicability of these retrofitting solutions and techniques for monumental structures.

1. INTRODUCTION

Public bodies in charge of the maintenance and preservation of cultural heritage are more and more expressing their willingness to assess the seismic vulnerability of important monumental structures and to develop suitable strengthening and retrofitting techniques. The joint research project, *the COSISMO project*, on seismic analysis/assessment of monuments is a first attempt to contribute to the required progress in the field. The Portuguese General-Directorate for National Buildings and Monuments (DGEMN) and the National Laboratory Civil Engineering (LNEC), in Lisbon, and the Joint Research Centre of the European Commission (JRC) setup a research programme including the following main tasks: 1) Dynamic characterization of a representative monumental structure, the São Vicente de Fora monastery in Lisbon, by *in situ* testing and numerical modelling; 2) Laboratory testing of a representative model of part of the structure, which will enable the calibration and/or development of non-linear numerical models to be used for predicting the earthquake response of such structures; 3) Development and calibration of non-linear and equivalent linear models appropriate for high intensity shaking; 4) Assessment of the seismic vulnerability of the Monastery, using the developed and calibrated models and appropriate seismic hazard characteristics; 5) Investigation of the applicability of some retrofitting solutions and techniques for monumental structures.

This paper focuses on the laboratory tests on a full-scale façade model carried out at the ELSA Laboratory under the framework of the above-mentioned project. In particular, to the recently performed tests on the retrofitted model is devoted special attention.

2. STRUCTURE AND TEST MODEL

The S. Vicente monastery (Fig. 1) represents, from the architectonic/engineering point of view, a typical monument of Lisbon, where limestone block masonry columns and arches, forming a resistant structure, are harmoniously combined with stone masonry bearing walls and ceramic dome floors/roofs.

The monument survived the catastrophic November 1st, 1755 earthquake; however, some of the effects of the strong ground shaking are still visible today. The 2 meters thick south external wall of the monastery became curved (mid-span dislocation of about 40 cm); the same happened to the west end-side external wall; and the East end-side edifice, where the Pantheons are presently located, collapsed. A quite detailed description of the damages inflicted to the monastery during the 1755 earthquake is available. Prediction of the damages to the monastery using the present modelling capabilities calibrated on the basis of the experimental results obtained by 'in situ' tests and by laboratory tests is therefore a great challenge.

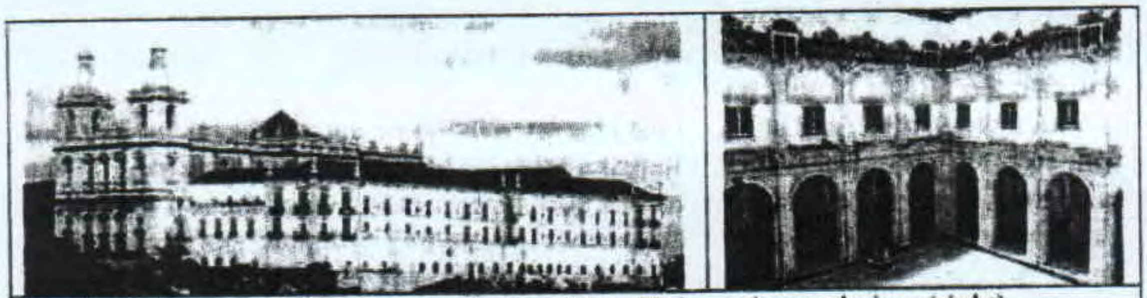

Fig. 1. S. Vicente de Fora: General view (left), Cloisters internal view (right)

Test model

The test model (Fig. 2) was built using materials and construction techniques (stone blocks arrangement) similar to the prototype cloister facade. It is a plane structure with three stone block columns, two complete arches and two external half arches. The upper part of the model is made of stone masonry. Three millimeters thick mortar joints were assured during the construction. The model was defined taking into account the following two main aspects: it should represent typical monumental structures and it should reproduce the construction techniques (realistic, in terms of materials, scale and stone arrangements).

The 'retrofitting' technique

The model was retrofitted with four internal continuous bond anchors, two at each level with 2 meters overlapping (Fig. 2). The anchors were placed in horizontal holes drilled from each end side of the model and the grouting was carrying out in two phases. First, anchorage of the anchor was guaranteed. Then, a pre-compression of 20 kN was imposed in each steel bar and the finally grouting of the holes was done.

3. TESTING OF THE FULL SCALE MODEL

The question of using a scaled model has been raised many times by the team involved in this project and it was agreed to work with a full-scale partial test-model. It was additionally required a model to which simple and realistic boundary conditions could be applied. After several numerical simulations with existing models, see Pegon & Pinto (1998), it was decided to adopt the model given in Fig. 2. Even the boundary conditions for a long façade (periodic structure), which include equal displacement of the two lateral end-sides, zero rotation and equal vertical displacements, would had rendered the test set-up very complex. On the basis of the numerical simulations, it was concluded that post-tensioning of the two opposite end-sides of the model would lead to suitable test conditions. Moreover, such testing conditions may represent two distinct

The drilling and installation of Cintec Reinforcement Anchors retrofitted to full-scale model.

parts of a long façade: one internal (internal column and the two internal half-arches) with period boundary conditions; and the two lateral parts of the model (external columns) representing the external parts of the edifice.

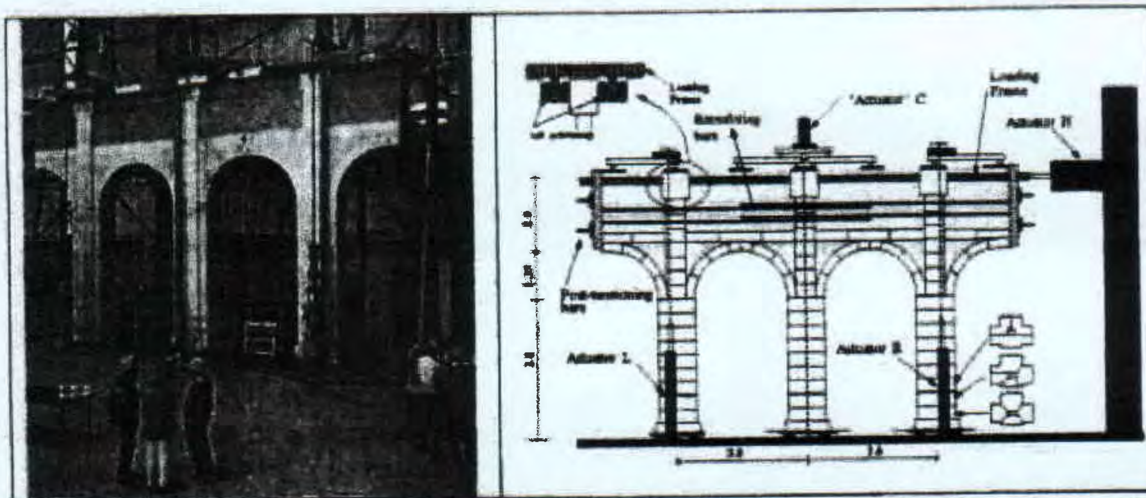

Fig. 2. Façade full-scale model in the ELSA reaction wall laboratory (left) and test set-up - schematic (right)

In addition to the lateral boundary conditions, the upper part of the façade (the height of the stone-masonry wall) was also investigated; obviously, it depends on the kind of existing opening (door, window or no opening). In the S. Vicente Monastery, at the second floor level of the cloisters exist windows, which justifies a model with a medium height wall.

Test set-up, loading devices and measuring system

Having defined the physical model and the required boundary conditions a second phase should be undertaken, which is the definition of realistic loading conditions. Two loading types are considered: the vertical loading due to the remaining upper part of the monument; and the horizontal loading resulting from earthquake excitations.

Concerning earthquake loading two important conditions should be guaranteed namely a rather uniform distribution of forces and a deformation pattern similar to the one resulting from the long façade. Therefore, overturning moment due to horizontal forces applied at the top of the model (see Fig. 2) must be compensated by means of the vertical servo-actuators located close to the columns of the model. In order to simplify the testing apparatus a constant vertical force at the central actuator was further assumed. Therefore, in addition to the vertical forces due the missing upper part of the monument, the two vertical external actuators must impose a deformation pattern dictated by the *shear-like* deformation. The control of the two external actuators is performed by imposing the following two (displacement and force) conditions

$$d_L = d_R \quad and \quad F_L + F_R = F_V \qquad (1)$$

where d denotes the vertical displacement at the top of the column, the subscripts L and R stand for Left actuator and Right actuator, respectively and F_V is the total constant vertical force representing the weight of the 'missing upper part'. This additional load at the top of each column was estimated in 440 kN.

As anticipated above, the additional vertical loads represent the weight of the upper part of the monument, not reproduced in the physical model. Hence, distribution of these loads should reflect the real conditions. For a homogeneous structure a uniform force distribution would be appropriate, but, in our case, a very stiff column-arch structure (limestone blocks) is combined with masonry walls (stone masonry with poor mortars) leading to non-uniform distribution of loads. Numerical simulations with linear and non-linear models indicated that a distribution of forces, in the column and masonry parts represents quite accurately the real situation. Values of 400 and 100 kN were estimated for column and masonry parts, respectively.

The application of horizontal forces, resulting from earthquake excitations was made through an 'original' loading system, which provides uniform distribution of forces. As shown schematically in Fig. 2 (detail at top-left-side), the horizontal forces are equally distributed because they are transmitted from the loading frame to the model through water-pad bearings, which are interconnected. In fact, at each transversal beam of the loading frame, the Left-side pad bearings (L bearing in Fig. 2 – detail) communicate. Therefore, the same pressure develops and the force is proportional to the area of the pad bearing. When the horizontal loading is applied in the left to right sense, the same happens to the right side bearings, R, in Fig. 2). Such a system guarantees a pre-defined distribution of horizontal forces and avoids unrealistic local deformations at the zones of application of loads. In spite of the long and 'flexible' steel-loading frame, a specified distribution of forces can be imposed.

Concerning the measuring system designed for this test, the following types of measurement were adopted: 1) at the base of the columns, underneath the steel base plate, load cells were placed in order to obtain the evolution of vertical forces and the column-base flexural moments; 2) rotations of the column stone-blocks were measured along the height; the horizontal deformation pattern of the columns (internal and one external) can be also obtained from the set of horizontal displacement traducers placed at several levels; 3) deformation of the arches can be derived from the 3 transducers located at the contact zones and from global displacements of the arch stones; 4) vertical and horizontal relative displacements at the key-stone of the arches were also recorded; 5) deformation of the masonry part of the model (upper part) can be derived from the displacement transducers placed in this zone;6) the forces in each pre-stressing horizontal bar were monitored; in addition, application of forces (horizontal and vertical actuators) were continuously recorded as well as the controlling displacements.

4. TESTING PROGRAMME AND TEST RESULTS

Several tests were envisaged for this model apart from the initial dynamic characterization tests in order to obtain frequencies and mode shapes and evaluate damping for very low deformation levels (microns) and the initial stiffness tests. First, two pseudo-dynamic tests were performed for earthquake intensities corresponding to a low and a medium value of return period and then the model was subjected to cyclic tests with pre-defined displacement histories. After these tests the model was retrofitted and more cyclic tests were performed. The cyclic tests with pre-defined displacement histories performed on the original and retrofitted models permitted to investigate the influence of boundary conditions on the results, namely the post-tensioning forces in the horizontal ties, and the effectiveness of the retrofitting technique.

Pseudo-dynamic tests

From the dynamic characterization tests and the stiffness tests, initial stiffness was obtained, which, in conjunction with the required initial frequency of the model, dictated the mass to be used in the pseudo-dynamic tests. It is noted that for the pseudo-dynamic tests a one-degree of freedom system (1DOF) was considered. It is well known that the pseudo-dynamic test method allows such an uncoupled definition of the stiffness and mass characteristics, which is not possible in a truly dynamic test. Taking advantage of this feature, the frequency of the system was set to 4Hz, because, the experimental and analytical studies identified such a frequency value (Dyngeland *et al*, 1998). In fact, the results of the *in situ* dynamic tests of the Monastery (Campos-Costa *et al*, 1996), and of the numerical dynamic linear analyses found that the frequency corresponding to the first mode of vibration involving global deformation of the cloisters (façade) is approximately 4Hz. Consequently, a mass value of m = 400 tons was adopted for the equivalent 1 DOF system.

Two pseudo-dynamic tests were initially envisaged, corresponding to moderate and high earthquake intensities. Moreover, two types of earthquake loading were used, because two earthquake scenarios should be considered for Lisbon (far-field and near-field) with rather different spectral energy content. As shown in Fig. 3, which plots the displacement response spectra (5% damping) for the two earthquake types and for two return periods (174 and 975 years), the near-field accelerogram response spectrum contains energy in the higher frequency ranges (frequencies higher than 2 Hz), while the contrary happens for low frequencies. Based on these spectral differences and on the pre-test numerical simulations, the following strategy for the pseudo-dynamic tests was adopted. The low-level seismic test was carried out with a near-field type accelerogram (174 years return period) and the high level test was performed with the far-field 975 years return period accelerogram. The accelerograms used in the tests are shown in Fig. 3. The near-field signal, for the low-level test, is a 10 seconds duration while the high-level tests were performed for 30 seconds duration accelerograms. After the high level test, the input signal was multiplied by a factor of 1.5 and a new pseudo-dynamic test was carried out.

The results from the pseudo-dynamic (PSD) tests, in terms of global force displacement diagrams, are given in Fig. 4 for the low-level (LL) and high-level (HL) tests. It must be underlined that the LL test was carried out with a pre-compression force of about 50kN, while the HL test was carried out with a much higher compression force (350kN). During the first PSD test, the LL test, which reached a top displacement less than 8 mm, the moderate value of the compression force lead to the cracking of the model between the column and the masonry. Due to this, the model lost part of its structural 'framing' capacity and the horizontal resisting force dropped down. This case may be considered as representative of a structure without ties located at the extremity of an edifice. In order to avoid premature collapse without exploring other deformation mechanisms, the high level tests were performed with much higher compression forces. Hence, the pre-compression action through the steel bars must impose a higher strength of the upper part of the model in order to explore the 'deformation capacity' of the columns.

It is apparent from the diagrams presented in Fig. 4 that for the high-level earthquake tests the structure reaches its strength capacity and maintains it even for high deformation levels. The butterfly-type diagram is typical of these structures (a very high stiffness for low deformation levels and a 'sudden' decrease for important deformations); however, as it will be discussed latter on, the equivalent damping is quite important for the high level tests. It should be noted that the second high-level test (1.5 times the initial accelerogram (1.5HL)) led to diagrams and mechanisms similar to the HL test and to top displacements proportional to the input intensity. Peak values of top displacement of 43 mm and 64 mm were respectively recorded for the HL and the 1.5HL earthquake tests. There is another important aspect reflected in the energy diagrams given in Fig. 4. In fact it is apparent that the dissipated energy is, for all seismic tests, directly proportional to the cumulative displacement, which confirms that energy dissipation results from friction.

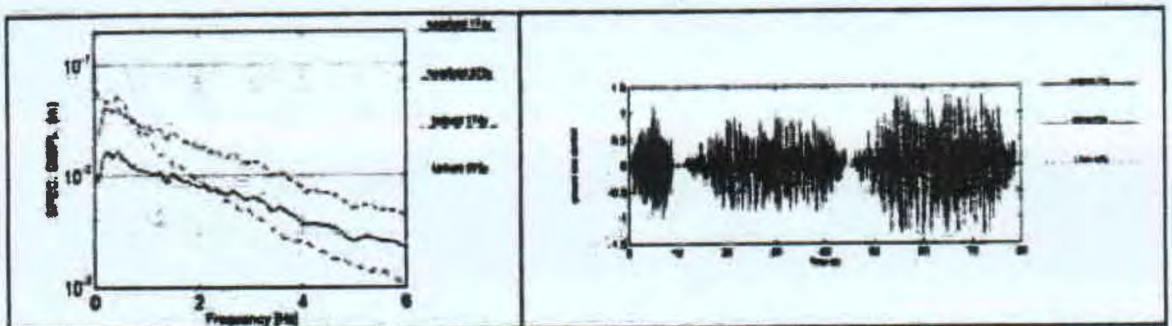

Fig. 3. Accelerograms used and Spectral displacements for far-field and near-field earthquake samples.

As already mentioned, one of the aims of the research project was to identify suitable and realistic parameters to be used in equivalent linear analyses. To this end, the earthquake test results were analyzed using time-domain identification methods (Molina and Pegon, 1998; Molina et al., 1999) and the outcome of this study is shown in Fig. 5. The left column shows the displacement response for the three earthquake tests and the corresponding evolution of eigen-frequencies and equivalent damping. The right column shows the relationship between the displacement amplitude and the eigen frequency and between the displacement amplitude and the equivalent damping. It is apparent that a linear relationship holds between displacement amplitude and equivalent frequency. Moreover, values no lower than 5% were computed for the equivalent damping for very small displacement amplitudes and the equivalent damping reaches quite high values (7–12%) for medium amplitude levels.

Quasi-static cyclic tests (original and retrofitted models)

Cyclic quasi-static tests were performed on the original and on the retrofitted model in order to obtain data to calibrate analytical models, to study the effect of the pre-compression forces on the behavior of the façade and to study the effectiveness of the retrofitting bars. Two cyclic tests were performed on the original model and three were performed on the retrofitted one. The imposed increasing-displacement histories had two constant amplitude cycles for each level and rang from 8 to 100 mm.

The first cyclic test on the retrofitted model was carried out with the pre-compression force of the last cyclic test carried out on the original model (175 kN). A displacement controlled time history was imposed with amplitudes ranging from 8 to 30 mm. Then the pre-compression force was reduced to 45 kN and a displacement controlled time history was imposed with amplitudes from 30 to 100 mm. Finally, no pre-compression was applied and a new displacement history was imposed, these time with amplitudes ranging from 8 mm to 60 mm.

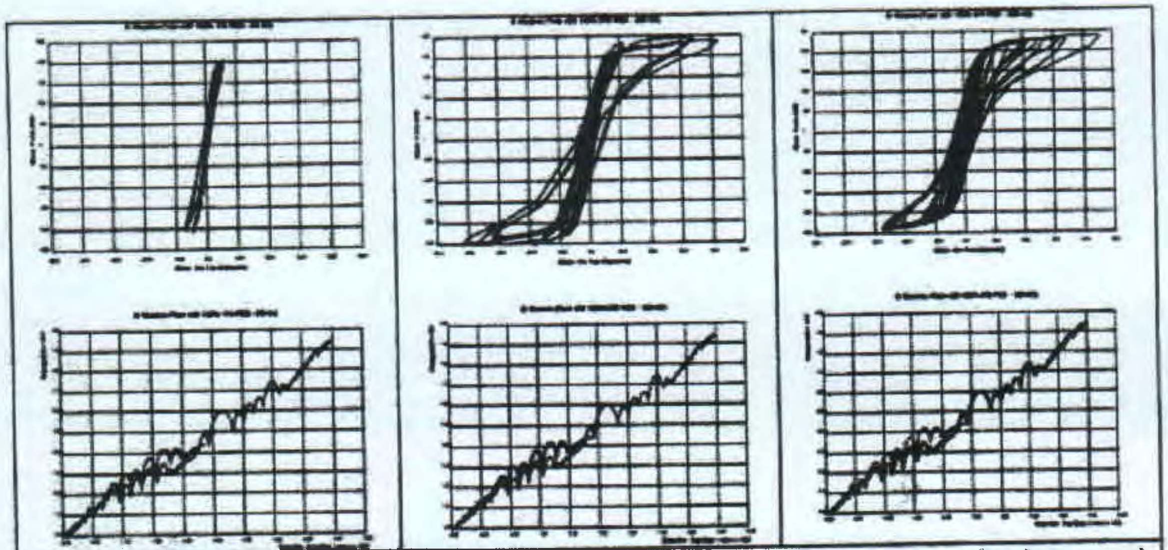

Fig. 4. Force-Displacement and Energy-Displacement (cumulative displacement) diagrams for the low-level (*Column 1*), high-level (*Column 2*) and 1.5x high level (*Column 3*) earthquake tests

Force-displacement diagrams for the cyclic tests are given in Fig. 6. The type of the diagrams is apparently the same for both cyclic tests and also for the high-level earthquake test (see Fig. 4). However, a slight difference exists between the test with high compression forces and with the medium compression forces. Both situations develop equivalent strengths for the maximum amplitude (100mm). The main difference between the two comparable cyclic tests (only the pre-compression forces are different) is the smoother transition between the two stiffness (closing and opening of the column-block joints). Therefore, one may conclude that 'a minimum' pre-compression level should be applied in order to maintain stability of the upper part of the façade and to improve deformation capacity of these structures. Furthermore, it was verified that compression forces higher than a minimum limit do not improve significantly the cyclic performance of the structure, for the experienced deformation amplitudes.

The results from the tests on the retrofitted model (with continuous-bond anchors), in terms of force-displacement, are given and compared, with the case with low compression forces, in Fig. 6 (right). Similar diagrams were obtained for both cases. However, the retrofitted case showed very important differences in terms of elongation of the upper part of the model. This is apparent in Fig. 7a), which presents the evolution of the elongation of the upper part of the model for the retrofitted and the other comparable cases. Furthermore, the initial length is almost completely recovered in the original models whilst a final permanent deformation is observed in the continuous-bond anchors case. It is shown in Fig. 7c) and 7d) that the opening takes place mainly at the masonry-column interface for all cases; i.e., deformation patterns and mechanism are similar for all cases. Therefore, the stiffness and strength of the anchors passing through the interface zones control this opening. The continuous-bond anchor (one 20mm diameter bar) and the passing bars (two 36 mm diameter bars) can develop quite different stiffness and strengths, which depend on the steel section, the bond length and the steel yielding strength. It is suppose that strong nonlinear deformations were experienced in the continuous bond anchor. However, the ultimate deformation capacity was not reached during the tests.

Furthermore, the model has shown important dissipation characteristics due to cracking and friction. Cracking appeared at the interfaces block–masonry in the arch zones and at the interface between the upper part of the columns and the masonry (see Fig. 8). Such dissipation characteristics for important deformation levels exist thanks to the tie bars (pre-compression forces), which allow developing the required confining of the upper masonry part of the façade model.

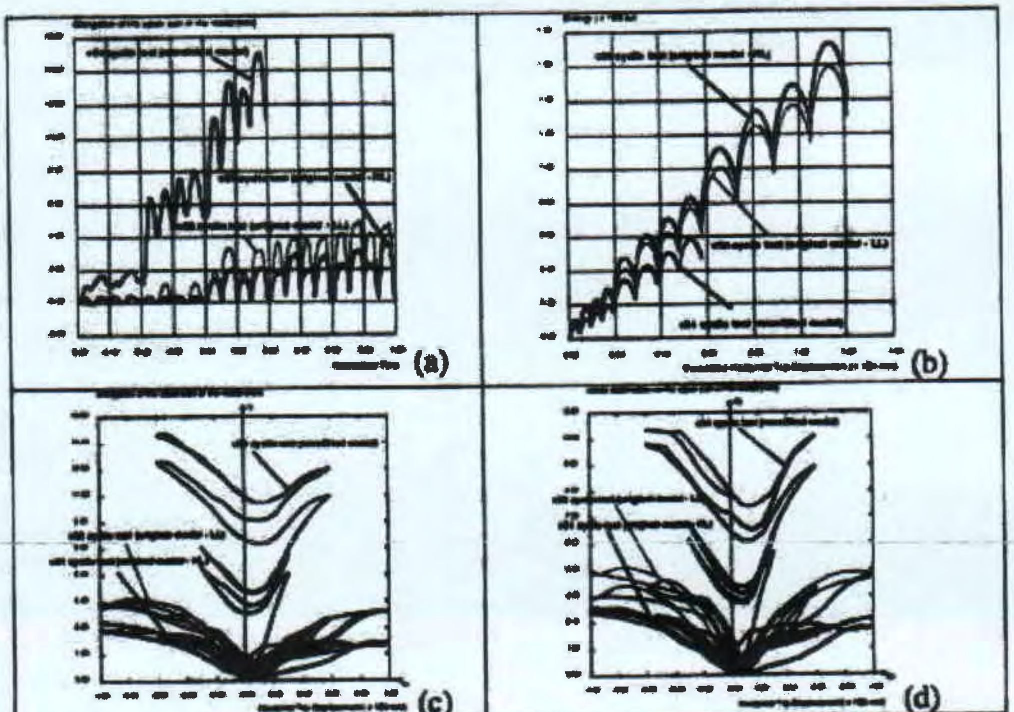

Fig. 5. a) evolution of the elongation of the upper part of the model; b) energy dissipation; c) evolution of the elongation of the upper part of the model; d) evolution of the total joint openings in the upper part of the model.

'Only' local damages were observed during the tests, namely slight dislocation of column and arch stone-blocks (15 mm maximum value), crushing and delamination of stone blocks at the most stressed contact zones, large cracks in the masonry between contiguous arch-bases and passing through the upper columns and failure (spalling) of a few limestone cover plates. Good deformation and dissipation characteristics of this type of structures are expected if a rational distribution of ties at the floor levels exists. Design and practical application of these tie systems using new analysis tools and construction techniques are under investigation. The test campaign carried out on the retrofitted model showed that the continuous-bond anchors have a rather good performance comparable with the one of the pre-compression ties. Moreover, it was apparent a 'better distribution' of the cracking in the upper part of the model. In fact, distributed cracks appeared in the stone-masonry wall and the cracks in the masonry-column interface. These tests have shown the applicability and effectiveness of such a kind of retrofitting in terms of deformation capacity and strength of the model. Another important issue was the performance of the system in the anchors overlapping zone (2m overlapping). It was observed that no damages or loss of bond appeared in this overlapping zone. Therefore, one may assume that such a system can be applied drilling

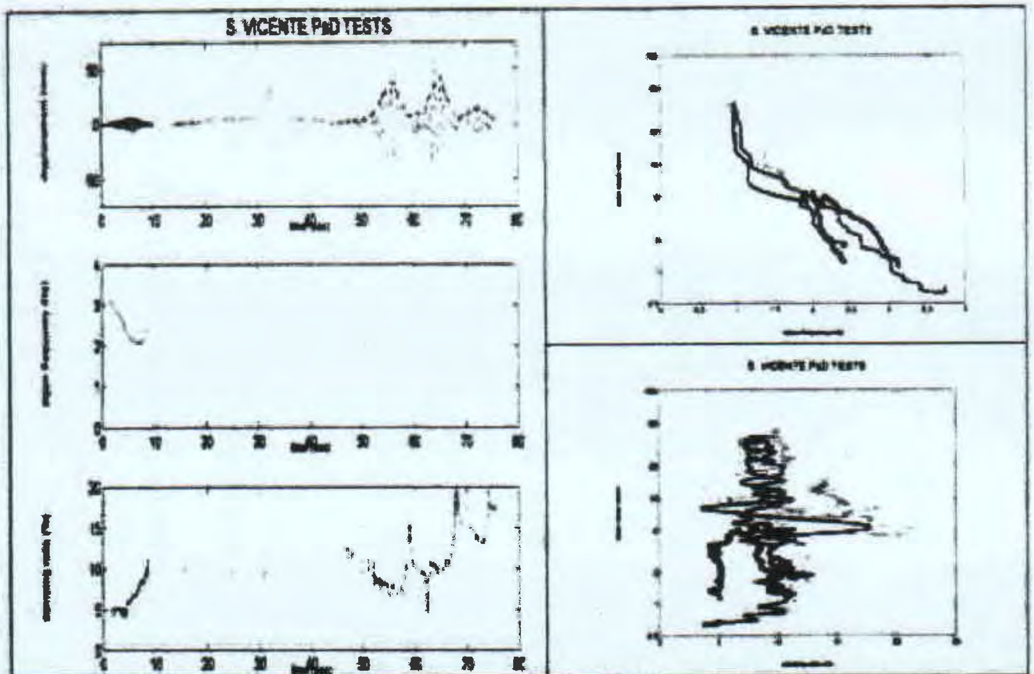

Fig. 5. History of the response to the three imposed earthquakes (left), correlation between displacement amplitude and natural frequency as obtained from the experimental response to the three earthquakes (top right) and correlation between displacement amplitude and equivalent viscous damping ratio as obtained from the experimental response to the three earthquakes (bottom right).

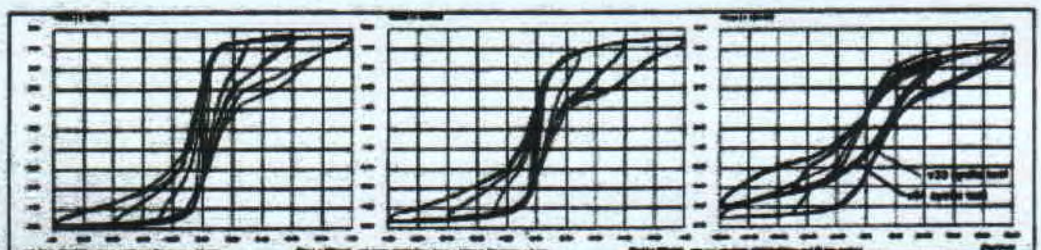

Fig. 6. Force displacement diagrams for the cyclic displacement controlled tests on the original model with high pre-compression forces (*left*), with low pre-compression forces (central) and comparison between the original low compression forces and the retrofitted cases for displacements up to 60 mm (*right*).

As shown in figure 7b), the energy dissipation is comparable for all cases. In addition, the dissipation mechanisms are similar. Therefore, one may conclude that both solutions are effective. The open issue is to develop suitable models for the design (dimensioning) of the anchors.

5. FINAL REMARKS

The tests on the model of the S. Vicente façade of the cloisters have shown the deformation capacity of the column-arch system commonly used in many monumental structures. Drifts of about 2% were imposed without loss of the load carrying capacity.

holes with rather small diameters from both sides of the construction, which is an important advantage. There remains however the question of reversibility of the retrofitting solution, which should be taken into account.

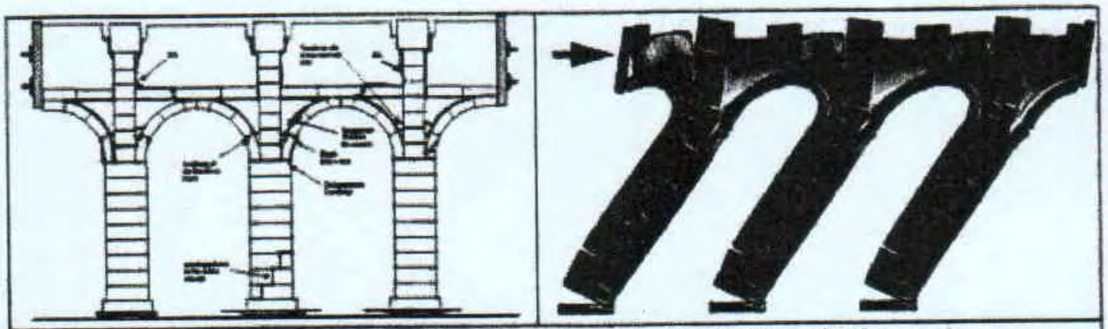

Fig. 8. Deformation pattern and damages (*top*)- analytical result (*bottom*)

6. ACKNOWLEDGEMENTS

The Portuguese General Directorate for National Buildings and Monuments (DGEMN) financed the work herein presented. The DGEMN and the National Laboratory for Civil Engineering (LNEC), in Lisbon, and the Joint Research Centre of the European Commission (JRC, Ispra, Italy) jointly developed the project COSISMO. The contribution given by Professor C. Sousa Oliveira, IST-Lisbon, to the project definition and development is gratefully acknowledged. Also, the co-operation established with Dr. C.T. Vaz and Dr. E.C. Carvalho from the LNEC in the model construction and testing is underlined. Gratitude is also expressed to P. Pegon from the JRC, for the decisive contribution, through numerical studies, to the definition and preparation of the model and the laboratory tests including the study of the retrofitted model.

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CINTEC ANCHOR TESTING

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LOAD TEST REPORT ON CINTEC GROUTED ANCHORS

DATE:	**FEBRUARY 7, 2001**
PROJECT NO:	**2001-5037**
TLSC NO:	
CLIENT:	**MISSION SAN JUAN CAPISTRANO** **ATTN: GERALD MILLER, ADMINISTRATOR** **PO BOX 697** **SAN JUAN CAPISTRANO, CA 92693**
PROJECT:	**STABILIZATION OF THE STONE WALLS** **OF THE GREAT STONE CHURCH** **MISSION SAN JUAN CAPISTRANO, CA**
DATE OF TEST:	**FEBRUARY 5, 2001**
PERFORMED BY:	**EUGENE RAYMUNDO, STAFF ENGINEER** **OSCAR SANCHEZ & KENT GRAY, TECHNICIANS**

Transmitted herewith is the Load Test Program for Cintec Grouted Anchors of the Old Stone Church at Mission San Juan Capistrano. This test program is in accordance to the Roselund Engineering Company Test Program of Cintec Grouted Anchors. This report contains Testing Requirements, Calibration Chart (Aluminum Dual Ram), Load Deformation Curves, and Photographs.

SUBMITTED BY: TWINING LABORATORIES OF SOUTHERN CALIFORNIA , INC

Prepared by:

Raymundo

Eugene Raymundo
Staff Engineer

Reviewed by:

Elmer D. Marapao, PE
Civil Engineer

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TEST REPORT ON CINTEC GROUTED ANCHORS
OLD STONE CHURCH, MISSION SAN JUAN CAPISTRANO

Purpose of Test

Twining Laboratories conducted a pull test program of the Cintec Grouted Anchors in the walls of the Old Stone Church, San Juan Capistrano to observe, evaluate, and tension-test of the anchors in order to establish the suitability of the anchors for bonding of the inner and outer wythes of the stone walls to prevent internal separation in the walls.

Location of Tests & Test Setup.

A total of thirteen (13) anchors were installed by Cintec by drilling with a 1 ¼" diameter dry-diamond core drill and inserting a Stainless steel pins with a fabric grout sock, then a Presstec cementitious grout was applied by Cintec. Grout was allowed to cure undisturbed for a minimum of 64 hours prior to testing.

Two test assembly were used during the test program: 1). Hilti Mark V Tester Kit or 2). Aluminum Dual Piston Ram by Twining Laboratories with calibrated Pump and Gauge. A digital strain gauge (0.0001") Mitutuyo displacement gauge was installed into the 5/16" threaded adapter to record displacement of the stainless steel pins. Each test location was marked and selected by Mr. Nel Roselund of Roselund Engineering Company. A preload of 200 lbs. was applied prior to actual load testing at 200 lbs. increments. Visual observation was performed at each 200 lbs. increments to observe any pin movement, or mortar crumbling.

Test Results and Photographs
(Individual Test Data, Load Deformation Curves, and Photographs are enclosed)

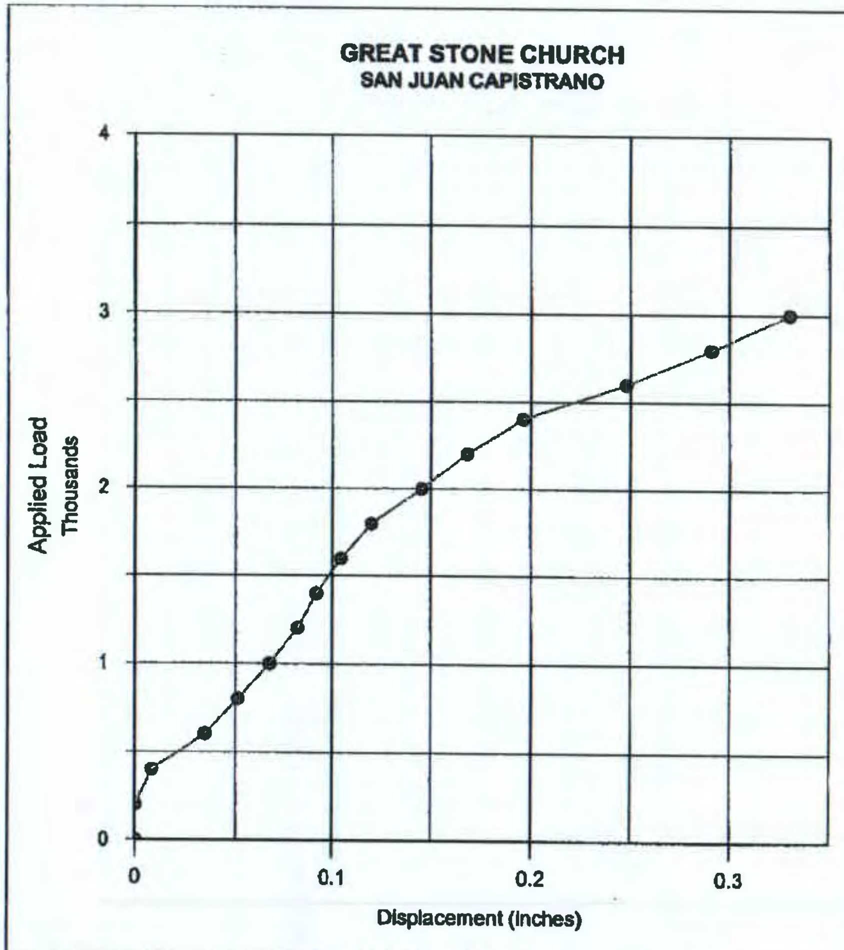
Modification
During the actual testing, weld failure at the coupler were encountered on Test #2 (3000 lbs.) and Test # 4 (1800 lbs.), we recommended to remove the displacement gauge at a load of 3000 lbs. or any sign of weld failure to avoid any damage of the electronic displacement gauge. Further visual observation will be performed as we continue applying the load to a maximum of 3400 lbs.

PROJECT: GREAT STONE CHURCH, SAN JUAN CAPISTRANO

TEST DATA FOR CINTEC GROUTED ANCHORS

TEST LOCATION No.	1	No. of Anchors	1
TEST SET UP:	DUAL RAM-TLSC	Max. Spec. Load	1900 lbs
DATE	02/05/01	D. GAUGE	MITUTUYO

Applied Load, lbs.	Measured* Displacement	OBSERVATION
0	0.0000	Displacement sensor mounted on track.
200	0.0000	Preload, release load and zero the displacement gauge
400	0.0085	
600	0.0350	
800	0.0515	
1000	0.0675	
1200	0.0820	
1400	0.0915	
1600	0.1045	sleeve anchor beginning to slip from grout sock
1800	0.1200	
2000	0.1455	
2200	0.1680	
2400	0.1960	
2600	0.2480	
2800	0.2910	
3000	0.3305	displacement gauge removed prior to weld failure.
3200		
3400		maximum load, prior to release of load

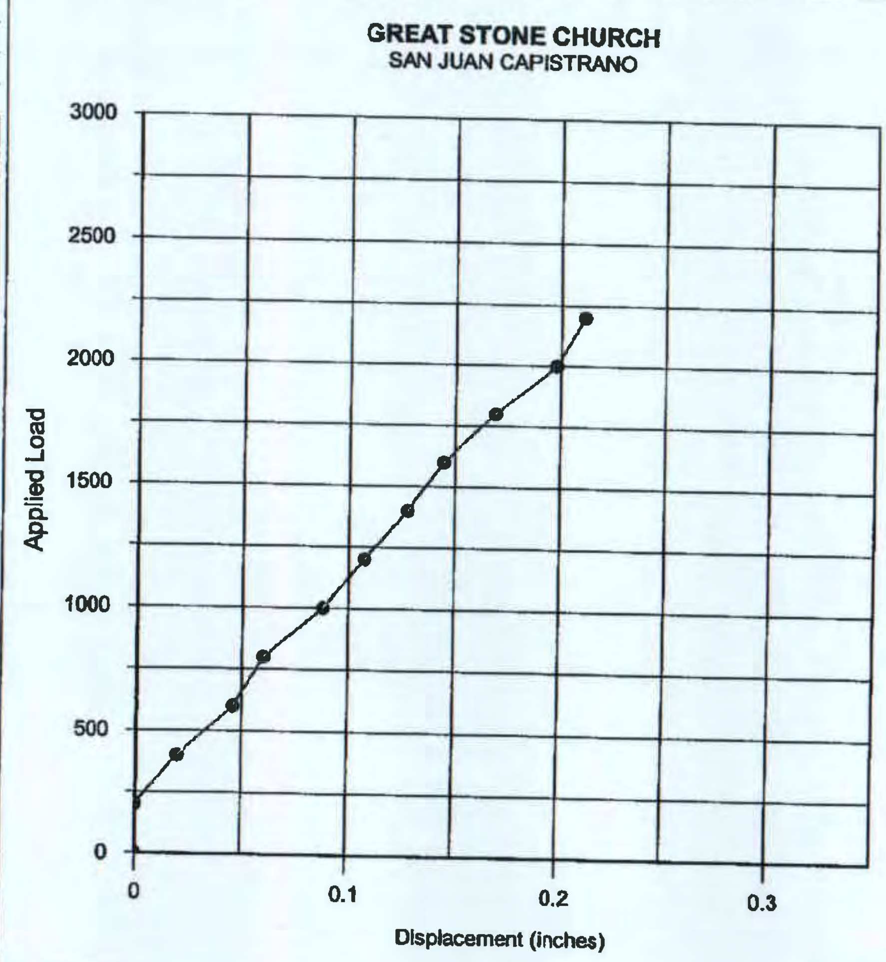


PROJECT: GREAT STONE CHURCH, SAN JUAN CAPISTRANO

TEST DATA FOR CINTEC GROUTED ANCHORS

TEST LOCATION No.	6	No. of Anchors	1
TEST SET UP:	DUAL RAM-TLSC	Max. Spec. Load	1900 lbs
DATE	02/05/01	Disp. Gauge	MITUTUYO

Applied Load, lbs.	Measured* Displacement	OBSERVATION
0	0.0000	Displacement sensor mounted on track.
200	0.0000	Preload, release load and zero the displacement gauge
400	0.0195	
600	0.0455	
800	0.0600	
1000	0.0880	
1200	0.1075	
1400	0.1275	sleeve anchor beginning to slip from grout sock
1600	0.1445	
1800	0.1690	
2000	0.1980	
2200	0.2110	displacement gauge removed prior to weld failure.
2400		
2600		
2800		maximum load, prior to release of load

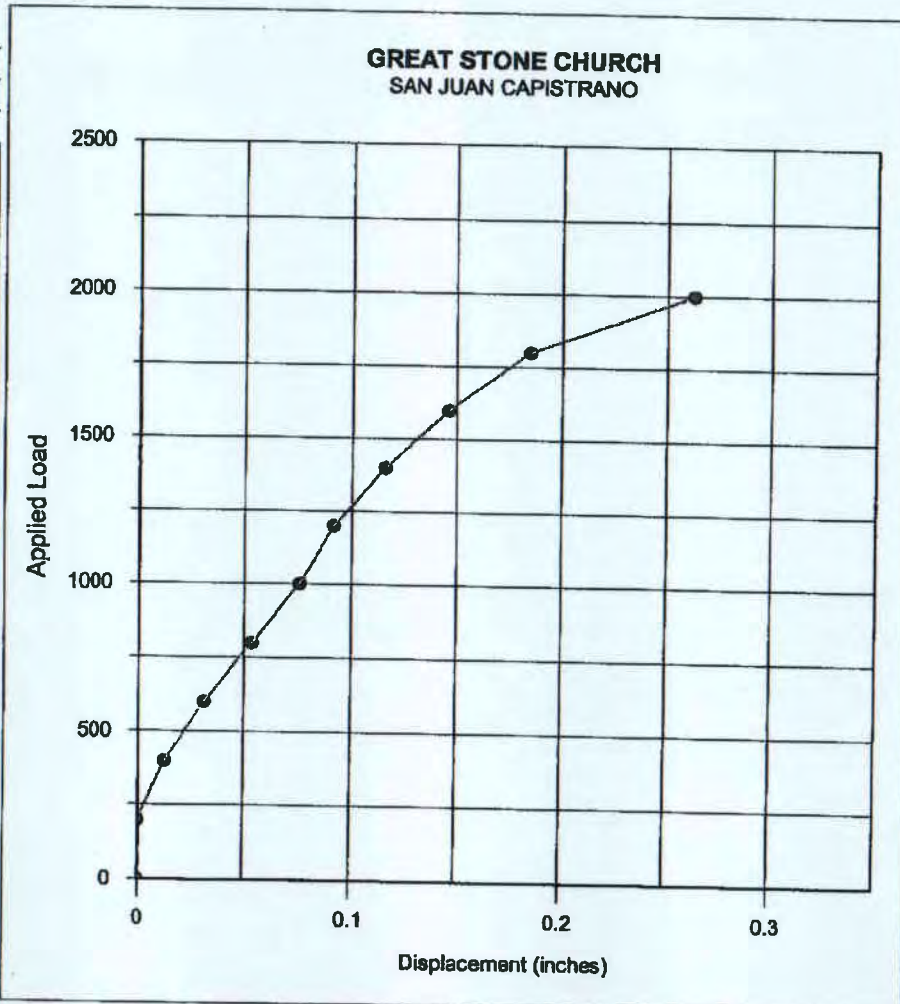


PROJECT: GREAT STONE CHURCH, SAN JUAN CAPISTRANO

TEST DATA FOR CINTEC GROUTED ANCHORS

TEST LOCATION No.	7	No. of Anchors	1
TEST SET UP:	DUAL RAM-TLSC	Max. Spec. Load	1900 lbs
DATE	02/05/01	Disp. Gauge	MITUTUYO

Applied Load, lbs.	Measured* Displacement	OBSERVATION
0	0.0000	Displacement sensor mounted on track.
200	0.0000	Preload, release load and zero the displacement gauge
400	0.0130	
600	0.0315	
800	0.0535	
1000	0.0755	
1200	0.0915	
1400	0.1160	sleeve anchor beginning to slip from grout sock
1600	0.1460	
1800	0.1845	
2000	0.2630	displacement gauge removed prior to weld failure.
2200		maximum load, prior to release of load

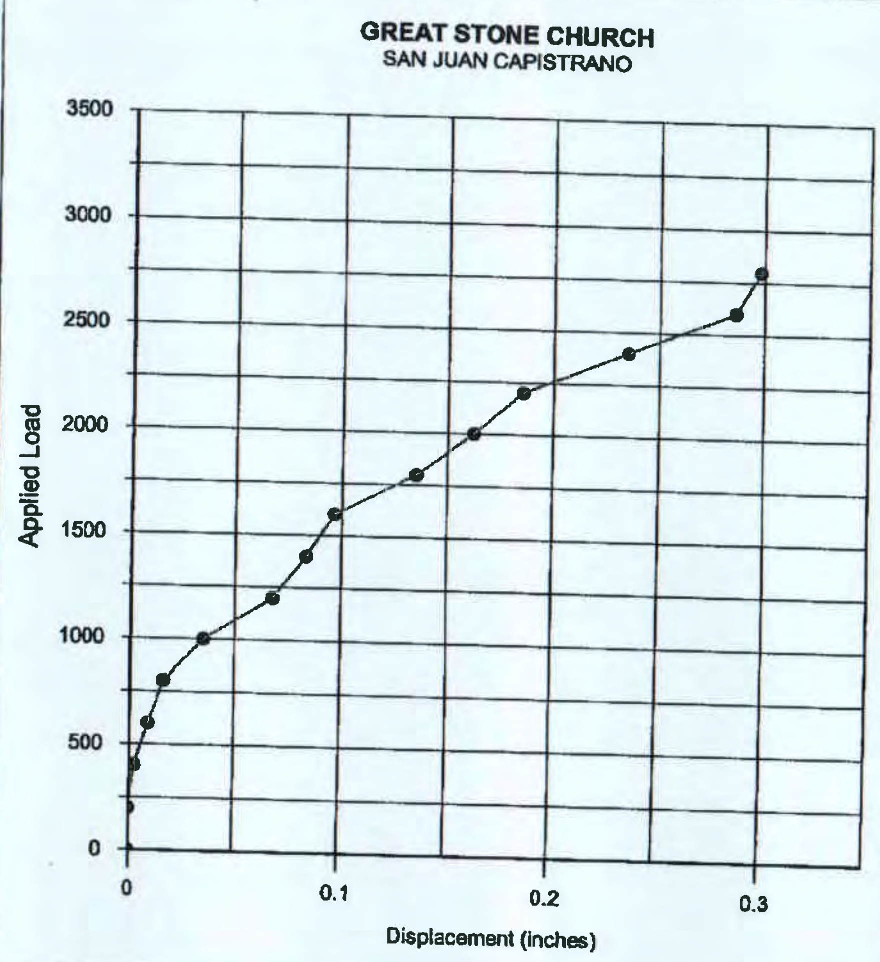


PROJECT: GREAT STONE CHURCH, SAN JUAN CAPISTRANO

TEST DATA FOR CINTEC GROUTED ANCHORS

TEST LOCATION No.	8	No. of Anchors	1
TEST SET UP:	HILTI	Max. Spec. Load	1900 lbs
DATE	02/05/01	Disp. Gauge	MITUTUYO

Applied Load, lbs.	Measured* Displacement	OBSERVATION
0	0.0000	Displacement sensor mounted on track.
200	0.0000	Preload, release load and zero the displacement gauge
400	0.0025	
600	0.0090	
800	0.0160	
1000	0.0350	
1200	0.0680	
1400	0.0835	
1600	0.0965	sleeve anchor beginning to slip from grout sock
1800	0.1350	
2000	0.1620	
2200	0.1855	
2400	0.2350	
2600	0.2860	
2800	0.2980	displacement gauge removed prior to weld failure.
3000		
3200		maximum load, prior to release of load

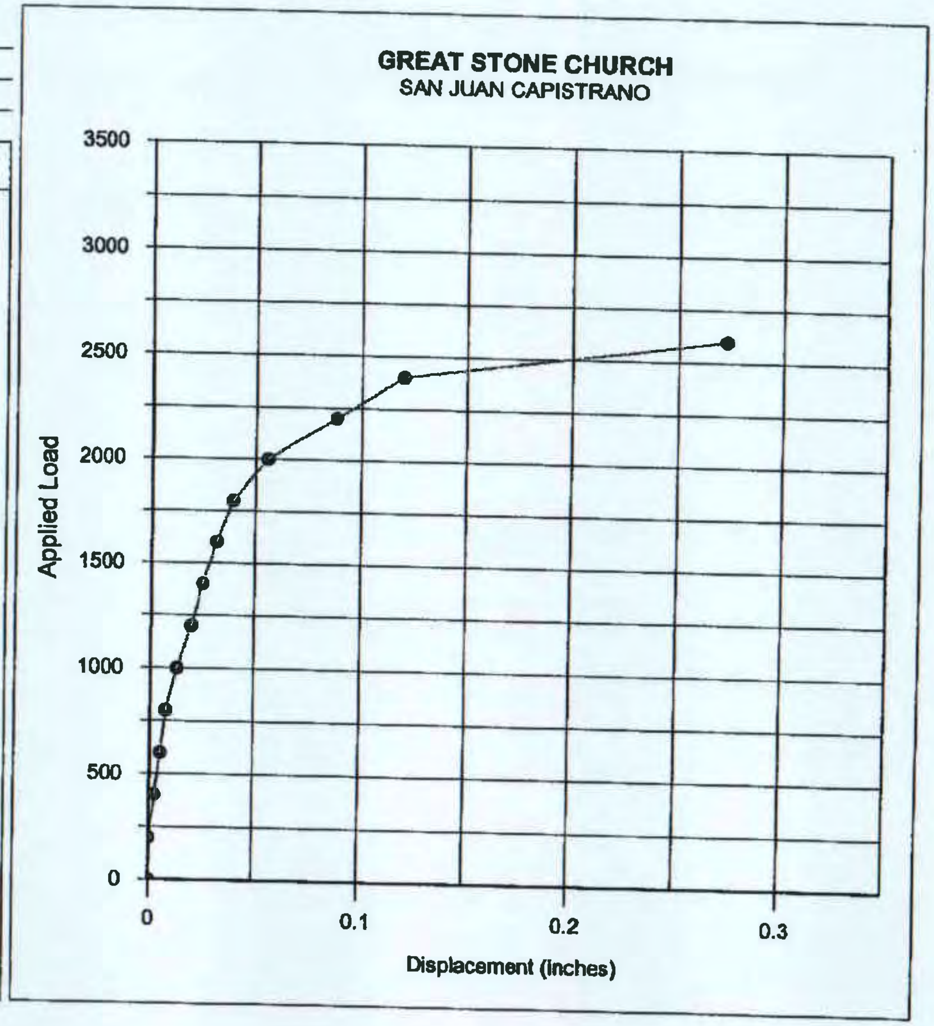


PROJECT: GREAT STONE CHURCH, SAN JUAN CAPISTRANO

TEST DATA FOR CINTEC GROUTED ANCHORS

TEST LOCATION No.	9	No. of Anchors	1
TEST SET UP:	HILTI	Max. Spec. Load	1900 lbs
DATE	02/05/01	Disp. Gauge	MITUTUYO

Applied Load, lbs.	Measured* Displacement	OBSERVATION
0	0.0000	Displacement sensor mounted on track.
200	0.0000	Preload, release load and zero the displacement gauge
400	0.0030	
600	0.0055	
800	0.0080	
1000	0.0130	
1200	0.0195	
1400	0.0250	
1600	0.0315	
1800	0.0390	
2000	0.0555	
2200	0.0880	
2400	0.1200	sleeve anchor beginning to slip from grout sock
2600	0.2730	
2800		displacement gauge removed prior to weld failure.
3000		
3200		maximum load, prior to release of load

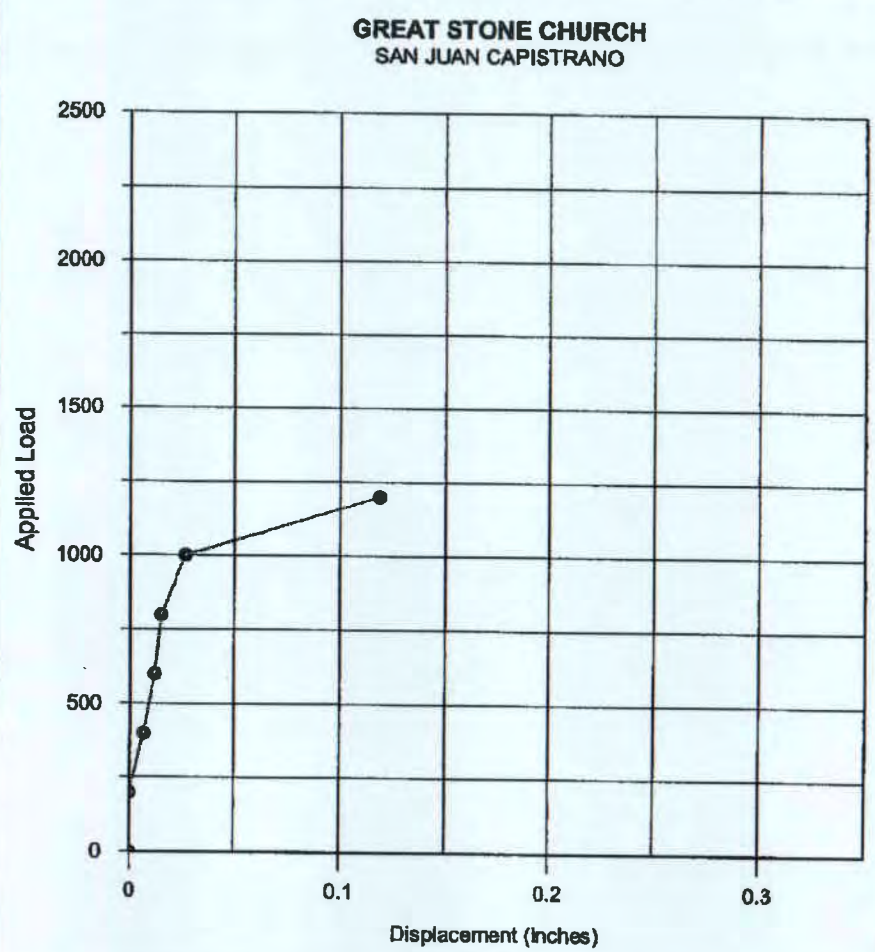


PROJECT: GREAT STONE CHURCH, SAN JUAN CAPISTRANO

TEST DATA FOR CINTEC GROUTED ANCHORS

TEST LOCATION No.	10	No. of Anchors	1
TEST SET UP:	HILTI	Max. Spec. Load	1900 lbs
DATE	02/05/01	Disp. Gauge	MITUTUYO

Applied Load, lbs.	Measured* Displacement	OBSERVATION
0	0.0000	Displacement sensor mounted on track.
200	0.0000	Preload, release load and zero the displacement gauge
400	0.0070	
600	0.0120	
800	0.0150	
1000	0.0265	
1200	0.1190	sleeve anchor beginning to slip from grout sock
1400		displacement gauge removed prior to weld failure.
1600		
1800		
2000		
2200		maximum load, prior to release of load

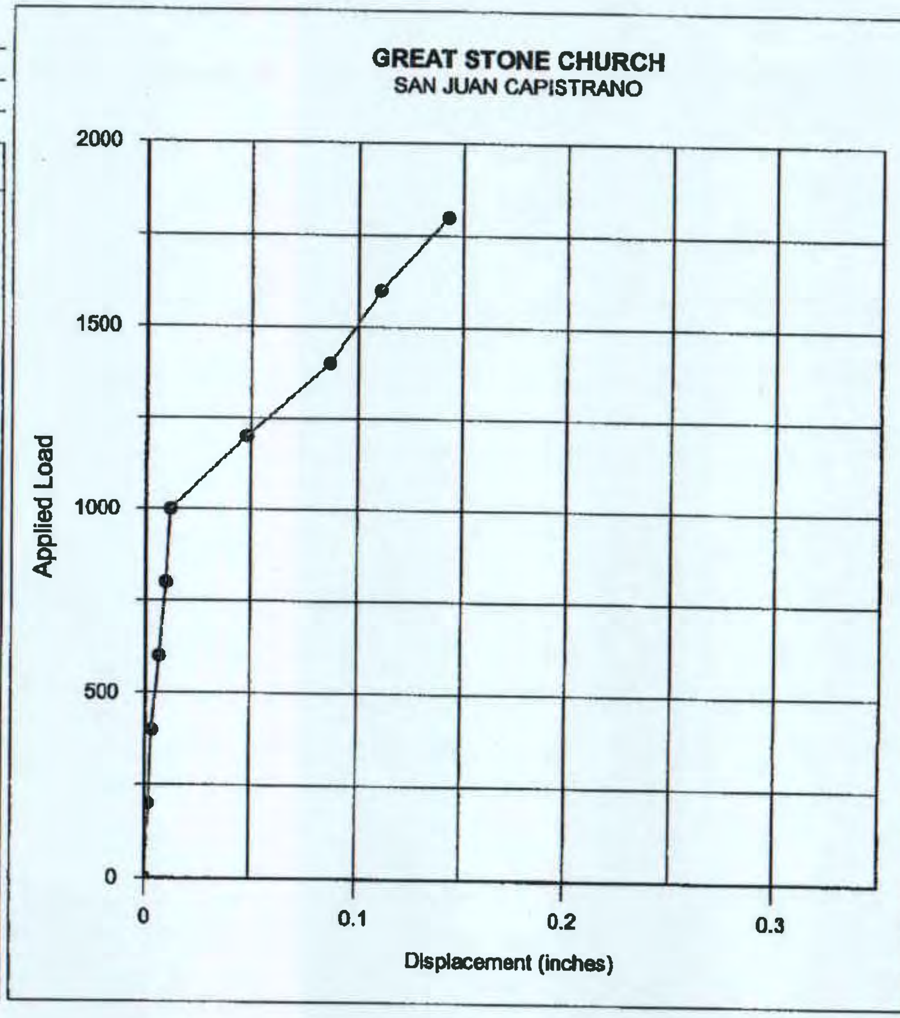


PROJECT: GREAT STONE CHURCH, SAN JUAN CAPISTRANO

TEST DATA FOR CINTEC GROUTED ANCHORS

TEST LOCATION No.	11	No. of Anchors	1
TEST SET UP:	HILTI	Max. Spec. Load	1900 lbs
DATE	02/05/01	Disp. Gauge	MITUTUYO

Applied Load, lbs.	Measured* Displacement	OBSERVATION
0	0.0000	Displacement sensor mounted on track.
200	0.0020	Preload, release load and zero the displacement gauge
400	0.0035	
600	0.0070	
800	0.0100	
1000	0.0120	
1200	0.0480	
1400	0.0865	sleeve anchor beginning to slip from grout sock
1600	0.1105	
1800	0.1430	maximum load, prior to release of load

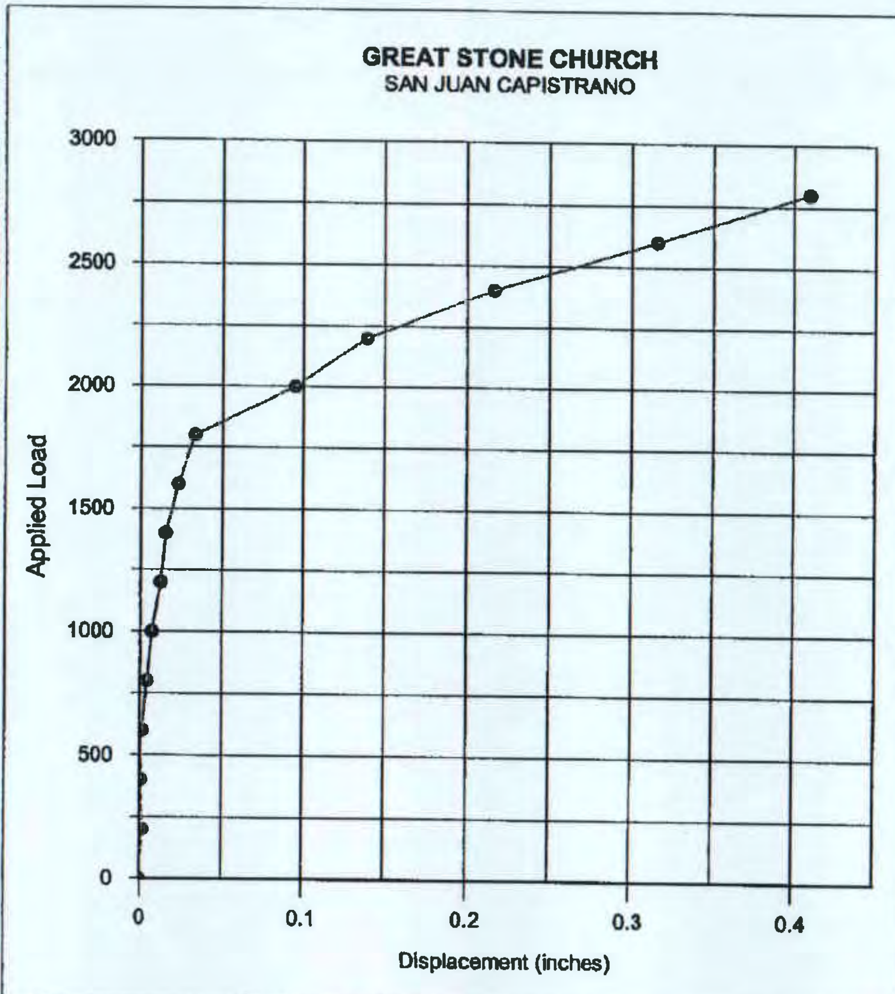


PROJECT: GREAT STONE CHURCH, SAN JUAN CAPISTRANO

TEST DATA FOR CINTEC GROUTED ANCHORS

TEST LOCATION No.	12	No. of Anchors	1
TEST SET UP:	HILTI	Max. Spec. Load	1900 lbs
DATE	02/05/01	Disp. Gauge	MITUTUYO

Applied Load, lbs.	Measured* Displacement	OBSERVATION
0	0.0000	Displacement sensor mounted on track.
200	0.0020	Preload, release load and zero the displacement gauge
400	0.0010	
600	0.0015	
800	0.0045	
1000	0.0075	
1200	0.0125	
1400	0.0155	
1600	0.0230	
1800	0.0335	
2000	0.0950	sleeve anchor beginning to slip from grout sock
2200	0.1390	
2400	0.2160	
2600	0.3155	
2800	0.4095	maximum load, prior to release of load

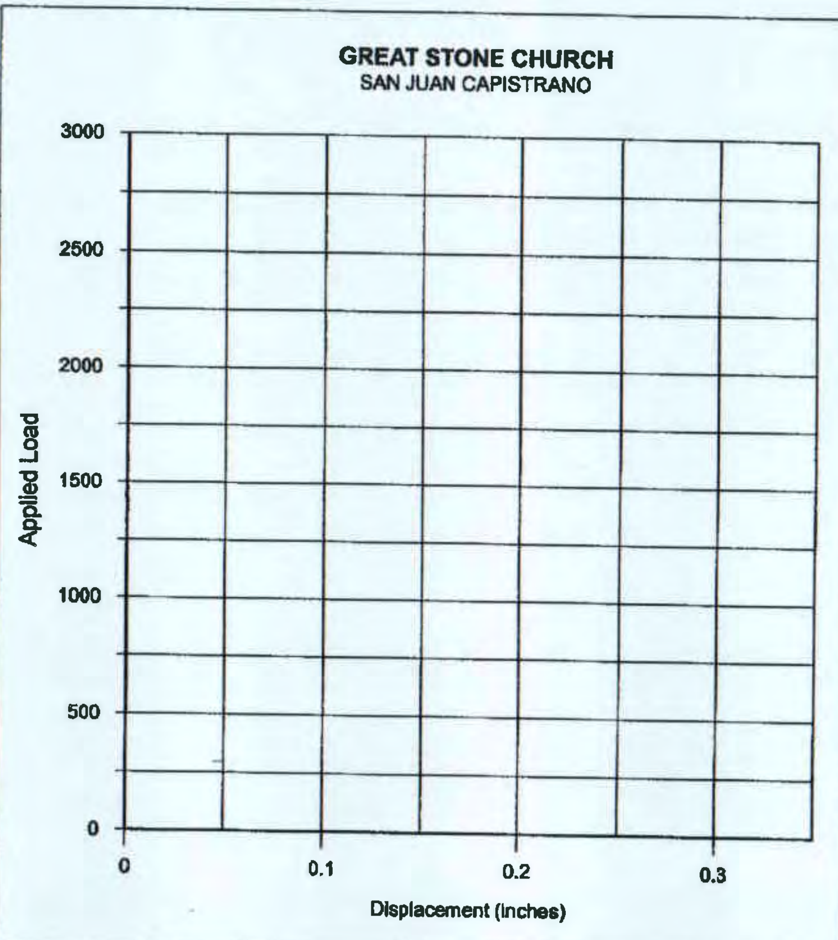


PROJECT: GREAT STONE CHURCH, SAN JUAN CAPISTRANO

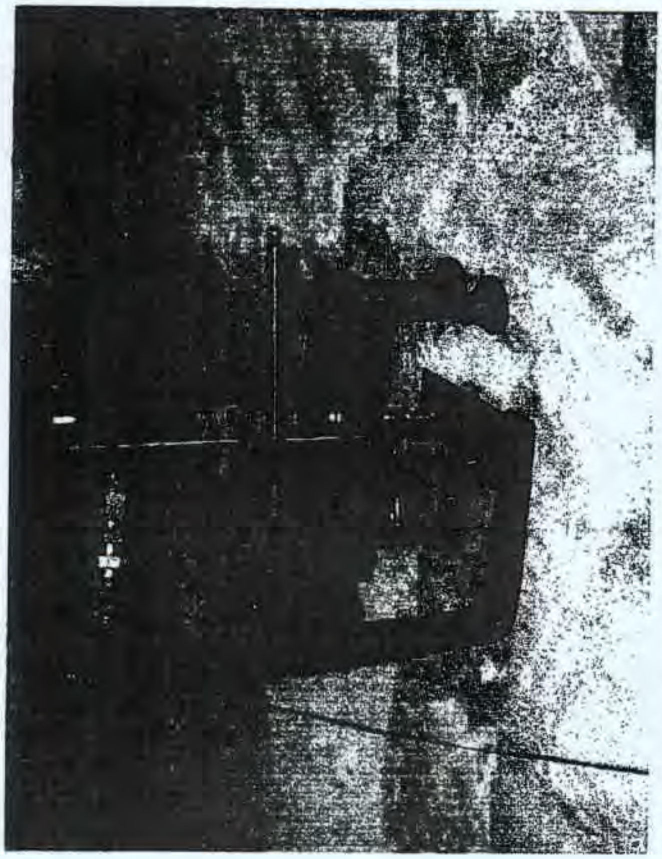
TEST DATA FOR CINTEC GROUTED ANCHORS

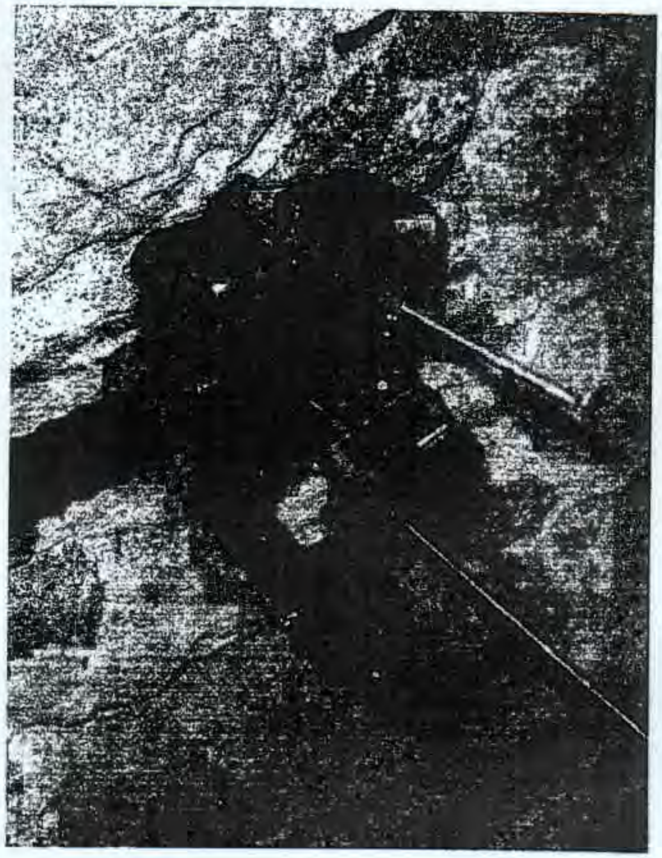
TEST LOCATION No.	14	No. of Anchors	1
TEST SET UP:	HILTI	Max. Spec. Loac	1900 lbs
DATE	02/05/01	Disp. Gauge	MITUTUYO

Applied Load, lbs.	Measured* Displacement	OBSERVATION
0		
200		Preload
400		
600		
800		
1000		
1200		
1400		
1600		
1800		sleeve anchors appears to slip at this load.
2000		
2200		crumbling of mortar at this load
2400		
2600		
2800		
3000		maximum load, prior to release of load

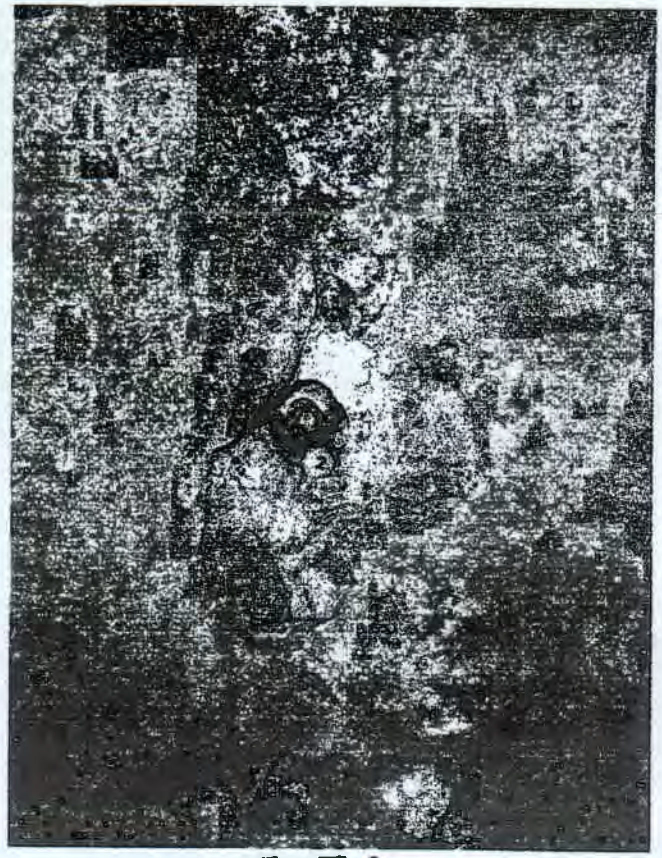


Note:
1. No displacement gauge was set up at this test location. Mortar around the test anchor cannot hold the magnetic displacement gauge bracket.
2. Visual observation only, first movement of the sleeve anchor was observed at 1800 lbs., when mortar around the grout sock begins to crack and crumble to pieces.





TEST 1

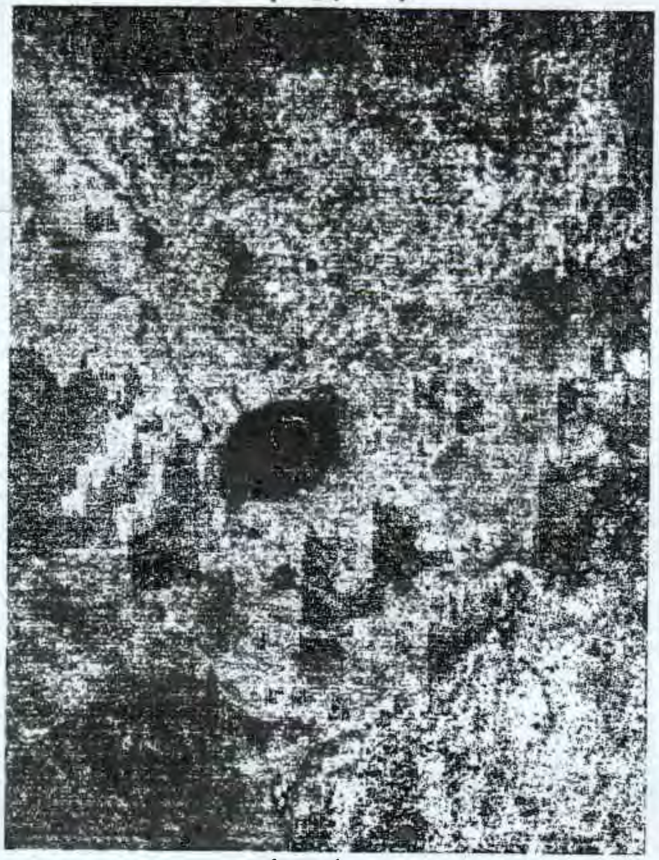


TEST 2

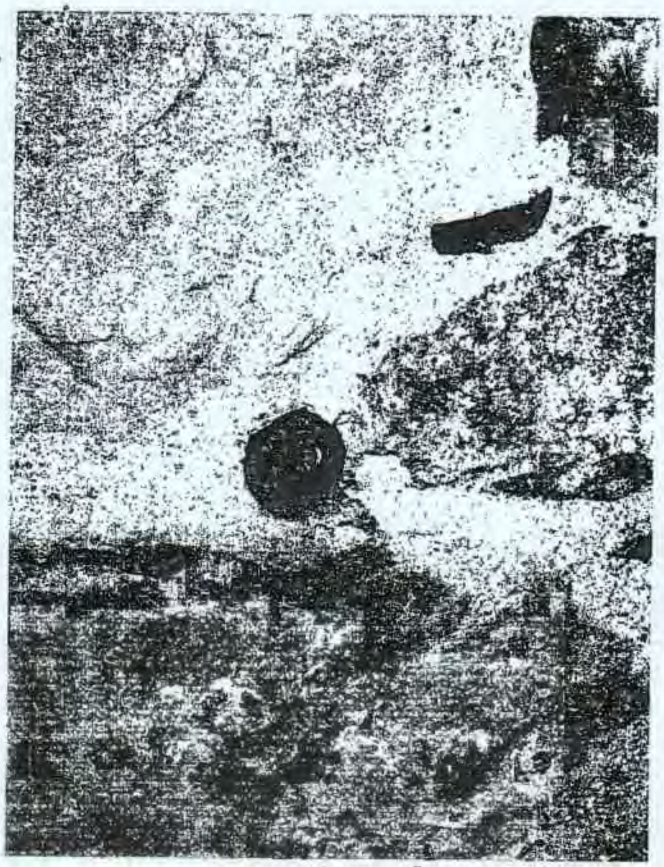
TEST 3

TEST 4

TEST 5



TEST 6



TEST 7

TEST 8

TEST 9

TEST 10

TEST 11

TEST 12

TEST 14

the ROSELUND
ENGINEERING COMPANY
626-573-2441

Client: Mission San Juan Capistrano
Project: Stabilization of the Stone Walls
of the Great Stone Church
Inner and Outer Stone Wythe Bonding by Pinning

Sheet 1 of 2
4/11/01by Nels
Job 00-107

Sequence and Procedure - Test Program of Cintec Grouted All-Thread Anchors

A. Objective

The objective of this Test Program is to observe, evaluate and tension-test Cintec Grouted Anchors in the walls of the Old Stone Church in order to establish the suitability of the Anchors for bonding of the inner and outer wythes of the stone walls to prevent internal separation in the walls. Internal separations may occur due to:

1. Internal shear stresses as the wall rocks on the base of a wythe, its edge acting as a fulcrum so that the weight of the wall is supported by internal vertical shear stresses.
2. Seismic horizontal acceleration of an inner or outer wythe normal to the plane of the wall.

The Anchors will be judged suitable provided

1. They can be demonstrated to be reliably installed,
2. Anchors with a 12 " embedment in stone, in mortar, or in a combination of stone and mortar have tension Anchor capacity with a design strength of 1,900 pounds as determined in Section H., below.
3. No unanticipated disadvantages are observed during the Test Program.

B. Materials

1. Grout: Presstec Cementitious Injection Grout supplied by Cintec in 25kg bags.
2. Stainless steel pins: ½" diameter 316 stainless steel with fabric grout-confinement sock, by Cintec.
3. Water: Clean, potable water from domestic supply.

C. Equipment

1. Drill: 1¼" diameter dry-diamond core-drill bit - no impact or vibratory force shall be used on bit.
2. Grouting Equipment: Comply with Equipment Required for the Installation of the Cintec Anchoring System, a Cintec document.
3. Test equipment:

   Jack: center-pull hydraulic jack with hand-operated hydraulic pump, and digital-reading pressure gauge. The jack/pump/gauge apparatus shall have been calibrated within the preceding 180 days and shall be provided with a chart or other means to allow on-site determination of test loads during the test procedure (submit calibration record).

   Bridge: tripod bridge that applies the jack reactions to the wall at 3 spots, a minimum of 12" from the center of the Anchor.

   Strain gauge: dial or digital-reading strain gage capable of registering increments of 0.001 inch.

   Adapter: coupling nut or other device to connect Anchors to the center-pull jack.

D. Anchor Locations for Tension Test

Tension-test Anchors. Layout locations of 12" depth Anchor installations at convenient locations about 24" to 30" above ground in a wall of the Old Stone Church. Six Anchors shall be installed in holes that enter the wall through a mortar joint; six shall be installed in holes that enter the wall in a stone. The anchors were installed on March 6, 2001.

E. Testing

1. Install a strain gauge to measure the deflection of the Anchor. The strain gage shall be positioned to measure directly the displacement on the anchor along its centerline axis.

2. Install the center-pull jack and bridge to apply a direct tension load to the Anchor and that is

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REPORT OF TEST

30th Street Architect

2821 Newport Beach
Newport Beach, CA 92660
Attn: Accounts Payable

TENSION TESTS FOR CINTEC WALL ANCHORS "STABILIZATION OF THE STONE"

Job Name: Mission San Juan Capistrano

REPORT DATE: May 2, 2001

TWINING LABORATORIES OF SOUTHERN CALIFORNIA, INC.

TEST REPORT NO. 01-9025

Mike Fattal

MIKE FATTAL
DIVISION MANAGER
SPECIAL PRODUCT TESTING DIVISION

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MAY 2, 2001

PROJECT 98-9025

Test Results

Anchor # 3 (stone)

Load (lbs)	Deflection (in)	Mode of Failure
0	0	No sign of failure
200	0	No sign of failure
400	0	No sign of failure
600	0	No sign of failure
800	0	No sign of failure
1,000	0	No sign of failure
1,200	0	No sign of failure
1,400	0	No sign of failure
1,600	0	No sign of failure
1,800	0	No sign of failure
2,000	0	No sign of failure
2,200	0	No sign of failure
2,400	0	No sign of failure
2,600	0	No sign of failure
2,800	0	No sign of failure
3,000	0	No sign of failure
3,000 (30 seconds)	0.0036	No sign of failure

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MAY 2, 2001

PROJECT 98-9025

Test Results

Anchor # 4 (stone)

Load (lbs)	Deflection (in)	Mode of Failure
0	0	No sign of failure
200	0	No sign of failure
400	0	No sign of failure
600	0	No sign of failure
800	0	No sign of failure
1,000	0	No sign of failure
1,200	0	No sign of failure
1,400	0	No sign of failure
1,600	0	No sign of failure
1,800	0.0035	No sign of failure
2,000	0.0035	No sign of failure
2,200	0.0070	No sign of failure
2,400	0.0070	No sign of failure
2,600	0.0070	No sign of failure
2,800	0.0106	No sign of failure
3,000	0.0106	No sign of failure
3,000 (30 seconds)	0.0141	No sign of failure

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MAY 2, 2001

PROJECT 98-9025

Test Results

Anchor # 5 (Mortar)

Load (lbs)	Deflection (in)	Mode of Failure
0	0	No sign of failure
200	0	No sign of failure
400	0	No sign of failure
600	0	No sign of failure
800	0	No sign of failure
1,000	0	No sign of failure
1,200	0	No sign of failure
1,400	0	No sign of failure
1,600	0	No sign of failure
1,800	0	No sign of failure
2,000	0	No sign of failure
2,200	0	No sign of failure
2,400	0.0036	No sign of failure
2,600	0.0036	No sign of failure
2,800	0.00071	No sign of failure
3,000	0.0071	No sign of failure
3,000 (30 seconds)	0.0106	No sign of failure

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PROJECT 98-9025

Test Results

Anchor # 7 (Mortar)

Load (lbs)	Deflection (in)	Mode of Failure
0	0	No sign of failure
200	0	No sign of failure
400	0	No sign of failure
600	0	No sign of failure
800	0	No sign of failure
1,000	0	No sign of failure
1,200	0	No sign of failure
1,400	0	No sign of failure
1,600	0	No sign of failure
1,800	0.0035	No sign of failure
2,000	0.0035	No sign of failure
2,200	0.0035	No sign of failure
2,400	0.0035	No sign of failure
2,600	0.0070	No sign of failure
2,800	0.0070	No sign of failure
3,000	0.0070	No sign of failure
3,000 (30 seconds)	0.0105	No sign of failure

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Test Results

Anchor # 8 (stone)

Load (lbs)	Deflection (in)	Mode of Failure
0	0	No sign of failure
200	0	No sign of failure
400	0	No sign of failure
600	0	No sign of failure
800	0	No sign of failure
1,000	0	No sign of failure
1,200	0	No sign of failure
1,400	0	No sign of failure
1,600	0	No sign of failure
1,800	0.0035	No sign of failure
2,000	0.0035	No sign of failure
2,200	0.0035	No sign of failure
2,400	0.0035	No sign of failure
2,600	0.0070	No sign of failure
2,800	0.0070	No sign of failure
3,000	0.0070	No sign of failure
3,000 (30 seconds)	0.0105	No sign of failure

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Test Results

Anchor # 9 (Mortar)

Load (lbs)	Deflection (in)	Mode of Failure
0	0	No sign of failure
200	0.0071	No sign of failure
400	0.0106	No sign of failure
600	0.0106	No sign of failure
800	0.0106	No sign of failure
1,000	0.0106	No sign of failure
1,200	0.0142	No sign of failure
1,400	0.0142	No sign of failure
1,600	0.0142	No sign of failure
1,800	0.0177	No sign of failure
2,000	0.0177	No sign of failure
2,200	0.0213	No sign of failure
2,400	0.0213	No sign of failure
2,600	0.0248	No sign of failure
2,800	0.0248	No sign of failure
3,000	0.0248	No sign of failure
3,000 (30 seconds)	0.0248	No sign of failure

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Test Results

Anchor # 10 (stone)

Load (lbs)	Deflection (in)	Mode of Failure
0	0	No sign of failure
200	0	No sign of failure
400	0	No sign of failure
600	0	No sign of failure
800	0	No sign of failure
1,000	0.0071	No sign of failure
1,200	0.0071	No sign of failure
1,400	0.0071	No sign of failure
1,600	0.0071	No sign of failure
1,800	0.0071	No sign of failure
2,000	0.0071	No sign of failure
2,200	0.0071	No sign of failure
2,400	0.0071	No sign of failure
2,600	0.0071	No sign of failure
2,800	0.0071	No sign of failure
3,000	0.0071	No sign of failure
3,000 (30 seconds)	0.0106	No sign of failure

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MAY 2, 2001

PROJECT 98-9025

Test Results

Anchor # 11 (stone)

Load (lbs)	Deflection (in)	Mode of Failure
0	0	No sign of failure
200	0	No sign of failure
400	0	No sign of failure
600	0	No sign of failure
800	0	No sign of failure
1,000	0	No sign of failure
1,200	0	No sign of failure
1,400	0	No sign of failure
1,600	0	No sign of failure
1,800	0	No sign of failure
2,000	0	No sign of failure
2,200	0	No sign of failure
2,400	0	No sign of failure
2,600	0	No sign of failure
2,800	0	No sign of failure
3,000	0	No sign of failure
3,000 (30 seconds)	0.0035	No sign of failure

TESTING OF CINTEC WALL ANCHORS

[To resist a seismic event]

for

BRITISH NUCLEAR FUELS

Magnox Generation

Wylfa Power Station

By

Celtest Ltd

and

Cintec International

November – December 2001

Wylfa power station, Anglesey, Wales.

WS/Atkins

RSO

TAYWOOD ■ ENGINEERING

NNC

MOTHERWELL BRIDGE

BNFL
Magnox Generation

Project – Wylfa LTSR

Document Type – Report

Document Number –14392/30/REP/1003

Trial installation and testing of Cintec wall anchors.

Revision	Date	Originator	Checked by	Reviewed for the Consortium
A	10/1/02	D Castree	T. Kirkland	[signature]

Approved for LTSR by:	Name	Signature	Date
	Ron A Alasin	Ronald A Alasin	17.1.02

DOCUMENT REVISION RECORD

Revision	Date	Reason for Revision	Approval Reference
0	22/12/01	Issue for approval purposes	
A	14/1/02	Incorporation of internal LTSR team comments	

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1 INTRODUCTION

As part of the LTSR project, over 400 stretcher-bond cavity brick walls at Wylfa are to be modified in order for them to meet the design requirements for strengthening. Currently the designs from W S Atkins and Babtie require the strengthened walls' leaves to behave in a composite manner under loading from either seismic, hot gas or steam release events. The design from Babtie specifies the use of Cintec steel anchors embedded in grout in an array over the wall panel to tie the two leaves together. Babtie have produced a substantiation report on the use of Cintec anchors based on these test results (document reference 1). It applies only to walls to be strengthened to resist a seismic event. The W S Atkins' design incorporating Cintec is currently being developed.

2 PURPOSE

It is clear from the nature and history of the use of Cintec anchors that they are specifically designed to be used as retro-fitted anchors in brickwork. The purpose of the in-situ testing at Wylfa is therefore simply confirmatory not defining.

The objectives of the installation and the testing were:

- To check the ease of installation
- To check the speed of the installation procedures
- To assess the grout-to-brickwork bond strength (if critical)
- To assess the grout-to-bar bond strength (if critical)
- To assess the brickwork's crushing strength (if critical)
- To assess the grout's crushing strength (if critical)
- To measure bar deflections under shear load (vertical and horizontal)
- To measure bar displacements under pull-out load
- To assess the effect of frog up or down orientation on Cintec anchors
- To note the modes of failure

3. THE TRIALS

3.1 The choice of wall:

A North East loading bay wall was chosen for the trial as it had no safety significance, provided easy access to both sides and was a representative example of the majority of the reactor building's stretcher bond cavity brickwork which will require strengthening.

3.2 Wall description:

The trial wall extends 5.1 metres vertically between two concrete beams/slabs and 3.7 metres horizontally between two concrete columns.

Each leaf is built in stretcher bond from clay brickwork. Evidence (i.e. size, appearance and testing (ref 2)) suggests that generally London clay brickwork has been used for wall construction at Wylfa although evidence of use of Butterleys brick was revealed in earlier cores taken from the trial wall. The wall has been painted. Details of the brickwork were assumed to be similar to those tested and reported in the design philosophy document (ref 1).

3.3 Anchor choice:

Cintec anchor bars of the type and size selected by Babtie were adopted for the tests with the exception of the anchors' length. In the Babtie design for the seismically loaded walls the anchor will be grouted fully in the near leaf and partially in the far leaf. For the tests, in order to more accurately assess the 'real' anchor's capacities, the test anchor chosen was only long enough to be embedded 80mm into one leaf. A typical hole/sock/bar configuration is shown in Appendix A sheet 1. The anchor pattern on the wall is shown in Appendix A sheet 2.

The single leaf approach simplifies the post-test mathematical modelling to calculate characteristic strengths.

3.4 Anchor description:

The test anchors were specially manufactured for the test program by Cintec. They consist of a 10 mm diameter x 125 long steel bar with a plastic centring washer at the end in the wall. A polypropylene 'sock' with relatively open weave is clamped to the washer and extends for 90 mm along the length of the bar. Inside the sock there is a plastic tube extending along the bar to deliver grout to the back of the sock first. The tube is taped to the bar. At the front end the sock is gathered into both the bar and tube and tied off, leaving approximately 25 mm of bar and 50mm of grout tube projecting. The projecting bar is threaded (10 diameter metric thread). After the grout has hardened the projecting grout-filled tube is cut off and discarded.

3.5 Anchor action:

The grout is a mixture of a German equivalent of Ordinary Portland Cement and a volcanic sand (finely milled volcanic trass). These are pre-mixed at the factory. Water is added at site and the mixture is thoroughly stirred with a mechanical agitator. The grout is injected under pressure into the sock, which expands to the limits of its containment or to approximately 40 mm diameter – whichever is the smaller. See photograph of grout-inflated sock in Appendix G. The pressure is held for a few seconds, and grout 'milk' – largely cementitious water - seeps out of the weave of the sock, effectively reducing the percentage of water in the grout still retained by the sock. At this point the retained grout loses its liquid-like properties and becomes a pressurised solid. The sock becomes rigid. When the pressure is disconnected from the grout tube, leakage of grout is negligible.

4 DESCRIPTION OF THE INSTALLATION AND TESTING.

4.1 Documentation:

A method statement reference 14392/30/GMS/1005 rev B, quality plan reference 14392/30/GQP/1005 rev B and risk assessment reference 14392/30/GMS/1005 rev B were prepared for the in-situ installation and testing and they were approved before work on the installation began.

4.2 Cintec anchors' installation:

On 25 and 26/11/01 twenty-five 10mm diameter Cintec anchors were installed – generally at 460mm centres (see Appendix A Sheet 2) - and grouted into 30 mm diameter holes 85 mm deep into one leaf of the brickwork. The holes were formed by dry, rotary-percussive drilling to a depth of 70 mm, then rotary drilling to the final depth of 85 mm. The holes were then purged of debris by vacuum cleaning. All drilling took place on 26/11/01.

The bars were made from high yield reinforcement (fy = 460 N/sq mm) and the projecting ends were threaded. All anchor installation and grouting took place on 27/11/01. The holes' bores were wetted with an unused anchor's wet sock. Prestec cementitious grout with a projected mean compressive stress of 51.5 N/sq mm at 28 days was used for all anchors. Two batches of grout (designated #1 and #2 – see Appendix B Sheets 1 and 2) were used to fill the anchors' socks and to make twelve 100mm test cubes (see 4.3 below). The anchors were inserted into the holes and the grout injected into the sock using compressed air at 3.0~3.5 bar as the propellant. Each sock was 90 mm long – 80 mm of which was embedded in the hole. The installation log is attached as Appendix F

The total drilling time for all 25 holes - including set-up time - was approximately four hours (x two men); installation and grouting took two hours (x two men). These figures give approximately 15 minutes x two men per anchor.

The drilling revealed a random laying pattern of frog-up and frog-down bricks (see Appendix A sheet 2).

4.3 Grout test cubes:

12 grout test cubes were made using the same grout and delivery method as for the anchors. Celtest Limited carried out the cube storage and testing. Celtest's facilities are UKAS approved. The cubes were stored under moist hessian in out-door conditions in Bangor. Pairs of cubes were laboratory tested at 3 days, 7 days, 10 days, and 14 days, using Celtest's 2000 kN in-house compressive testing machine (last calibrated on 8/2/01); a final pair were tested at 27 days (24/12/01). The test results are attached as Appendix B.

4.4 Anchor load tests:

On the thirteenth and fourteenth day after grouting - i.e. 10 & 11/12/01, ten anchors were tested in vertical shear, ten were tested in horizontal shear and five were tested in pull-out. All anchor load tests were carried out in-situ. Anchor V2 which had already been tested in vertical shear was also tested in pull-out (see Certificate numbers 0190 and 0197 in Appendix C). The total number of tests was therefore twenty-six.

4.5 Test equipment:

The shear loads were applied to the subject anchor using a hydraulic jack which delivered its force via a length of 20 diameter steel studding to a steel channel with a hole in its web. The hole fitted over the projecting part of the subject anchor, and was secured by a nut and washer. An adjacent anchor with a loosely fitted nut was used in a longitudinal slot-hole in the web of the channel to act as a guide – (see Appendix A sheet 3). The web of the channel was 10 mm thick; the back of the web was in contact with the outer end of the grouted sock, which projected 10 mm from the wall face. The centre of the shear load was therefore delivered to the anchor at 15mm from the face of the wall, thus representing a half of a 30 mm wide cavity. The purpose of this was to emulate the behaviour of anchor in the far leaf. The jack reacted against the ground floor slab as the fixed point of resistance for vertical shear tests and against the projecting main frame column for horizontal shear tests. Pull-out loads were applied to a nut on the threaded part of the anchor using a screw device. This was mounted on a four-legged steel bridging piece which used the wall's face as the fixed point of resistance.

4.6 Load measurement:

Measurement of loads in shear tests was intended to be by an independent load cell – fitted in series with the jack - reading in 10N (.01 kN) increments, with additional check readings from the hydraulic pressure gauge mounted integrally on the jack. In fact, during the third shear test it was noted that the jack's analog readings and the load cell's digital readings were becoming inconsistent. The divergence is recorded below:

Jack's integral analog gauge kN:	Load cell's digital display kN:
3 kN	2 kN
8 kN	6 kN
15 kN	12 kN
17 kN	14 kN

Where differences in reading were noted, the lower reading was used. Both instruments were checked in the Station's instrument workshop and the load cell's readings were found to be 'drifting' i.e. giving changing readings under a constant load. However the jack's pressure gauge reading under a known test load of 20 kN was found to be accurate and stable; it was therefore decided to use the jack's integral analog gauge for subsequent shear load readings. It is considered that the load cell readings which were used only for the first three of twenty-six tests, are representative as readings were made generally before the drift occurred. Where there was evidence of discrepancy in the third test – the lower value of the two was the one recorded and used. The jack's gauge (calibrated in August 2001) gave direct readings to 2 kN, or by assessment under close scrutiny - it was possible to use 1 kN increments. These 1 kN increments each represent a small load 'step' of approximately 5% of the total load at failure, so the accuracy of the results obtained can therefore be considered to be acceptable for the purposes of these tests.

The pull-out equipment was fitted with its own integral hydraulic pressure gauge (calibrated in August 2001) reading in 0.5 kN.

4.7 Displacement measurement:
A dial type strain gauge was used giving readings directly to 0.01 mm. The base of the strain gauge was mounted on a magnetic stand on a steel plate bolted rigidly to an adjacent, unloaded anchor. The end of the probe of the strain gauge bore onto one of the endplates of the jacking channel. Where this was the free, unloaded endplate the strain gauge accurately showed the free movement of the channel (and hence the projecting part of the anchor) relative to the wall. A concern was raised - by some witnesses in the post-testing meeting - that where the gauge's probe was bearing onto the loaded endplate (as it did in some cases) where the jack delivered its force, the readings might be distorted by endplate displacement due to local bending under the jack's loading. The calculations in Appendix E deal with these concerns and show the plate's

maximum deflection under load to be negligible when compared to the total displacement recorded.

5 RESULTS.

5.1 Failure types:

All ties were tested to failure. The failure modes for shear tests were load loss, brickwork crushing or excessive bar displacement accompanied by local grout crushing around the bar at the exposed end of the sock, or combinations of these. In all the shear tests the local deformation of the projecting part of the tie was the most significant part of the total displacement. For pull-out tests the mode of failure was load loss combined with slip displacement between the grouted sock and the brickwork. The wall remained stable overall throughout the tests, and there was no cracking or movement observed around individual bricks. Spalling of the brickwork only occurred in the area local to the sock.

5.2 Certificates:

The results of all the in-situ load tests are given on Cintec Test Certificates numbers 0189 to 0210 and 0240 to 0243 (representing 26 tests on 25 anchors). Measurement of loads and displacements were monitored closely by Magnox, W S Atkins and Babtie and a representative of each organisation signed each test certificate at the test site as each test was completed. The Certificates in Excel format are attached in Appendix C, together with summaries of load displacement charts for shear load tests; the original, signed paper versions of these certificates are held in LTSR files, and are available for inspection on request.

6 REFERENCES

Document ref 1: P Griffies BABTIE: Cintec Anchor test report ref 201995-R-)8 December 2001

Document ref 2: CK0509/R102 Wylfa Power Station – Civil Redesign of Hot Gas And Steam Release Stage Submissions 1, 2A and 6 Design Philosophy.

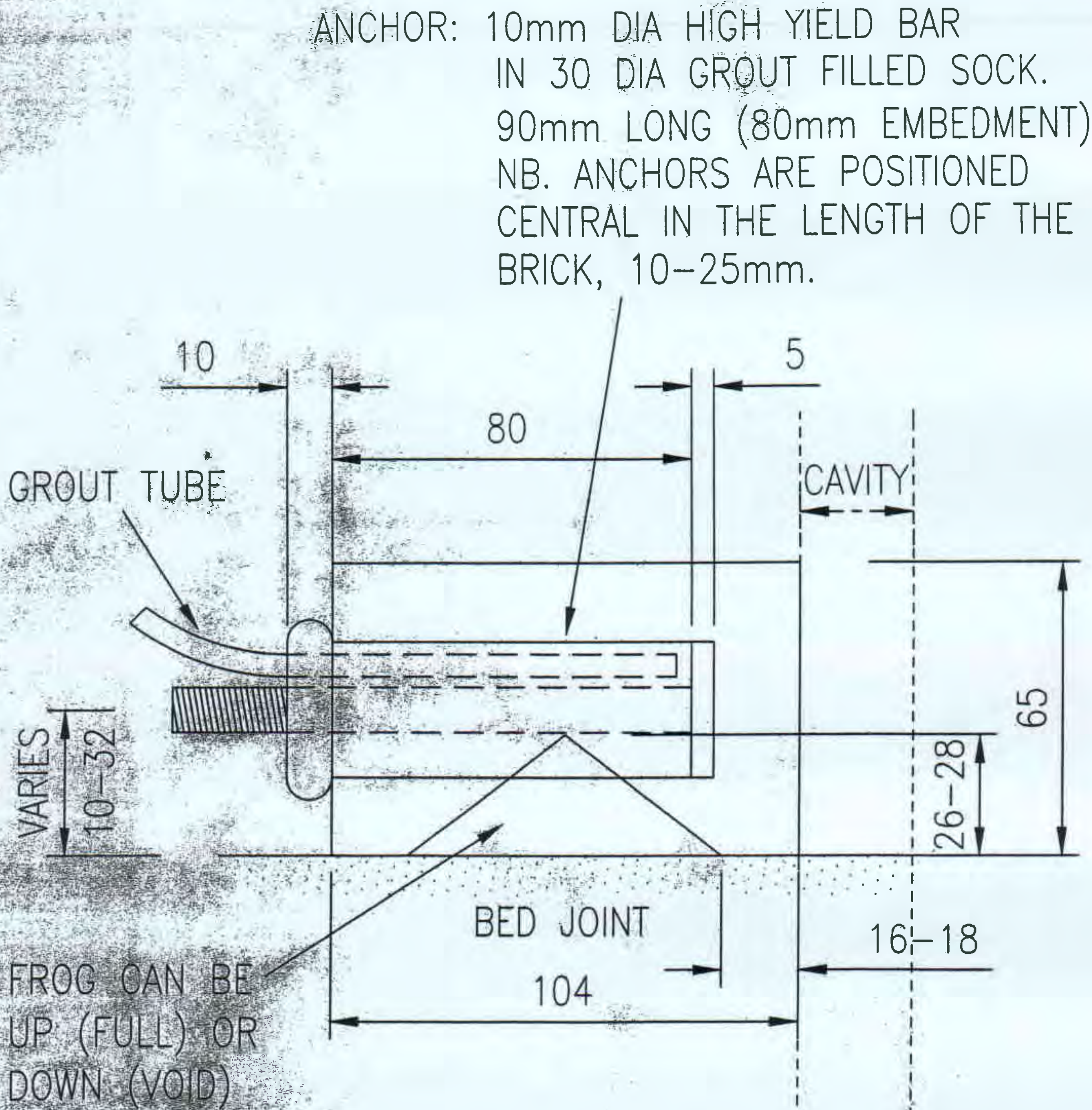
Documents ref 3: LTSR Quality Plan number 14392/30/GQP/1005 rev B and Method Statement number 14392/30/GMS/1005 rev B.

7 CONCLUSIONS

The installation was carried out without major difficulty and the speed of installation was good (see 4.2). The strengths of the various components is assessed in Appendix E. Load/deflection charts are shown in Appendix C. The results indicate that frog orientation (up or down) did not significantly affect the behaviour or the ultimate load of the anchor. The results demonstrate the ability of the Cintec anchor to adapt to non-uniform hole shapes The modes of failure were consistent throughout the tests; these are discussed in item 5.1 and illustrated in Appendix G.

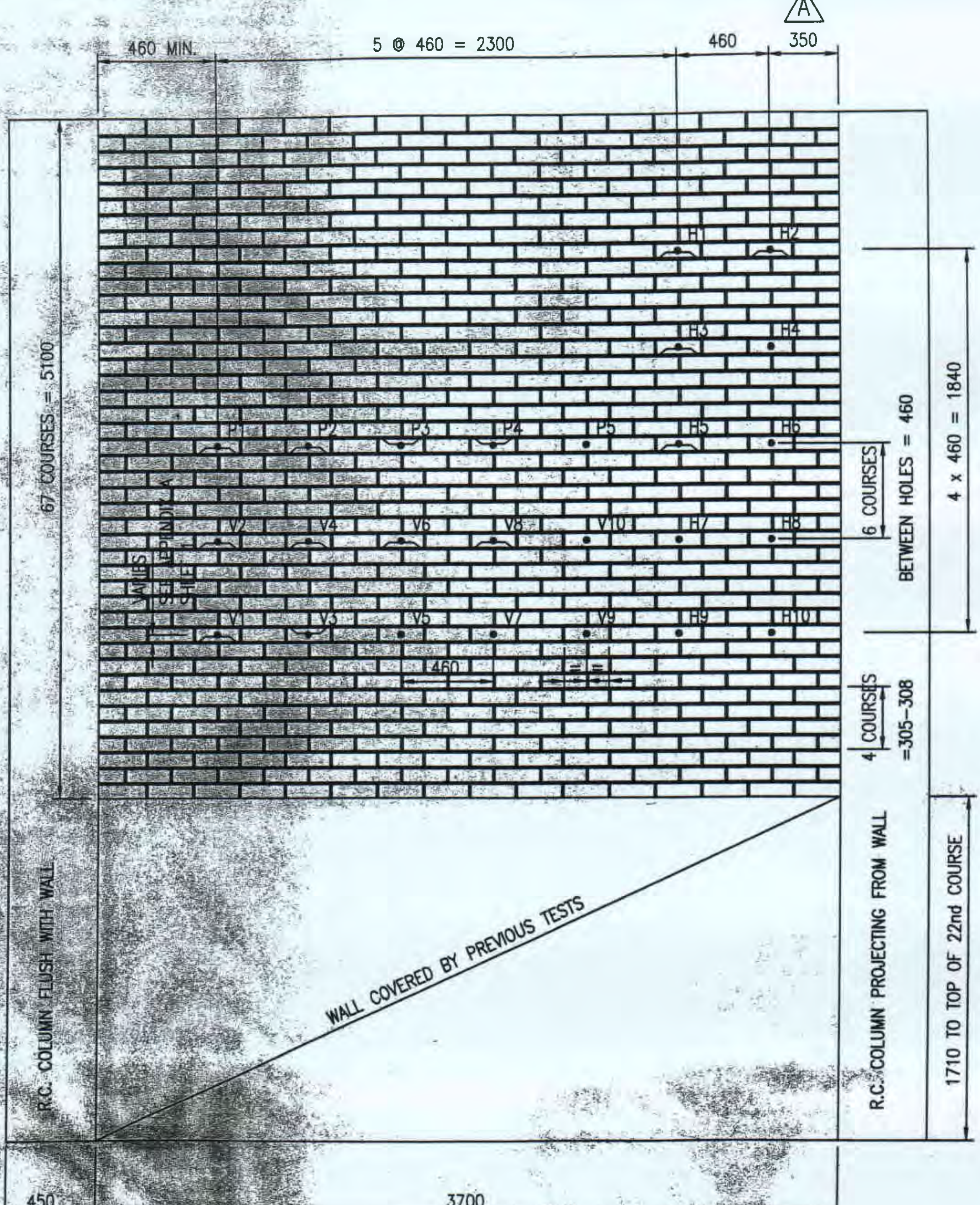
Overall, the installation and testing showed the suitability of Cintec anchors for use in walls of the Wylfa type, and demonstrated the anchors' load-bearing capacities in vertical and horizontal shear and pull-out to be consistent and adequate.

8 APPENDICES follow >>>>>



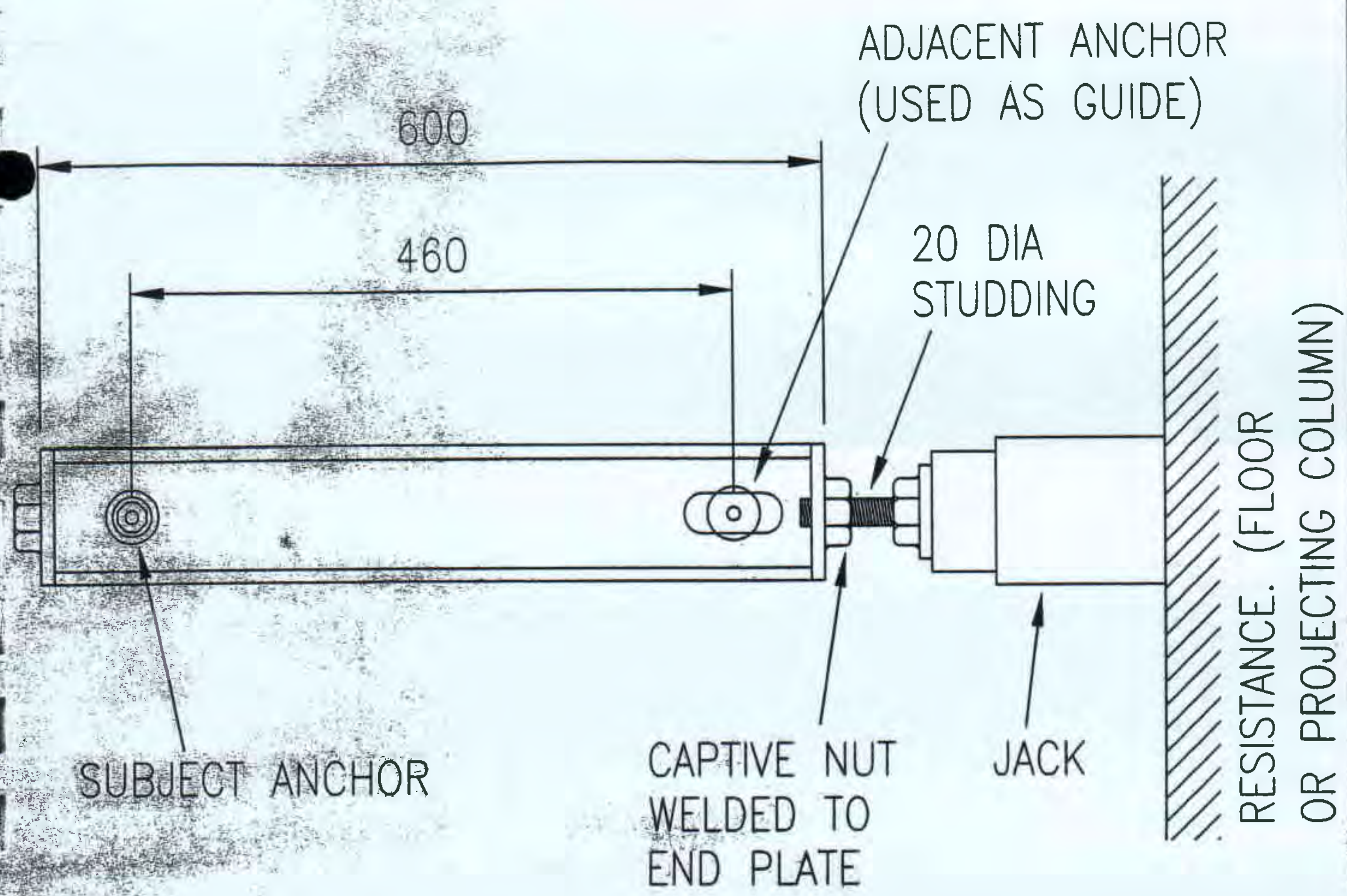
TYPICAL TEST ANCHOR IN BRICKWORK

APPENDIX A
SHEET 1

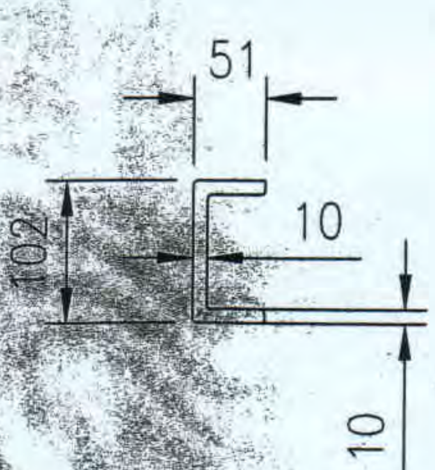


LAYOUT OF ANCHORS IN TRIAL WALL

APPENDIX A
SHEET 2 REV A.



DETAIL OF REVERSIBLE
JACKING CHANNEL



TYPICAL SECTION
THROUGH CHANNEL

APPENDIX A
SHEET 3

B

celtest

independent materials testing; structural investigation
site investigation, diamond core drilling & sawing

UKAS TESTING 0494

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Fax: +44 (0) 1248 351563 • Website: www.celtest.com • e-mail: postmaster@celtest.com

BNFL Magnox Ltd.
Wylfa Power Station
Cemaes Bay
Anglesey LL67 0DH

Date: 27 December 2001
Our Report Ref. No: CTR4943
Test Ref CC16
Your order no:

Compressive Strength of Concrete Cube Report Sheet 1.

Site: **Wylfa Power Station, Anglesey**
Location in works which sample represents: **Unknown**

Certificates of sampling / specimen preparation / curing received: **Yes**
Laboratory Sample Ref: **19665**
Date of receipt of specimens at Laboratory: **29 November 2001**
Preparatory treatment to cubes (e.g. removal of fins): **None**
Method of Density Determination: C for *Calculated* or W for *Water Displacement*

Specimens were moist cured under damp sacking at our Bangor Laboratory prior to storage in water between 18°C and 22°C until the time of test, unless otherwise noted. Concrete cubes in our receipt for 3 days or less prior to testing are moist at the time of test and the reported mass and density are the concrete in the moist condition. Concrete cubes over 3 days in our receipt prior to testing are saturated at the time of test and the reported mass and density are for concrete in the saturated condition.

We certify that the test has been carried out in accordance with BS 1881: Part 116: 1983

Cube Markings Our Ref	Your	Date Cast	Date of Test	Age at Test	Meas. Dim (mm) Lx BxH	Mass (g)	Density (kg/m3)	Method	Failure Load	Comp. Strength (N/mm2)	Mode of Failure	Appearance of Cube on Receipt
3089	1	27/11/01	30/11/01	3	100x100x100	2021	2025	W	339kN	34.0	Normal	Air Cured
3090	2	27/11/01	04/12/01	7	100x100x100	2018	2020	W	469kN	47.0	Normal	Air Cured
3091	3	27/11/01	07/12/01	10	100x100x100	2021	2020	W	500kN	50.0	Normal	Air Cured
3092	4	27/11/01	11/12/01	14	100x100x100	1999	2040	W	527kN	52.5	Normal	Air Cured
3093	5	27/11/01	27/12/01	30	100x100x100	2005	2025	W	535kN	53.5	Normal	Air Cured
3094	6	27/11/01			-	-	-	C	kN	-	Normal	Air Cured

Specified mix:	Grout - Site Mix		
Actual slump:	N/A	Size of load (m^3):	N/A
Location of sampling:	Loading Bay	Ticket no. (if applicable):	N/A
Date and time of sampling:	27/11/01 -	Time and place of making cubes:	at Site
Method of compaction:	Tamping Bar -	Name of person making cubes:	Unknown

Comments
Appearance of cubes on receipt satisfactory unless otherwise stated
(A) Poor compaction (B) Honeycombing (C) Bad dimension (D) Damaged surface (E) Damaged corners

E.R. Goulden, Technical Manager

celtest

independent materials testing; structural investigation
site investigation, diamond core drilling & sawing

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Fax: +44 (0) 1248 351563 • Website: www.celtest.com • e-mail: postmaster@celtest.com

UKAS TESTING
0494

BNFL Magnox Ltd.
Wylfa Power Station
Cemaes Bay
Anglesey LL67 0DH

Date: 27 December 2001
Our Report Ref. No: CTR4944
Test Ref CC16
Your order no:

Compressive Strength of Concrete Cube Report *Sheet 2*

Site: **Wylfa Power Station, Anglesey**
Location in works which sample represents: **Unknown**

Certificates of sampling / specimen preparation / curing received: **Yes**
Laboratory Sample Ref: **19665**
Date of receipt of specimens at Laboratory: **29 November 2001**
Preparatory treatment to cubes (e.g. removal of fins): **None**
Method of Density Determination: C for *Calculated* or W for *Water Displacement*

Specimens were moist cured under damp sacking at our Bangor Laboratory prior to storage in water between 18°C and 22°C until the time of test, unless otherwise noted. Concrete cubes in our receipt for 3 days or less prior to testing are moist at the time of test and the reported mass and density are the concrete in the moist condition. Concrete cubes over 3 days in our receipt prior to testing are saturated at the time of test and the reported mass and density are for concrete in the saturated condition.

We certify that the test has been carried out in accordance with BS 1881: Part 116: 1983

Cube Markings Our Ref	Your	Date Cast	Date of Test	Age at Test	Meas. Dim (mm) Lx BxH	Mass (g)	Density (kg/m3)	Method	Failure Load	Comp. Strength (N/mm2)	Mode of Failure	Appearance of Cube on Receipt
3095	7	27/11/01	30/11/01	3	100x100x100	2012	2015	W	359kN	36.0	Normal	Air Cured
3096	8	27/11/01	04/12/01	7	100x100x100	2011	2015	W	476kN	47.5	Normal	Air Cured
3097	9	27/11/01	07/12/01	10	100x100x100	2018	2020	W	492kN	49.0	Normal	Air Cured
3098	10	27/11/01	11/12/01	14	100x100x100	2011	2015	W	522kN	52.0	Normal	Air Cured
3099	11	27/11/01	27/12/01	30	100x100x100	2038	2020	W	554kN	55.5	Normal	Air Cured
3100	12	27/11/01			-	-	-	C	kN	-	Normal	Air Cured

Specified mix:	Grout - Site Mix		
Actual slump:	N/A	Size of load (m^3):	N/A
Location of sampling:	Loading Bay	Ticket no. (if applicable):	N/A
Date and time of sampling:	27/11/01 -	Time and place of making cubes:	at Site
Method of compaction:	Tamping Bar -	Name of person making cubes:	Unknown

Comments
Appearance of cubes on receipt satisfactory unless otherwise stated
(A) Poor compaction (B) Honeycombing (C) Bad dimension (D) Damaged surface (E) Damaged corners

E.R. Goulden, Technical Manager

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Shear	27.11.01	10.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position V1

Base Material
Brick

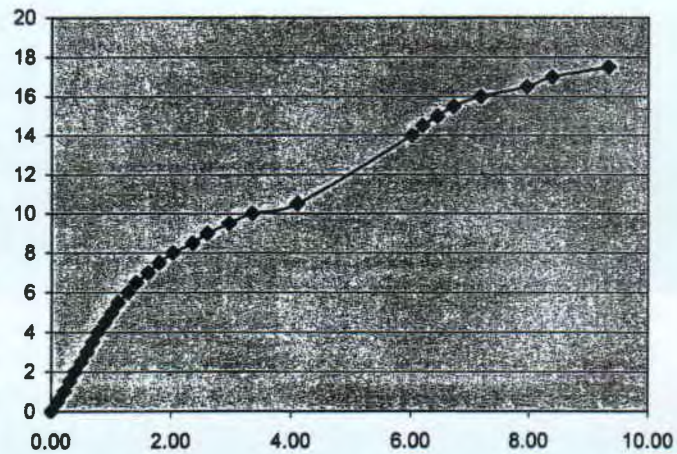
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
0.5	"	no change	0.11	0.12
1	"	no change	0.22	0.23
1.5	"	no change	0.31	0.32
2	"	no change	0.40	0.41
2.5	"	no change	0.50	0.51
3	"	no change	0.60	0.61
3.5	"	no change	0.69	0.70
4	"	no change	0.78	0.79
4.5	"	no change	0.90	0.91
5	"	no change	1.01	1.03
5.5	"	no change	1.12	1.15
6	"	no change	1.29	1.31
6.5	"	no change	1.42	1.45
7	"	no change	1.62	1.64
7.5	"	no change	1.80	1.83
8	"	no change	2.03	2.06
8.5	"	no change	2.36	2.41
9	"	8.5	2.60	2.65
9.5	"	9	2.96	3.01
10	"	9.53	3.35	3.39
10.5	"	–	4.1	–
Guage	mis-read	readings	ommitted	
14	"	13	6.02	6.06
14.5	"	13.8	6.2	6.24
15	"	14.2	6.45	6.49
15.5	"	14.5	6.72	6.78
16	"	14.8	7.18	7.24
16.5	"	15.3	7.96	8.03
17	"	15.5	8.39	8.45
17.5	"	16	9.34	9.37



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W. S Atkins		
ter Griffiths	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

190

Type of Test:	Install Date	Test Date
Shear	27.11.01	10.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position **V2**

Base Material
Brick

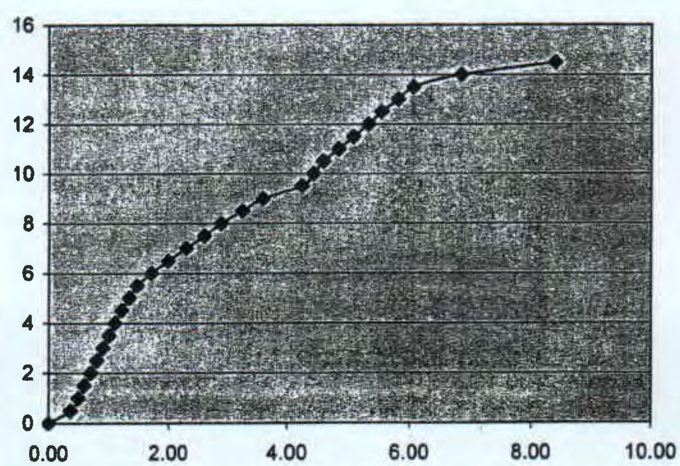
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	0.00
0.5	"	no change	0.37	0.37
1	"	no change	0.50	0.52
1.5	"	no change	0.59	0.61
2	"	no change	0.70	0.71
2.5	"	no change	0.80	0.81
3	"	no change	0.90	0.92
3.5	"	no change	1.00	1.02
4	"	no change	1.10	1.13
4.5	"	no change	1.21	1.23
5	"	4.9	1.34	1.36
5.5	"	5.3	1.47	1.51
6	"	5.7	1.71	1.00
6.5	"	6.1	1.99	77.00
7	"	6.5	2.30	2.03
7.5	"	6.8	2.61	2.36
8	"	7.4	2.89	2.68
8.5	"	7.8	3.25	3.02
9	"	8.2	3.60	3.37
9.5	"	9	4.24	3.66
10	"	9.5	4.43	4.33
10.5	"	9.8	4.59	4.46
11	"	10.2	4.84	4.66
11.5	"	10.6	5.08	4.88
12	"	11.1	5.33	5.13
12.5	"	11.7	5.54	5.38
13	"	12.2	5.81	5.59
13.5	"	12.7	6.07	5.86
14	"	13.5	6.85	6.12
14.5	"		8.4	
15	"			
15.5	"			



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
[illegible]ter Griffiths	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Shear	27.11.01	10.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Location
Position **V5**

Comments

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

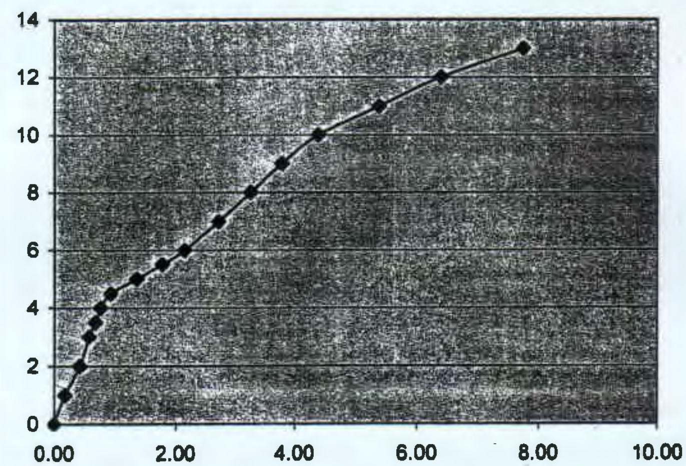
Base Material
Brick

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
1	"	1	0.18	0.19
2	"	2	0.42	0.43
3	"	2.95	0.57	0.58
3.5	"	3.5	0.67	0.68
4	"	3.9	0.76	0.77
4.5	"	4.4	0.93	0.96
5	"	5	1.36	1.47
5.5	"	5	1.78	1.89
6	"	6	2.15	2.29
7	"	7	2.72	3.86
8	"	8	3.26	3.46
9	"	9	3.79	4.00
10	"	10	4.39	4.75
11	"	11	5.38	5.70
12	"	12	6.39	6.95
13	"	13	7.74	8.44
14	"	test aborted		



Discrepancy between digital and analogue readings of load

digital	analogue
2kn	3kn
6kn	8kn
12kn	15kn
14kn	17kn

Readings V1,V2 & V5 only using digital load cell

Load cell and meter found to be unstable and unreliable when tested in the instument Lab. next day.

Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
Peter Griffiths	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Pull out	27.11.01	10.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position **P1**

Base Material
Brick

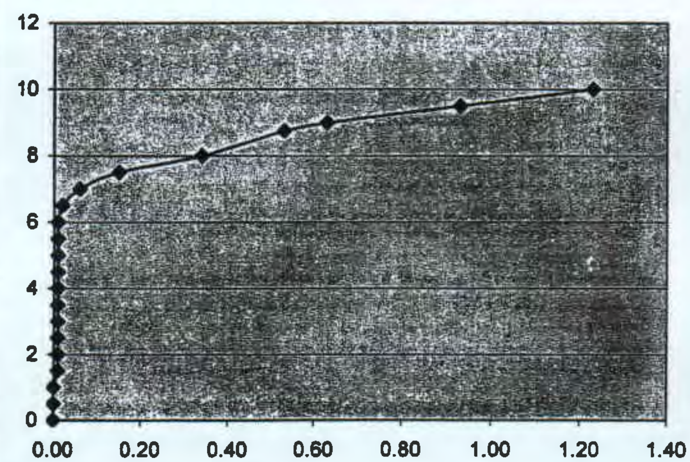
Comments
Bond failure

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
0.5	"	0.5	0.00	0.00
1	"	1	0.00	0.00
1.5	"	1.5	0.01	0.01
2	"	2	0.01	0.01
2.5	"	2.5	0.01	0.01
3	"	3	0.01	0.01
3.5	"	3.5	0.01	0.01
4	"	4	0.01	0.01
4.5	"	4.5	0.01	0.01
5	"	5	0.01	0.01
5.5	"	5.5	0.01	0.01
6	"	6	0.01	0.01
6.5	"	6.1	0.02	0.04
7	"	6.3	0.06	0.09
7.5	"	6.6	0.15	0.20
8	"	7.1	0.34	0.40
8.75	"	7.7	0.53	0.57
9	"	8.1	0.63	0.67
9.5	"	8.3	0.93	0.95
10	"	9.3	1.23	1.35



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
ter Griffiths	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Pull out	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position **P2**

Base Material
Brick

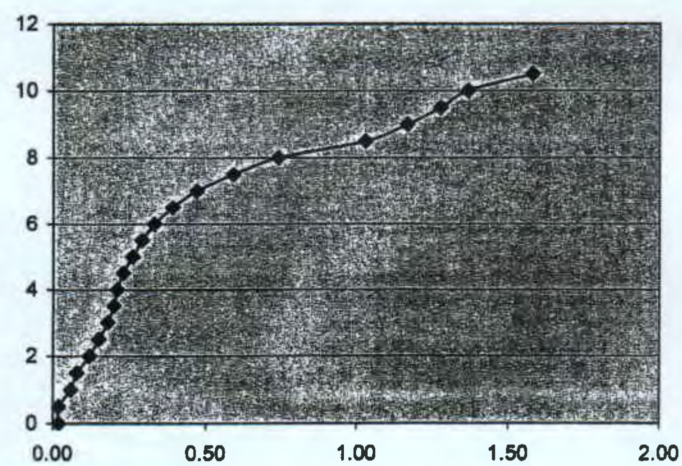
Comments
Bond Failure

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.02	
0.5	"	0.5	0.02	0.02
1	"	1	0.06	0.06
1.5	"	1.5	0.08	0.08
2	"	2	0.12	0.12
2.5	"	2.5	0.15	0.15
3	"	3	0.18	0.18
3.5	"	3.5	0.20	0.20
4	"	4	0.21	0.22
4.5	"	4.5	0.23	0.24
5	"	5	0.26	0.27
5.5	"	5.2	0.29	0.30
6	"	5.6	0.33	0.34
6.5	"	6	0.39	0.40
7	"	6.5	0.47	0.50
7.5	"	7	0.59	0.62
8	"	6.9	0.74	0.79
8.5	"	7.5	1.03	1.06
9	"	8.1	1.17	1.20
9.5	"	9	1.28	1.31
10	"	9.5	1.37	1.41
10.5	"	9.5	1.58	1.62
11	"			
relaxed load				
0			1.14	



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
eve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

194

Type of Test:	Install Date	Test Date
Pull out	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position **P3**

Base Material
Brick

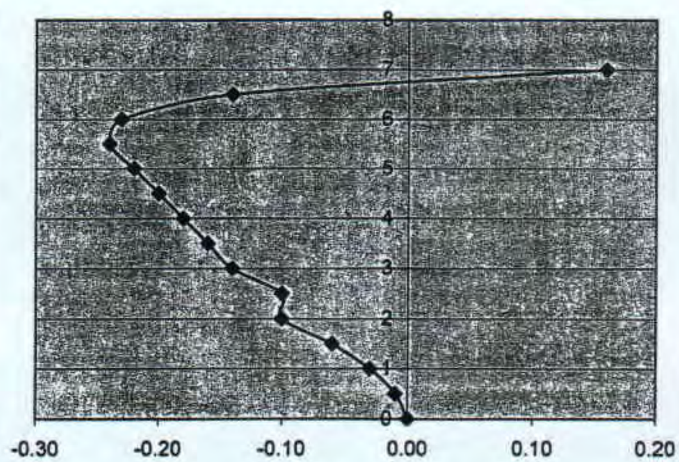
Comments
Brick Failure
Pull not straight creating lever
arm 100 mm long, result,
Negative readings

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
0.5	"	0.5	-0.01	-0.01
1	"	1	-0.03	-0.03
1.5	"	1.5	-0.06	-0.06
2	"	2	-0.10	-0.10
2.5	"	2.5	-0.10	-0.10
3	"	3	-0.14	-0.14
3.5	"	3.5	-0.16	-0.16
4	"	4	-0.18	-0.18
4.5	"	4.2	-0.20	-0.19
5	"	5	-0.22	-0.21
5.5	"	5.25	-0.24	-0.22
6	"	5.6	-0.23	-0.02
6.5	"	6	-0.14	0.12
7	"	6.5	0.16	0.20
7.5	"			



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Berislav Kralj	W S Atkins		
Steve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Pull out	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position **P4**

Base Material
Brick

Comments
100 lever arm producing
negative reading direction
as bolt straightens

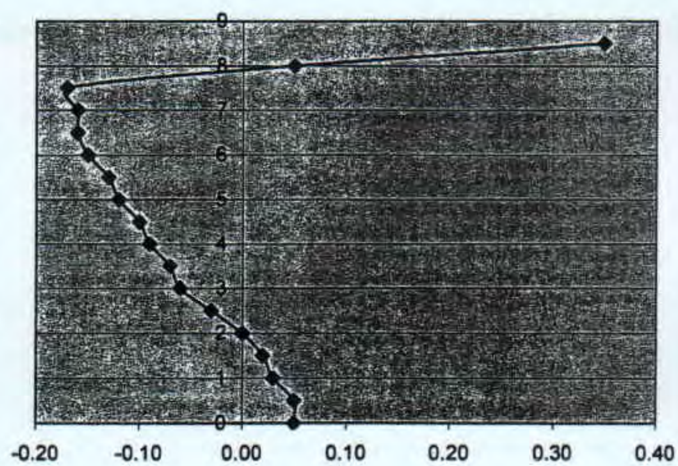
Required Load

Grout Type
Presstec standard

Max load sustained 8kN
Bond failure

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.05	
0.5	"	0.5	0.05	0.05
1	"	1	0.03	0.05
1.5	"	1.5	0.02	0.05
2	"	2	0.00	-0.01
2.5	"	2.5	-0.03	-0.01
3	"	3	-0.06	-0.01
3.5	"	3.5	-0.07	-0.01
4	"	4	-0.09	-0.01
4.5	"	4.2	-0.10	-0.01
5	"	4.9	-0.12	-0.01
5.5	"	5.3	-0.13	-0.01
6	"	5.9	-0.15	0.14
6.5	"	6.25	-0.16	0.14
7	"	6.8	-0.16	0.14
7.5	"	7.2	-0.17	0.16
8	"	6.7	0.05	0.14
8.5			0.35	



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
[illegible]eve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

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This Certificate must be filled in full and be signed by the Testing Officer and at least one Witness

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Pull out	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position **P5**

Base Material
Brick

Comments
See previous lever arm notes

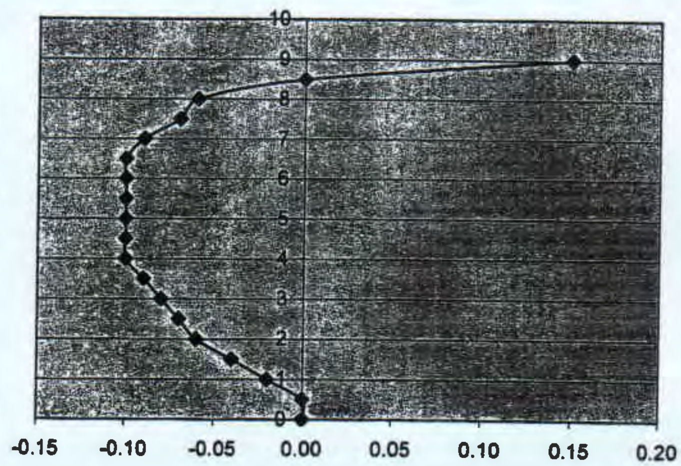
Bond Failure

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
0.5	"	0.5	0.00	0.00
1	"	1	-0.02	-0.02
1.5	"	1.5	-0.04	-0.04
2	"	2	-0.06	-0.06
2.5	"	2.5	-0.07	-0.07
3	"	2.9	-0.08	-0.08
3.5	"	3.25	-0.09	-0.09
4	"	4	-0.10	-0.10
4.5	"	4.25	-0.10	-0.10
5	"	4.5	-0.10	-0.11
5.5	"	5.2	-0.10	-0.11
6	"	5.8	-0.10	-0.11
6.5	"	6.2	-0.10	-0.09
7	"	6.9	-0.09	-0.08
7.5	"	7.2	-0.07	-0.07
8	"	7.8	-0.06	-0.05
8.5	"	7.5	0.00	0.02
9	"	8	0.15	0.18



Company Name:

Engineers Name:

Company Address:

Engineers Address:

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
[illegible]ve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

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This Certificate must be filled in full and be signed by the Testing Officer and at least one Witness

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Pull out	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position V2

Base Material
Brick

Comments
Pull out test on V2 previously tested in shear, to determine if there was any residual bond remaining.

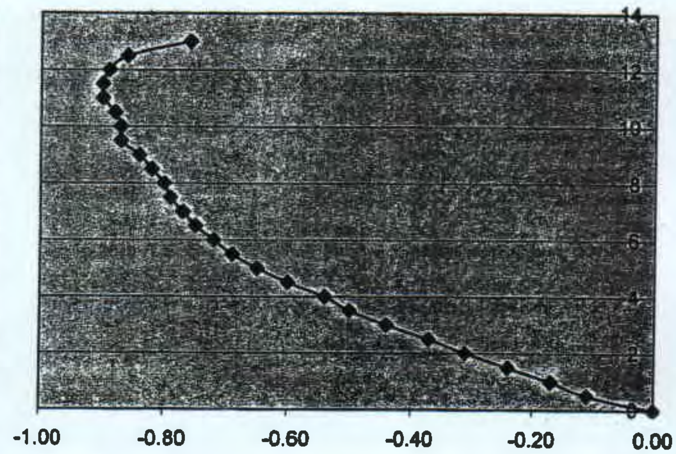
Bond failure

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
0.5	"	0.5	-0.11	-0.11
1	"	1	-0.17	-0.17
1.5	"	1.5	-0.24	-0.25
2	"	2	-0.31	-0.31
2.5	"	2.5	-0.37	-0.38
3	"	3	-0.44	-0.45
3.5	"	3.5	-0.50	-0.50
4	"	4	-0.54	-0.55
4.5	"	4.5	-0.60	-0.61
5	"	5	-0.65	-0.65
5.5	"	5.5	-0.69	-0.70
6	"	6	-0.72	-0.73
6.5	"	6.3	-0.75	-0.75
7	"	6.9	-0.77	-0.77
7.5	"	7.2	-0.79	-0.79
8	"	7.8	-0.80	-0.80
8.5	"	8	-0.82	-0.82
9	"	8.5	-0.84	-0.84
9.5	"	9.1	-0.87	-0.86
10	"	9.6	-0.87	-0.87
10.5	"	10	-0.88	-0.88
11	"	10.5	-0.9	-0.88
11.5	"	10.75	-0.9	-0.88
12	"	11	-0.89	-0.86
12.5	"	11.8	-0.86	-0.8
13	"	11.5	-0.76	-0.7



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
[illegible]ve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Shear	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position **V6**

Base Material
Brick

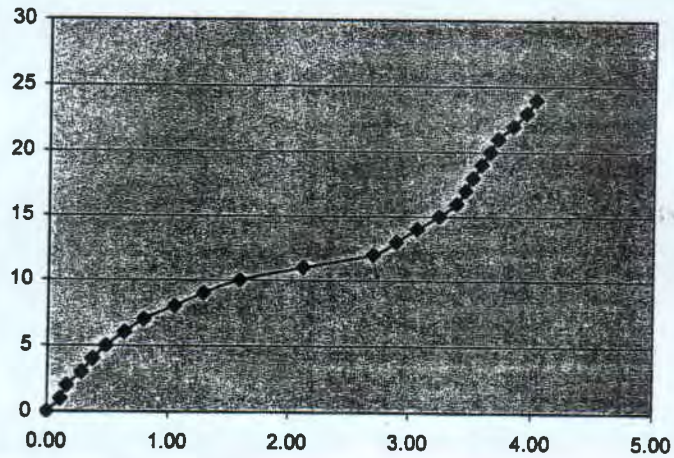
Comments
Hydraulic gauge only
load cell unreliable & meter
had a fault and would not hold
any reading evan when
disconnected from load cell

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
1	"	1	0.11	0.12
2	"	2	0.16	0.16
3	"	3	0.28	0.28
4	"	4	0.37	0.37
5	"	5	0.48	0.48
6	"	6	0.63	0.63
7	"	7	0.79	0.80
8	"	8	1.05	1.05
9	"	9	1.29	1.29
10	"	10	1.59	1.60
11	"	11	2.12	2.13
12	"	12	2.70	2.73
13	"	13	2.89	2.90
14	"	14	3.06	3.08
15	"	14.25	3.24	3.25
16	"	15	3.38	3.39
17	"	16	3.45	3.45
18	"	17	3.51	3.51
19	"	18	3.58	3.58
20	"	19	3.65	3.66
21	"	20	3.71	3.72
22	"	20	3.83	3.83
23	"	22	3.94	3.94
24	"	22	4.02	4.11
	"			
0				3.93



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
Steve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

199

Type of Test:	Install Date	Test Date
Shear	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location	
Position	**H10**

Base Material
Brick

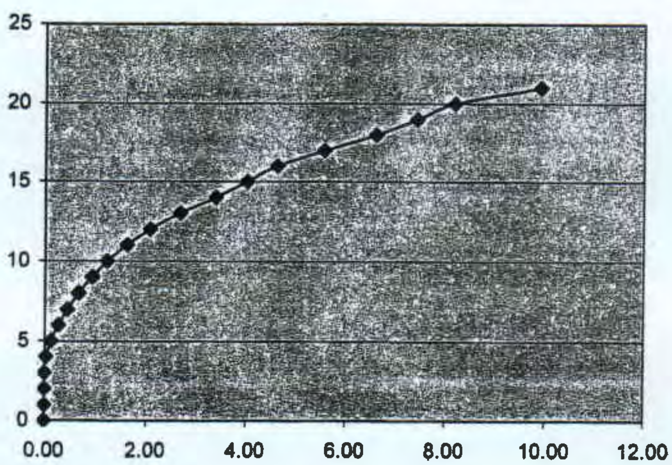
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
1	"	1	0.00	0.00
2	"	2	0.00	0.00
3	"	3	0.00	0.00
4	"	4	0.03	0.04
5	"	5	0.13	0.13
6	"	6	0.28	0.29
7	"	7	0.45	0.46
8	"	8	0.67	0.68
9	"	9	0.93	0.95
10	"	10	1.22	1.23
11	"	11	1.62	1.65
12	"	12	2.08	2.11
13	"	13	2.68	2.71
14	"	14	3.39	3.43
15	"	14.5	4.02	4.06
16	"	15	4.63	4.67
17	"	16	5.57	5.62
18	"	17	6.61	6.67
19	"	18	7.44	7.48
20	"	19	8.19	8.23
21	"		9.92	
0				8.26



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
Steve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

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CINTEC

Test Certificate

200

Type of Test:	Install Date	Test Date
Shear	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position V3

Base Material
Brick

Comments
V3 Damaged brick, hole passes through to cavity.

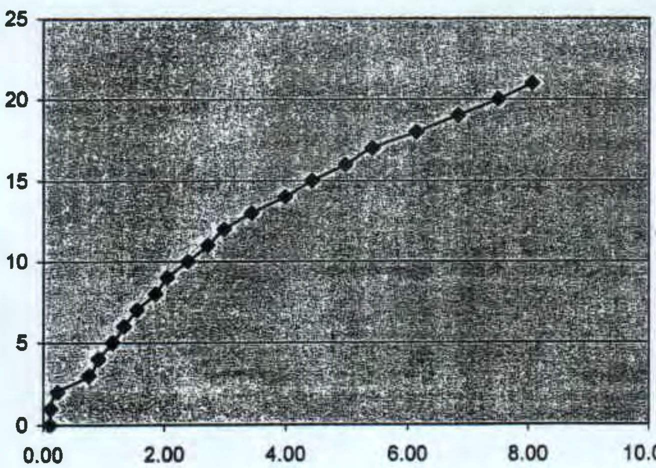
Test pushing anchor into cavity

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.12	
1	"	1	0.13	0.13
2	"	2	0.24	0.24
3	"	3	0.75	0.75
4	"	4	0.91	0.95
5	"	5	1.14	1.17
6	"	6	1.33	1.36
7	"	7	1.55	1.58
8	"	8	1.86	1.90
9	"	9	2.05	2.07
10	"	10	2.39	2.43
11	"	11	2.72	2.75
12	"	12	2.99	3.02
13	"	13	3.44	3.49
14	"	14	3.99	4.04
15	"	15	4.43	4.46
16	"	15	4.98	5.05
17	"	17	5.42	5.45
18	"	17	6.14	6.18
19	"	18	6.83	6.89
20	"	19	7.5	7.6
21	"	20	8.07	8.13
0				6.84



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
Steve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

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This Certificate must be filled in full and be signed by the Testing Officer and at least one Witness

CINTEC

Test Certificate

201

Type of Test:	Install Date	Test Date
Shear	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position V4

Base Material
Brick

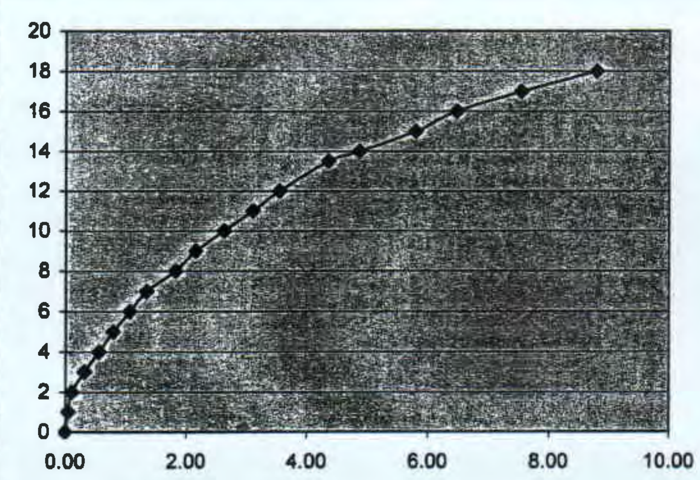
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
1	"	1	0.05	0.05
2	"	2	0.11	0.11
3	"	3	0.31	0.31
4	"	4	0.55	0.56
5	"	5	0.78	0.80
6	"	6	1.06	1.08
7	"	7	1.34	1.37
8	"	8	1.82	1.85
9	"	9	2.16	2.18
10	"	10	2.64	2.68
11	"	11	3.11	3.15
12	"	12	3.55	3.59
13.5	"	13	4.36	4.42
14	"	14	4.86	4.92
15	"	14	5.79	5.84
16	"	15.5	6.47	6.53
17	"	16	7.54	7.59
18	"	16	8.80	8.90
0				7.36



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
Steve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

203

Type of Test:	Install Date	Test Date
Shear	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position V7

Base Material
Brick

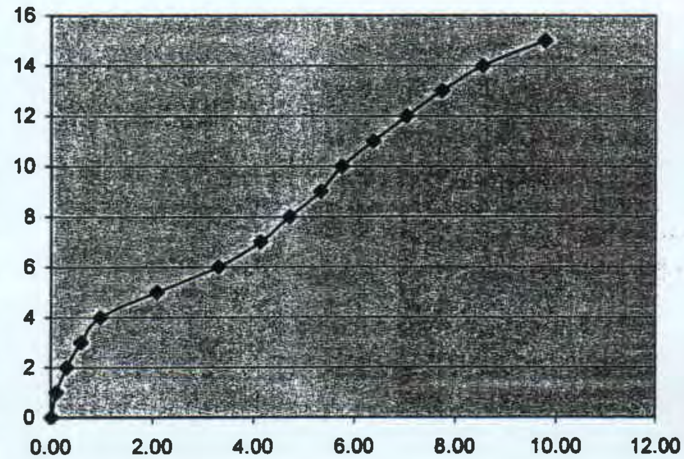
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.02	
1	"	1	0.11	0.11
2	"	2	0.31	0.31
3	"	3	0.59	0.59
4	"	4	0.97	0.99
5	"	5	2.09	2.11
6	"	6	3.31	3.33
7	"	7	4.14	4.16
8	"	8	4.72	4.75
9	"	9	5.33	5.36
10	"	10	5.75	5.78
11	"	11	6.38	6.42
12	"	12	7.04	7.09
13	"	12	7.74	7.79
14	"	13.5	8.54	8.59
15	"	14	9.78	9.82



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Steve Fitzgibbon	Babtie		
Tom Bulkeley	Magnox		
Peter Smith	Magnox		

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This Certificate must be filled in full and be signed by the Testing Officer and at least one Witness

CINTEC

Test Certificate

204

Type of Test:	Install Date	Test Date
Shear	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position V9

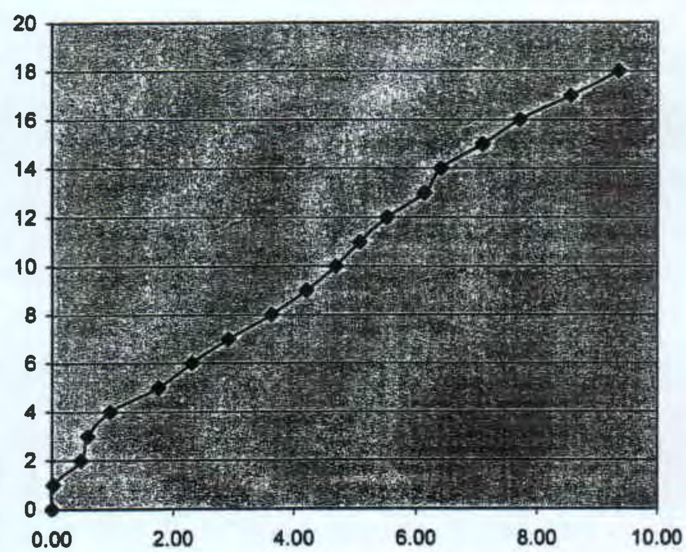
Base Material
Brick

Comments

Required Load

Grout Type
Presstec standard

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.01	
1	"	1	0.03	0.03
2	"	2	0.48	0.48
3	"	3	0.59	0.60
4	"	4	0.96	0.97
5	"	5	1.76	1.78
6	"	6	2.31	2.33
7	"	7	2.91	2.93
8	"	8	3.63	3.64
9	"	9	4.20	4.22
10	"	10	4.68	4.71
11	"	11.1	5.08	5.10
12	"	12	5.52	5.58
13	"	13.2	6.13	6.15
14	"	13.9	6.41	6.47
15	"	14.7	7.09	7.12
16	"	16	7.70	7.76
17	"	17	8.56	8.61
18	"		9.36	9.43
0				7.75



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Peter Smith	Magnox		
Steve Fitzgibbon	Babtie		
Tom Bulkeley	Magnox		

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This Certificate must be filled in full and be signed by the Testing Officer and at least one Witness

CINTEC

Test Certificate

205

Type of Test:	Install Date	Test Date
Shear	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position **V10**

Base Material
Brick

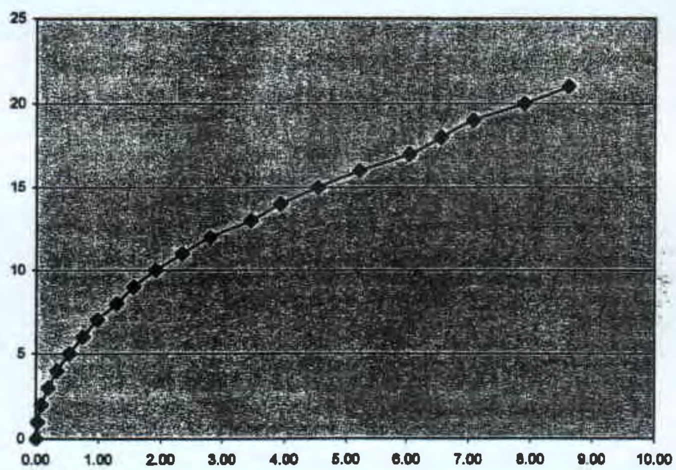
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,	0	0.01	
1	"	1	0.03	0.03
2	"	2	0.09	0.10
3	"	3	0.20	0.21
4	"	4	0.34	0.35
5	"	5	0.54	0.55
6	"	6	0.75	0.78
7	"	7	0.98	0.99
8	"	8	1.29	1.33
9	"	9	1.56	1.59
10	"	10	1.91	1.94
11	"	11	2.33	2.36
12	"	12	2.79	2.82
13	"	13	3.46	3.49
14	"	14	3.93	3.96
15	"	15	4.54	4.59
16	"	15.8	5.21	5.26
17	"	16	6.04	6.09
18	"	17	6.55	6.60
19	"	19	7.07	7.12
20	"	20	7.89	7.94
21	"	20	8.61	8.67
0				7.16



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Peter Smith	Magnox		
Steve Fitzgibbon	Babtie		
Tom Bulkeley	Magnox		

Cintec International Limited, Cintec House, 11 Goldtops, Newport, South Wales, NP20 4PH Tel: 01633 246614 - Fax: 01633 246110
This Certificate must be filled in full and be signed by the Testing Officer and at least one Witness

CINTEC

Test Certificate

206

Type of Test:	Install Date	Test Date
Shear	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position H9

Base Material
Brick

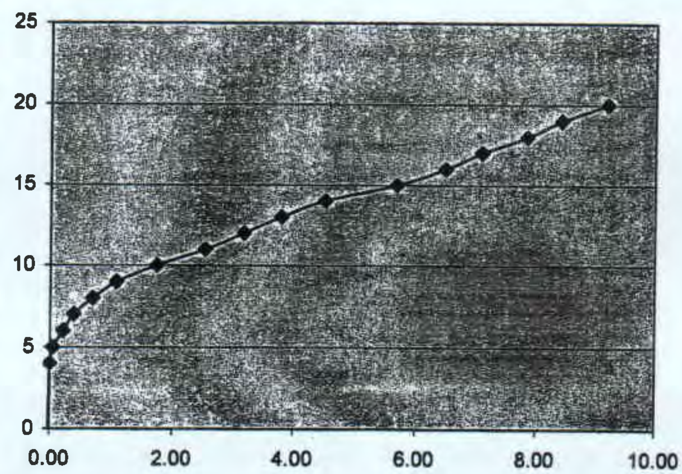
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		-0.02	
1	"	1	-0.02	-0.02
2	"	2	-0.01	-0.01
3	"	3	-0.01	-0.01
4	"	4	0.00	0.00
5	"	5	0.05	0.05
6	"	6	0.22	0.23
7	"	7	0.39	0.40
8	"	8	0.70	0.71
9	"	9	1.09	1.10
10	"	10	1.74	1.76
11	"	11	2.54	2.55
12	"	12	3.18	3.20
13	"	13	3.80	3.82
14	"	14	4.51	4.54
15	"	14	5.68	5.72
16	"	15.5	6.49	6.53
17	"	16	7.09	7.12
18	"	17	7.84	7.88
19	"	18	8.42	8.48
20	"	19	9.2	9.26
0				8.11



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Peter Smith	Magnox		
Steve Fitzgibbon	Babtie		
Sophie Edgar	W S Atkins		

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Shear	27.11.01	11.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position H7

Base Material
Brick

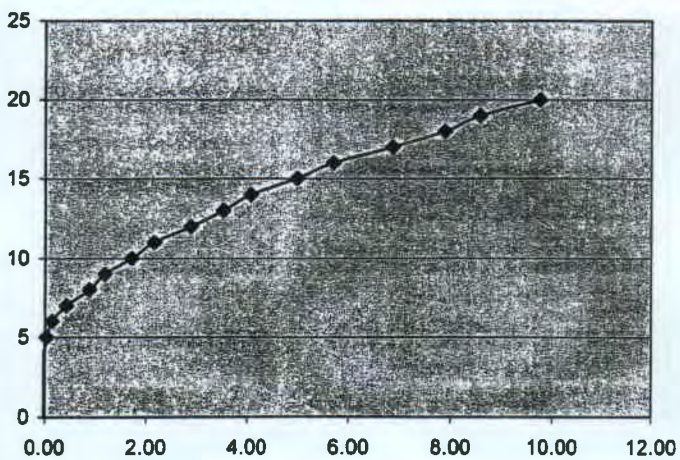
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		-0.02	
1	"	1	-0.02	-0.02
2	"	2	-0.02	-0.02
3	"	3	-0.02	-0.02
4	"	4	-0.01	-0.01
5	"	5	0.04	0.04
6	"	6	0.15	0.15
7	"	7	0.43	0.44
8	"	8	0.88	0.89
9	"	9	1.19	1.21
10	"	10	1.72	1.74
11	"	11	2.16	2.18
12	"	12	2.88	2.91
13	"	12.5	3.52	3.54
14	"	13	4.07	4.10
15	"	15	5.00	5.05
16	"	15.5	5.71	5.74
17	"	16	6.85	6.88
18	"	17	7.88	7.96
19	"	18	8.57	8.62
20	"	19	9.76	9.84
0				8.42



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
Steve Fitzgibbon	Babtie		
Peter Smith	Magnox		

CINTEC

Test Certificate

208

Type of Test:	Install Date	Test Date
Shear	27.11.01	12.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position H8

Base Material
Brick

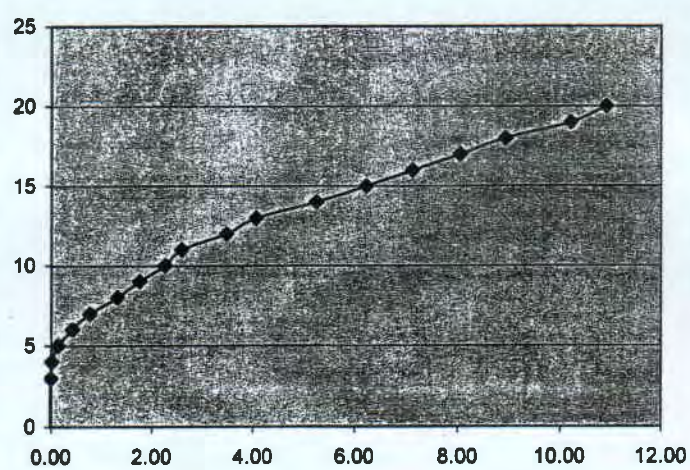
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		-0.01	
1	"	1	-0.01	-0.01
2	"	2	-0.01	-0.01
3	"	3	0.00	0.00
4	"	4	0.02	0.03
5	"	5	0.14	0.15
6	"	6	0.42	0.44
7	"	7	0.76	0.78
8	"	8	1.32	1.35
9	"	9	1.76	1.79
10	"	10	2.26	2.30
11	"	11	2.59	2.62
12	"	11.9	3.48	3.53
13	"	12.25	4.06	4.09
14	"	13.5	5.25	5.32
15	"	14	6.24	6.30
16	"	15	7.13	7.20
17	"	16	8.05	8.12
18	"	16.25	8.94	9.02
19	"	18	10.22	10.31
20	"	18	10.94	11.01
0				9.65



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
Steve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

Cintec International Limited, Cintec House, 11 Goldtops, Newport, South Wales, NP20 4PH Tel: 01633 246614 - Fax: 01633 246110

This Certificate must be filled in full and be signed by the Testing Officer and at least one Witness

CINTEC

Test Certificate

209

Type of Test:	Install Date	Test Date
Shear	27.11.01	12.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position H6

Base Material
Brick

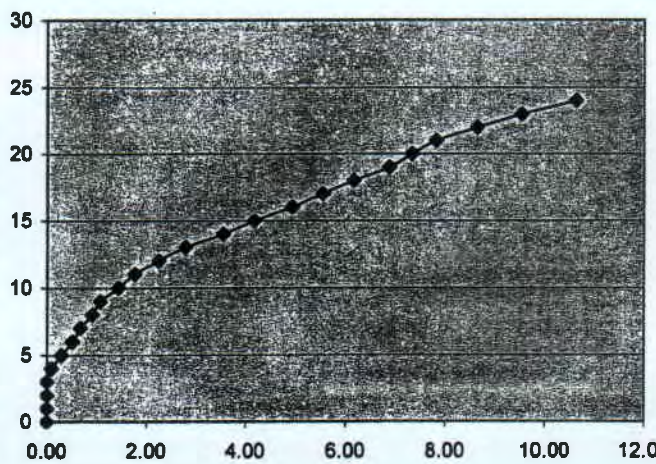
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
1	"	1	0.00	0.00
2	"	2	0.00	0.00
3	"	3	0.00	0.00
4	"	4	0.09	0.09
5	"	5	0.29	0.29
6	"	6	0.51	0.51
7	"	7	0.67	0.67
8	"	8	0.91	0.92
9	"	9	1.08	1.09
10	"	10	1.44	1.46
11	"	11	1.77	1.79
12	"	12	2.25	2.27
13	"	12.5	2.79	2.81
14	"	13.5	3.55	3.58
15	"	14.5	4.18	4.22
16	"	15.5	4.93	4.97
17	"	16.5	5.55	5.62
18	"	17	6.17	6.22
19	"	18	6.88	6.92
20	"	20	7.33	7.77
21	"	20	7.82	
22	"	21	8.64	8.68
23	"	21.5	9.54	9.59
24	"	22	10.64	10.71
0				9.47



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Sophie Edgar	W S Atkins		
Steve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

Cintec International Limited, Cintec House, 11 Goldtops, Newport, South Wales, NP20 4PH Tel: 01633 246614 - Fax: 01633 246110

This Certificate must be filled in full and be signed by the Testing Officer and at least one Witness

CINTEC

Test Certificate

210

Type of Test:	Install Date	Test Date
Shear	27.11.01	12.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position H4

Base Material
Brick

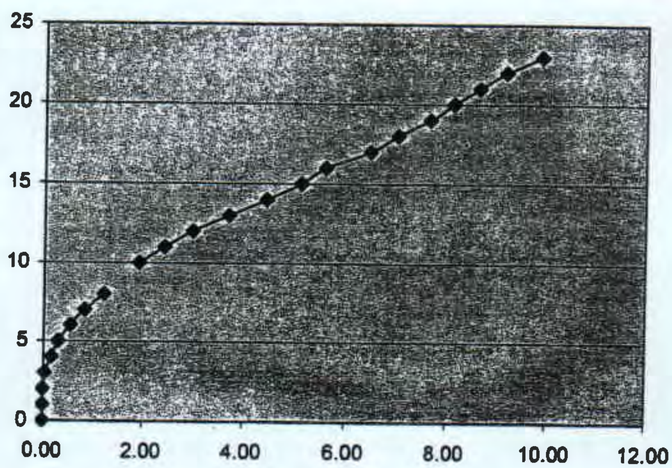
Comments
missed reading

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
1	"	1	0.00	0.00
2	"	2	0.00	0.00
3	"	3	0.04	0.04
4	"	4	0.17	0.17
5	"	5	0.31	0.32
6	"	6	0.55	0.56
7	"	7	0.83	0.84
8	"	8	1.22	1.24
9				
10	"	10	1.92	1.95
11	"	11	2.43	2.46
12	"	12	2.98	3.01
13	"	13	3.70	3.74
14	"	14	4.44	4.47
15	"	15	5.12	5.16
16	"	15.5	5.60	5.66
17	"	17	6.47	6.53
18	"	17.5	7.02	7.09
19	"	19	7.66	7.74
20	"	19.5	8.13	8.2
21	"	20.5	8.65	8.71
22	"	21.5	9.17	9.24
23	"	22	9.89	9.95
0				8.81



Company Name:	Engineers Name:
Company Address:	Engineers Address:

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Borislav Kralj	W S Atkins		
Steve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

Type of Test:	Install Date	Test Date
Shear	27.11.01	12.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position H3

Base Material
Brick

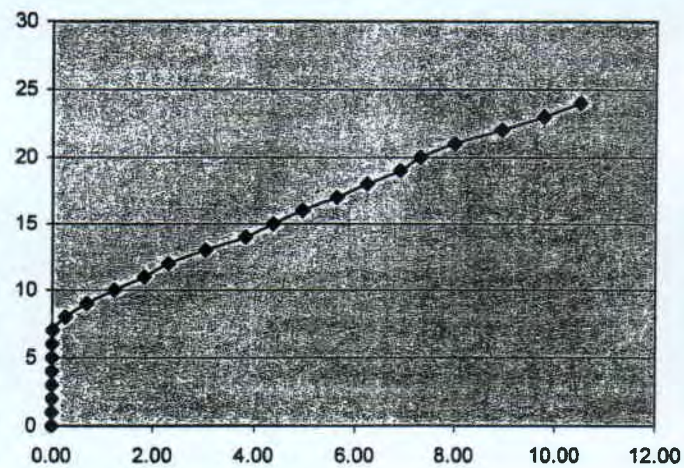
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.00	
1	"	1	0.00	0.00
2	"	2	0.00	0.00
3	"	3	0.00	0.00
4	"	4	0.00	0.00
5	"	5	0.00	0.00
6	"	6	0.00	0.00
7	"	7	0.01	0.01
8	"	8	0.27	0.28
9	"	9	0.69	0.70
10	"	10	1.23	1.26
11	"	11	1.82	1.85
12	"	12	2.30	2.33
13	"	13	3.04	3.07
14	"	13.5	3.82	3.87
15	"	14.5	4.37	4.42
16	"	15	4.95	5.00
17	"	16.5	5.62	5.65
18	"	17.5	6.23	6.20
19	"	18.5	6.89	6.92
20	"	19.5	7.3	7.34
21	"	20	7.99	8.02
22	"	21.5	8.93	8.99
23	"	22	9.77	9.84
24	"	23	10.5	10.68
0				9.4



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Berislav Kralj	W S Atkins		
Steve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

CINTEC

Test Certificate

242

Type of Test:	*Install Date*	*Test Date*
Shear	27.11.01	12.12.01

Site Address
Loading Bay
Reactor House
Wylfa
Anglesey

Anchor Type
10 mm Rebar

Embedment Depth
85 mm

Bore Hole Diameter
30 mm

Anchor Location
Position H1

Base Material
Brick

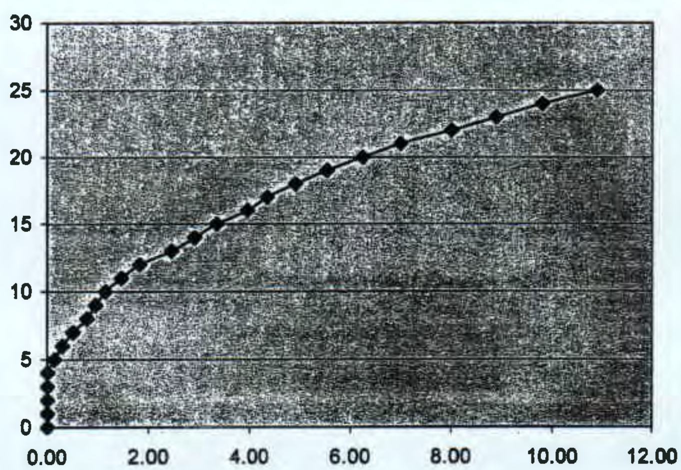
Comments

Required Load

Grout Type
Presstec standard

Anchor Material
Ferrous H.T.

Load - Start K.N.	Time Held	Load - End K.N.	Extension MM.	Extension at load end
0	30 secs,		0.02	
1	"	1	0.01	0.01
2	"	2	0.01	0.01
3	"	3	0.01	0.01
4	"	4	0.01	0.01
5	"	5	0.15	0.16
6	"	6	0.31	0.31
7	"	7	0.51	0.52
8	"	8	0.78	0.78
9	"	9	0.95	0.96
10	"	10	1.15	1.17
11	"	11	1.47	1.48
12	"	12	1.83	1.86
13	"	12.5	2.45	2.47
14	"	13.5	2.90	2.92
15	"	14.5	3.34	3.36
16	"	15.5	3.95	3.99
17	"	16	4.36	4.40
18	"	17	4.92	4.97
19	"	18	5.55	5.58
20	"	19.5	6.25	6.29
21	"	20.5	7	7.05
22	"	21	8.01	8.08
23	"	22	8.89	8.98
24	"	22	9.8	9.88
25	"	24	10.9	10.97
0				9.75



Company Name:	Engineers Name:
Company Address:	**Engineers Address:**

Any Stamps or Seals from Witnesses

Persons Present	Company	Position	Signature
Berislav Kralj	W S Atkins		
[illegible]eve Fitzgibbon	Babtie		
David Castree	Magnox LTSR		

Cintec International Limited, Cintec House, 11 Goldtops, Newport, South Wales, NP20 4PH Tel: 01633 246614 - Fax: 01633 246110

1

INSTALLATION LOG - CINTEC ANCHORS

26~27/11/01

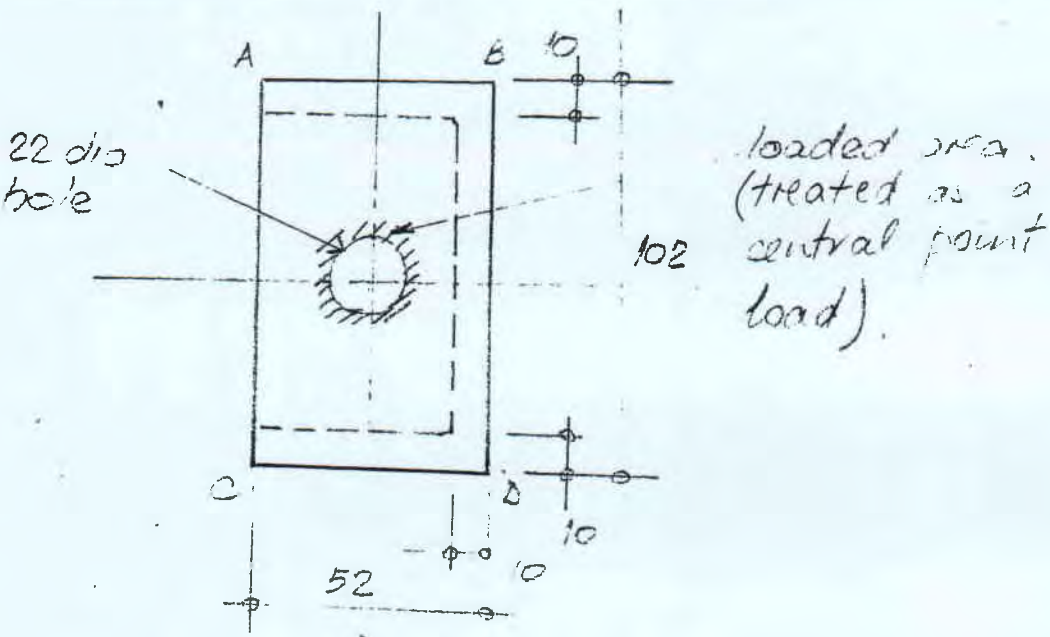
Tie #	Hole depth (mm)	Frog detected (Y/N)	Frog up/down	Frog void/full
P1	86	Y	Down	Void
P2	86	Y	Down	Void
P3	87	Y	Up	Full
P4	85	Y	Up	Full
P5	88	N	-	-
V1	87	Y	Down	Void
V2	86	Y	Down	Void
V3	98*	Y	Up	Full
V4	86	Y	Down	Void
V5	87	N	-	-
V6	86	Y	Down	Void
V7	88	N	-	-
V8	85	Y	Down	Void
V9	87	N	-	-
V10	88	N	-	-
H1	82	Y	Down	Void
H2	85	Y	Down	Void
H3	88	Y	Down	Void
H4	89	N	-	-
H5	98	Y	Down	Void
H6	85	N	-	-
H7	87	N	-	-
H8	86	N	-	-
H9	90	N	-	-
H10	85	N	-	-

* Brick damaged on cavity face

E

Calculation Sheet

Sheet No.: CT 01	Date: 27/12/01	Revision
Calculations by: DNC	Subject: Jacking channel endplate.	
Section: LTSR team.		
Checked by: D Francis	Plant item: Trial mill.	
	Location: NE loading bay.	
Date: 16.01.02.	Task No.: —	



10 mm thick end plate.

I for 10 mm plate $= \frac{10^3}{12}$ per mm width

Z for 10 mm plate $= \frac{10^2}{6}$ per mm width

So for a 25 kN load, hypothetical span of plate = 92 mm, deflection $\simeq \frac{1}{48} \times \frac{25 \times 92^3 \times 12}{210 \times 10^3 \times (52-22)}$

$= 0.77$ mm

NB this is the approximate deflection at the free edge AC, at BD deflection → 0

actual deflection at load $\simeq 0.77/2 = 0.385$ mm

This represents a minor part of the deflection (typ 10 mm)

Calculation Sheet

Sheet No.: CT02	Date: 27/12/01	Revision
Calculations by: ANC	Subject: Bending of bar	
Section: LTSR team.		
Checked by: P. Francis	Plant item: Trial wall.	
	Location: NE loading bay.	
Date: 16.01.02.	Task No.:	

Projecting part of 10 mm bar is threaded, so reducing the Z value. However if the bar is treated as an unthreaded 10 mm bar its $Z = \frac{d^3 \times \pi}{32} = 98\ mm^3$

and it will start to yield elastically at a bending moment of $> 460 \times 98$

$= 45\,080\ N.mm$

$= .045\ kN.m$

If the eccentricity of applied load is 15 mm $= .015\ m$, so non-elastic deformation may take place at loads $> \frac{.045}{.015} = 3\ kN$

So for the threaded bar, this 'yielding' load will be less than 3 kN.

Appendix F

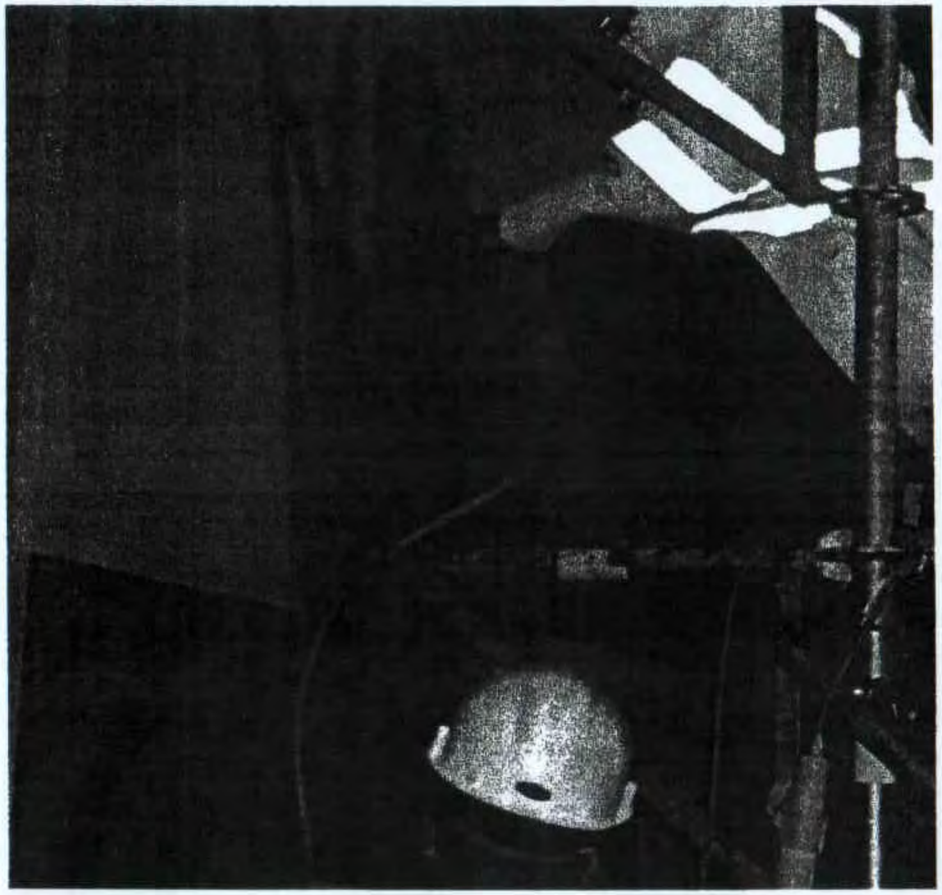
List of witnesses attending tests

Organisation:	Names:
BNFL Berkeley	Tom Bulkeley Peter Smith
BNFL/Magnox Wylfa	Trenholm Fisher Peter Francis David Castree
W S Atkins	Sophie Edgar Berislav Kralj
Babtie	Steve Fitzgibbon (11/12/01) Peter Griffie (10/12/01) Ian Drake (11/12/01)

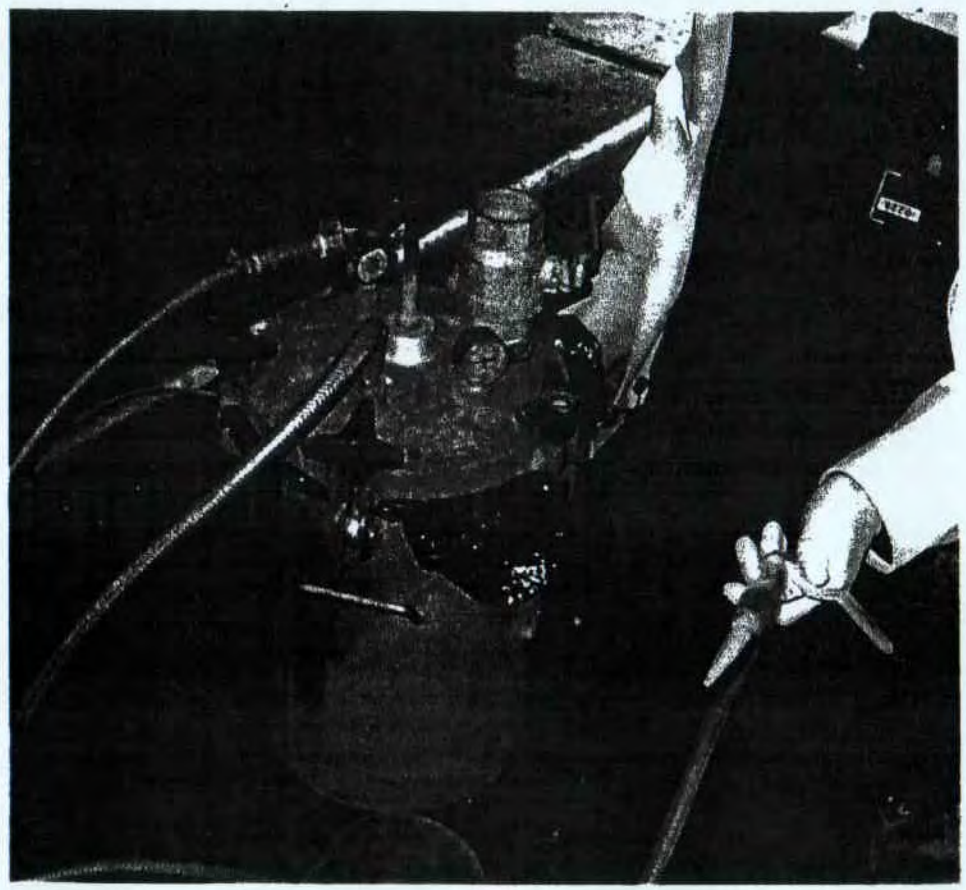
G

Drilling wall

Cleaning hole

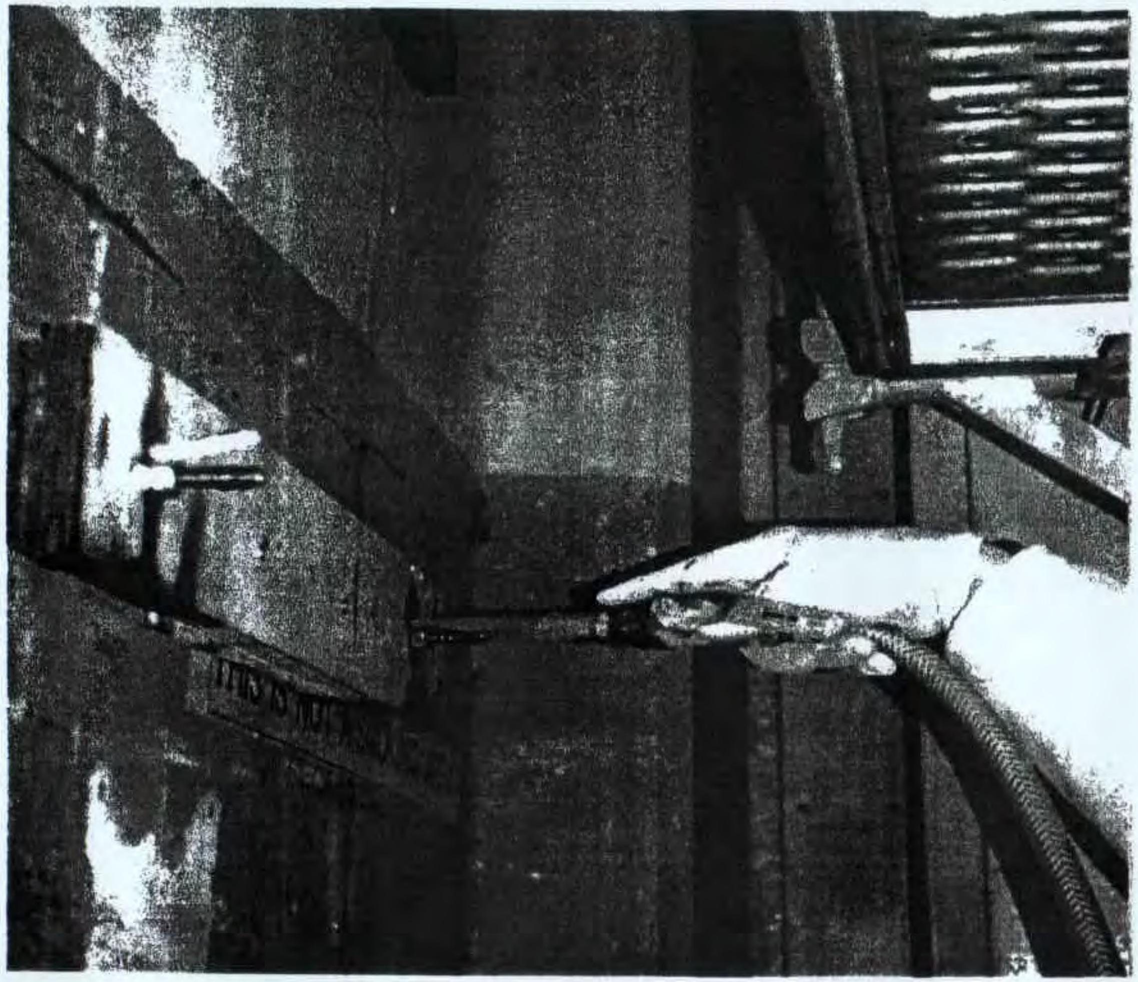


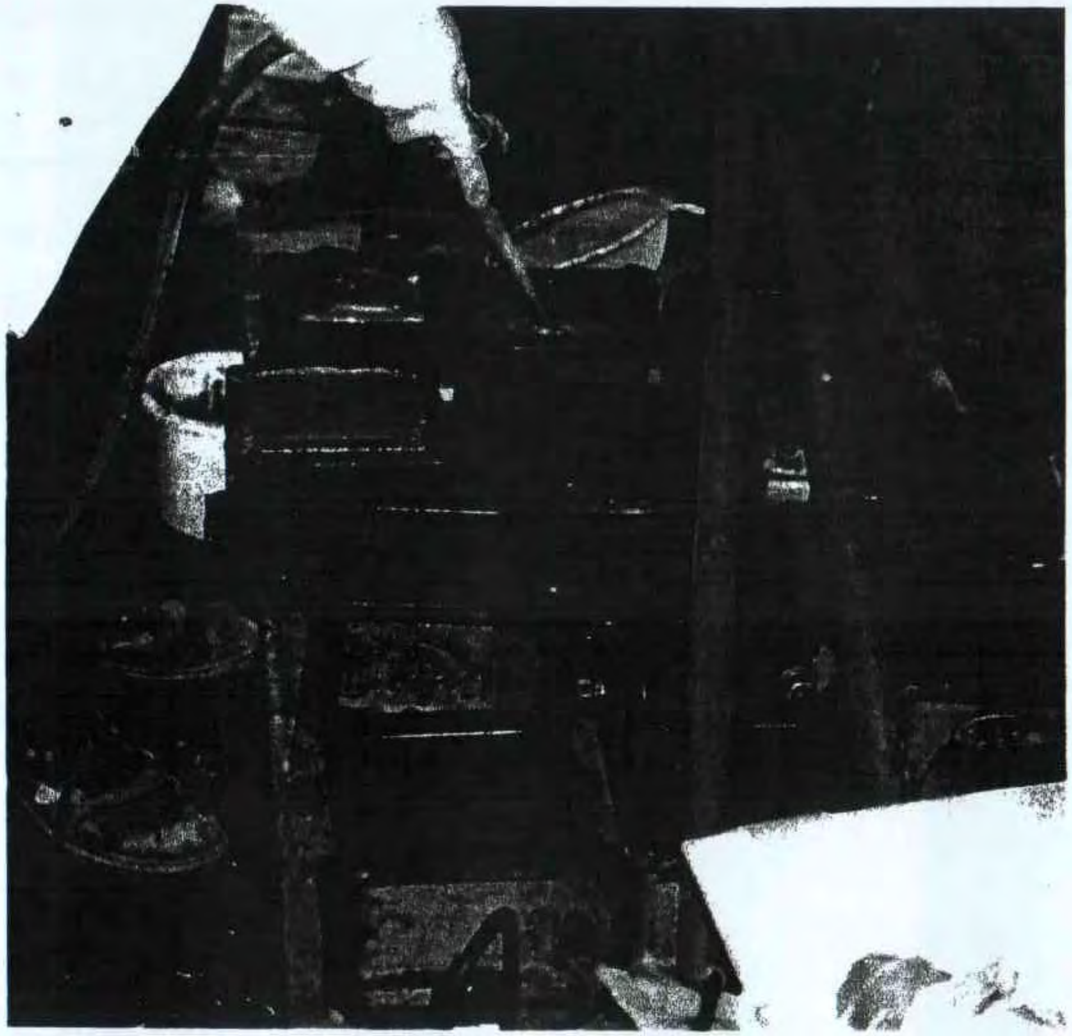
Filling the grout pressure flask.



Flask used for pressurising grout.

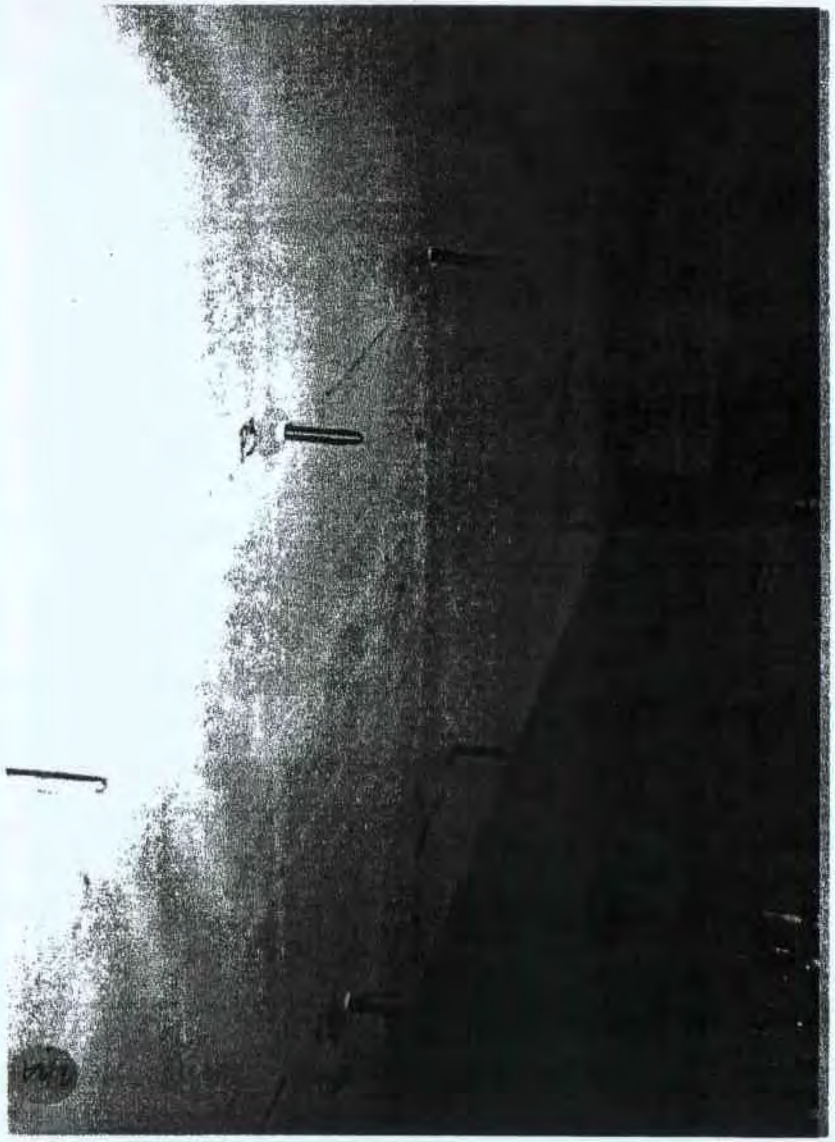
Injecting grout into sock through tube





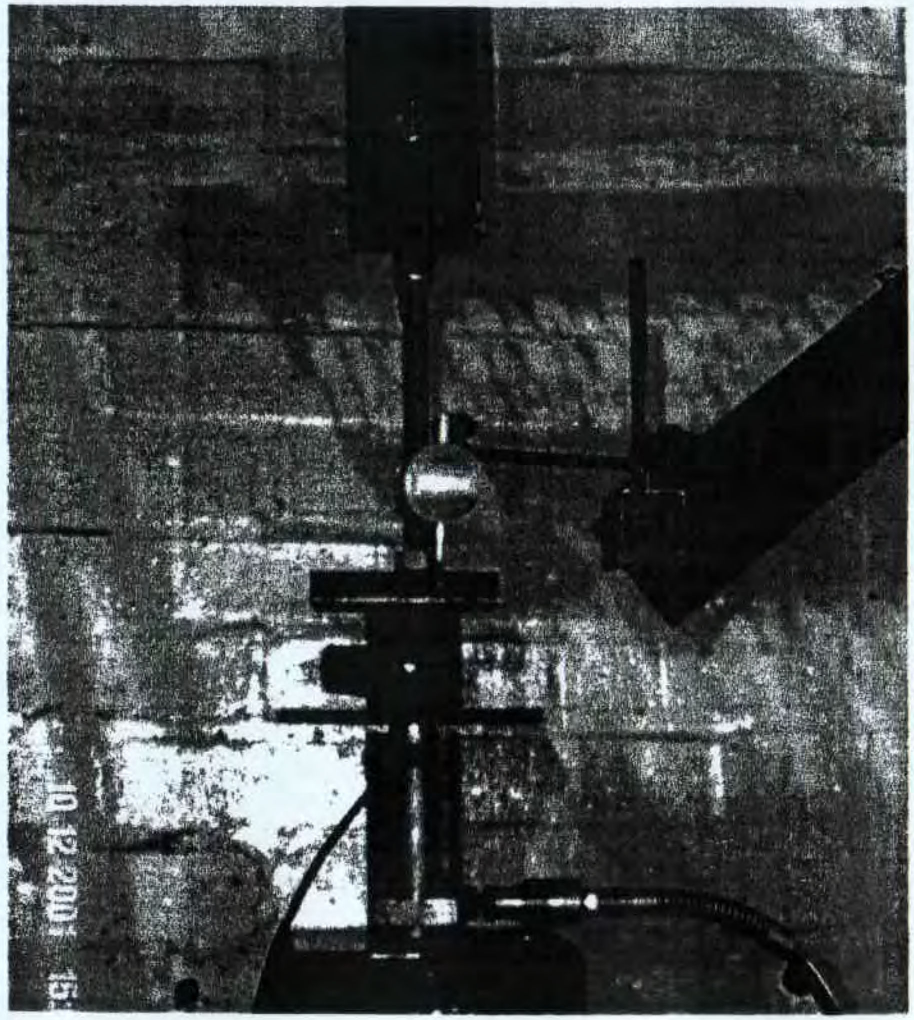
Filling test cube moulds using grout injection equipment.

Grout-inflated sock in dummy ‘brick’.

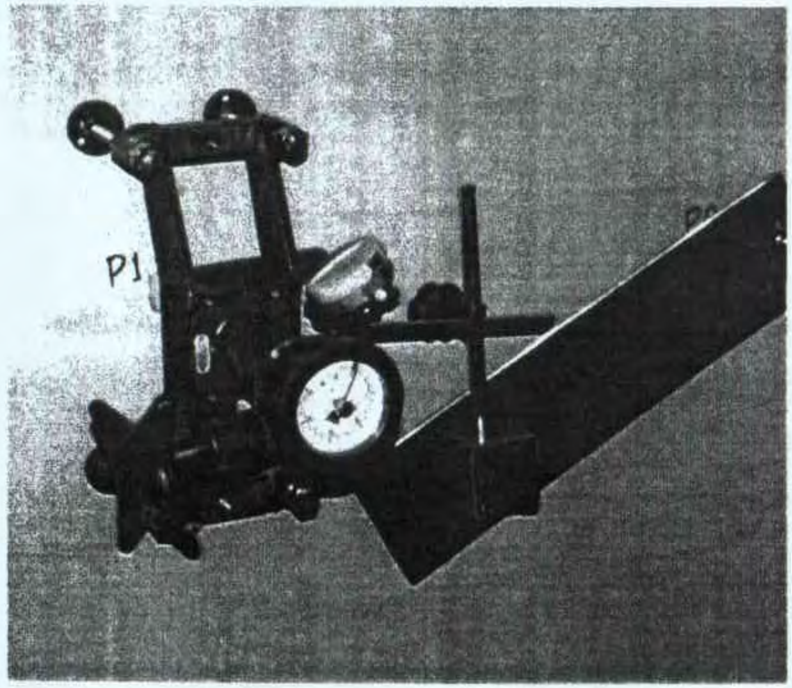


Part of the array of anchors immediately after grout injection.

Horizontal shear rig.



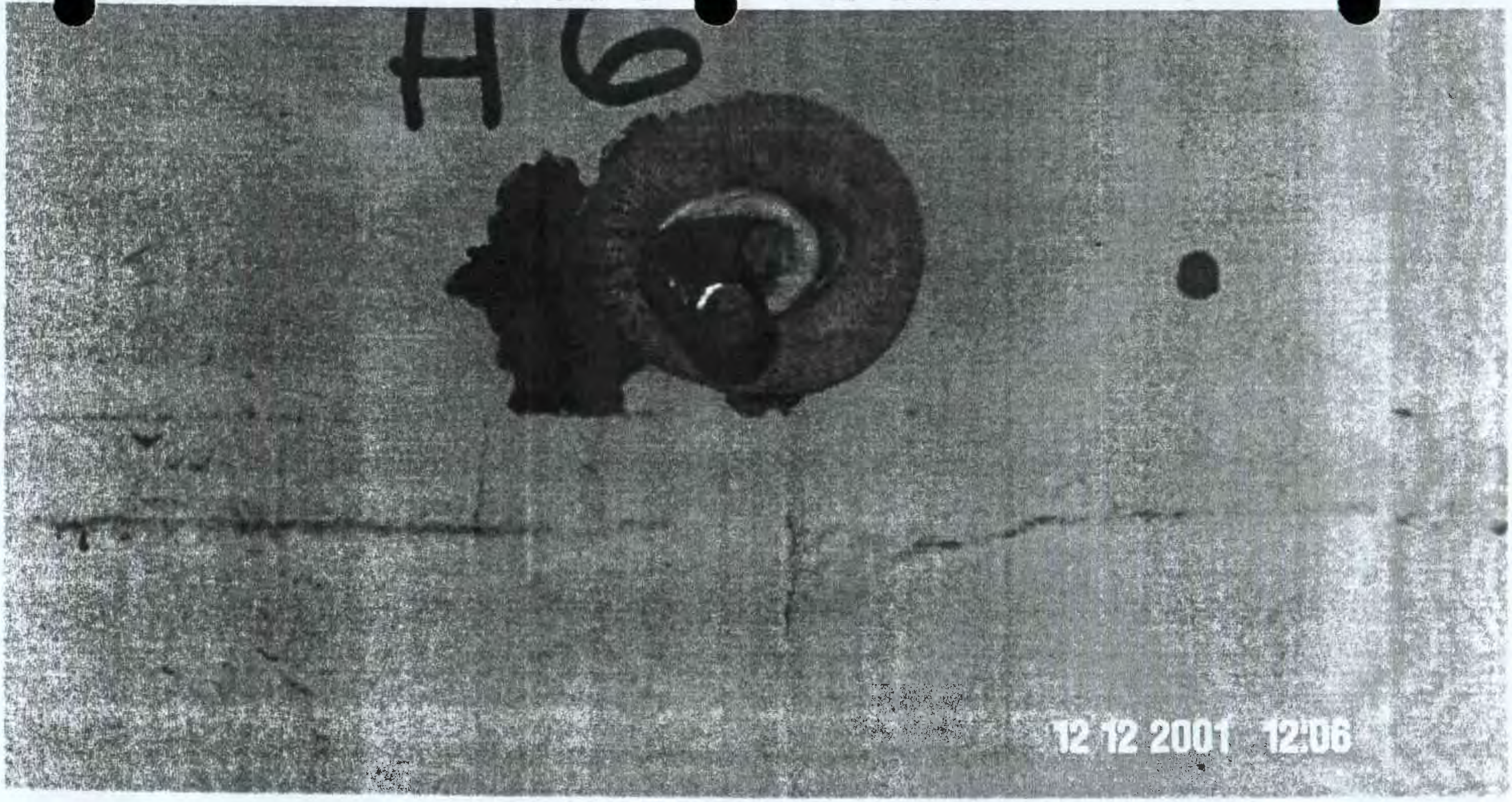
Vertical shear rig.



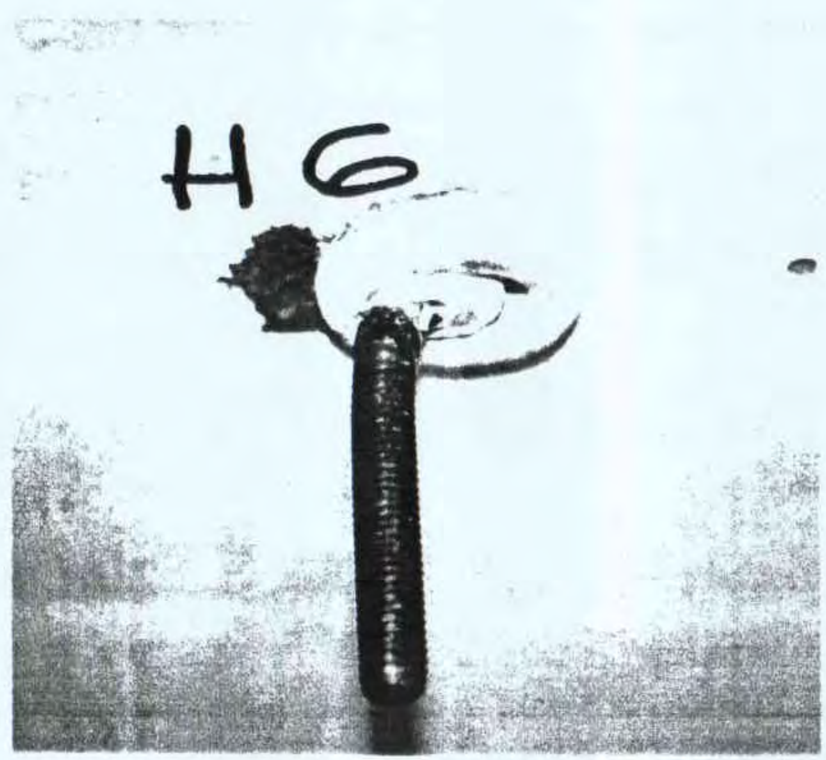
Pull-out rig in position on an anchor.

Typical anchor after vertical shear test. Note brickwork spalling and bar deformation.

Typical anchor after vertical shear testing showing significant vertical deformation of the projecting bar.



Typical anchor after horizontal shear test. Note spalling of brickwork and deformation of bar



Typical anchor after horizontal shear testing showing significant horizontal deformation of projecting part of bar.

HYDRAJAWS LIMITED

TESTING DIVISION

3 Tile Cross Trading Estate • Tile Cross Road • Birmingham • B33 ONW
Telephone (0121) 779 6856/7 •

Certificate of Calibration

GAUGE REF. No. ......55200..

GAUGE RANGE ......0-10KN..

MODEL No. ....2.42in2 AREA..

We certify that this Gauge is has been inspected and calibrated for accuracy and passed within our limits of plus or minus 2% F.S.D.

Results obtained are as follows:-

MASTER	KN	20.00	40.00	60.00	80.00	100.00	120.00
ACTUAL	KN	20.00	40.00	60.00	80.00	100.00	120.00

TRACEABILITY
Gauges manufactured to BS EN 837-1 1998

Calibration undertaken using an oil-operated pressure balance type 280H. Serial number M953 traceability to national physical laboratory (NPl) via certificate no. MP 03/00 P1085
Note: In all accordance with BS EN 30012-1 formally BS 5781 this certificate is valid for a period of 12 months from issue.

Accuracy of gauges as stated above cannot be guaranteed should the unit be subjected to mis-use. Gauge will be permanently damaged should maximum load to be exceeded.

This unit will be due for calibration on ....23rd AUGUST 2002........

Customer ....CINTEC INTERNATIONAL..........

Order No. ....10387.......... Date of Calibration ......23rd AUGUST 2001

Approved Signatory ....

Date ....23rd AUGUST 2001..........

HYDRAJAWS LIMITED

TESTING DIVISION

3 Tile Cross Trading Estate • Tile Cross Road • Birmingham • B33 ONW
Telephone (0121) 779 6856/7 •

Certificate of Calibration

GAUGE REF. No. 1764W

GAUGE RANGE 0-20KN

MODEL No. ....MK4 SER no 001632

We certify that this Gauge is has been inspected and calibrated for accuracy and passed within our limits of plus or minus 2% F.S.D.

Results obtained are as follows:-

MASTER	KN	4.00	8.00	12.00	16.00	20.00
ACTUAL	KN	4.00	8.00	12.00	16.10	20.20

TRACEABILITY
Gauges manufactured to BS EN 837-1 1998

Calibration undertaken using an oil-operated pressure balance type 280H. Serial number M953 traceability to national physical laboratory (NPl) via certificate no. MP 03/00 P1085
Note: In all accordance with BS EN 30012-1 formally BS 5781 this certificate is valid for a period of 12 months from issue.

Accuracy of gauges as stated above cannot be guaranteed should the unit be subjected to mis-use. Gauge will be permanently damaged should maximum load to be exceeded.

This unit will be due for calibration on ....23rd AUGUST 2002

Customer ....CINTEC INTERNATIONAL

Order No. ....10387 .... Date of Calibration 23rd AUGUST 2001

Approved Signatory ....

Date ....23rd AUGUST 2001

I

Blast Protection

Brent, UK. Gas Explosion Anchor Tests. Testing Engineers OVE ARUP and Partners (June, 1987)

On site structural testing and assessment of Cintec Anchors, Basildon, Essex, UK. Testing Building Research Establishment, UK (February, 1993)

Cintec Wall Trial. Testing by Cranfield University Ordnance Test and Evaluation Centre, UK (November, 1999)

Cintec Retro Reinforcement System to resist the effects of Blast Loads on Masonry structures.
Eur Ing Stephen P Ward C.Eng, MICE, M.I.Mgt MIExp E. (January, 2002)

CINTEC ANCHOR TESTING

London Borough of Brent
Gas Explosion Anchors

TESTING BY:

OVE ARUP AND PARTNERS
(JULY 1987)

adjusted for bedding down of the jack legs. It is essentially linear once the initial deflections have been passed. It follows the form of the load deflection curve of anchor 1 but with increased deflection values at the same load values. An ultimate strength of 9 tonnes was achieved, but the jack could not sustain 9.5 tonnes load.

After removal of the load from the anchors in each test, the threaded bolt was found to have play at the anchor end. This suggests that the connection between bolt and anchor was the cause of failure of the second anchor.

IN SUMMARY

1. The 30 x 30 Cintec anchors sustained an ultimate load of at least 9 tonnes.
2. The load/deflection curves adjusted for bedding down were essentially linear up to 6.0 tonnes.
3. The deflection of anchors 1 and 2 at 6 tonnes, were 1.36mm and 1.54mm, respectively. The first anchor with plug in the void naturally had the lower deflections.

Job Title BRENT LONDON PULL-OUT TEST	Date of test 18th JUNE, 1987
Company Name OVE ARUP & PARTNERS	Location of test BRENT
	Test carried out by :

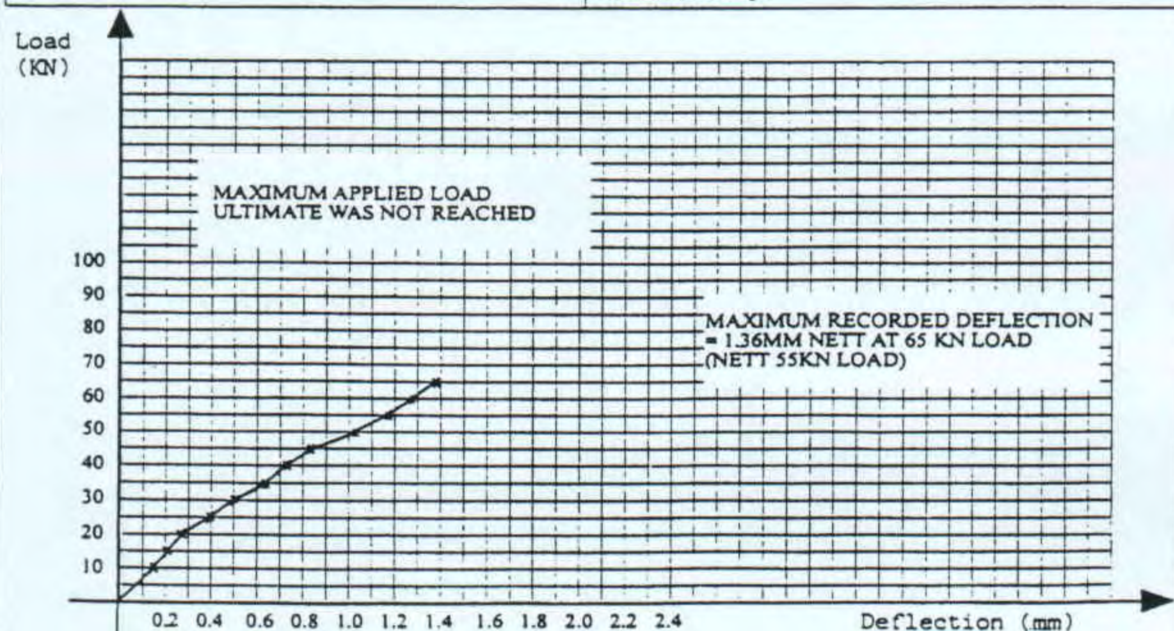


FIG 1.0 : LOAD/DEFLECTION CURVE – ANCHORE LENGTH 600mm WITH PLUG IN VOID

Cavity Lock Systems Limited - Test Data Reports

BRENT - GAS EXPLOSION ANCHOR TESTS
LONDON BOROUGH OF BRENT,
LONDON.

OVE ARUP & PARTNERS CAMBRIAN BUILDINGS
MOUNT STUART SQUARE CARDIFF
SOUTH GLAMORGAN
CF1 6QP
JULY 1987

GAS EXPLOSION ANCHOR TESTS.

Job Title BRENT LONDON PULL-OUT TEST	Date of test 18th JUNE, 1987
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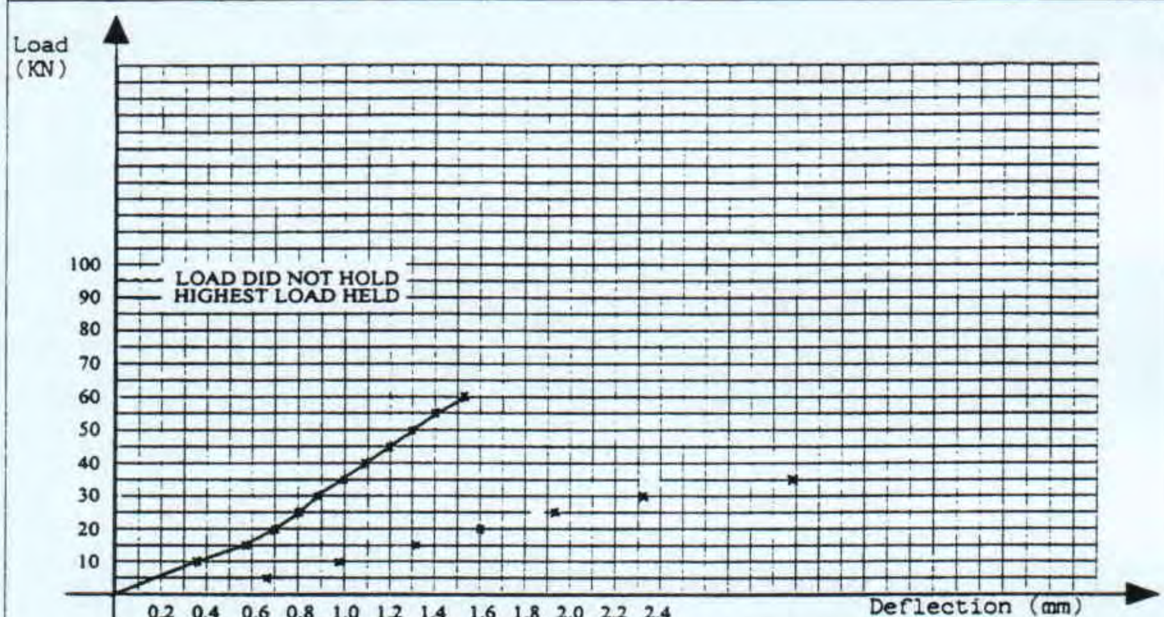
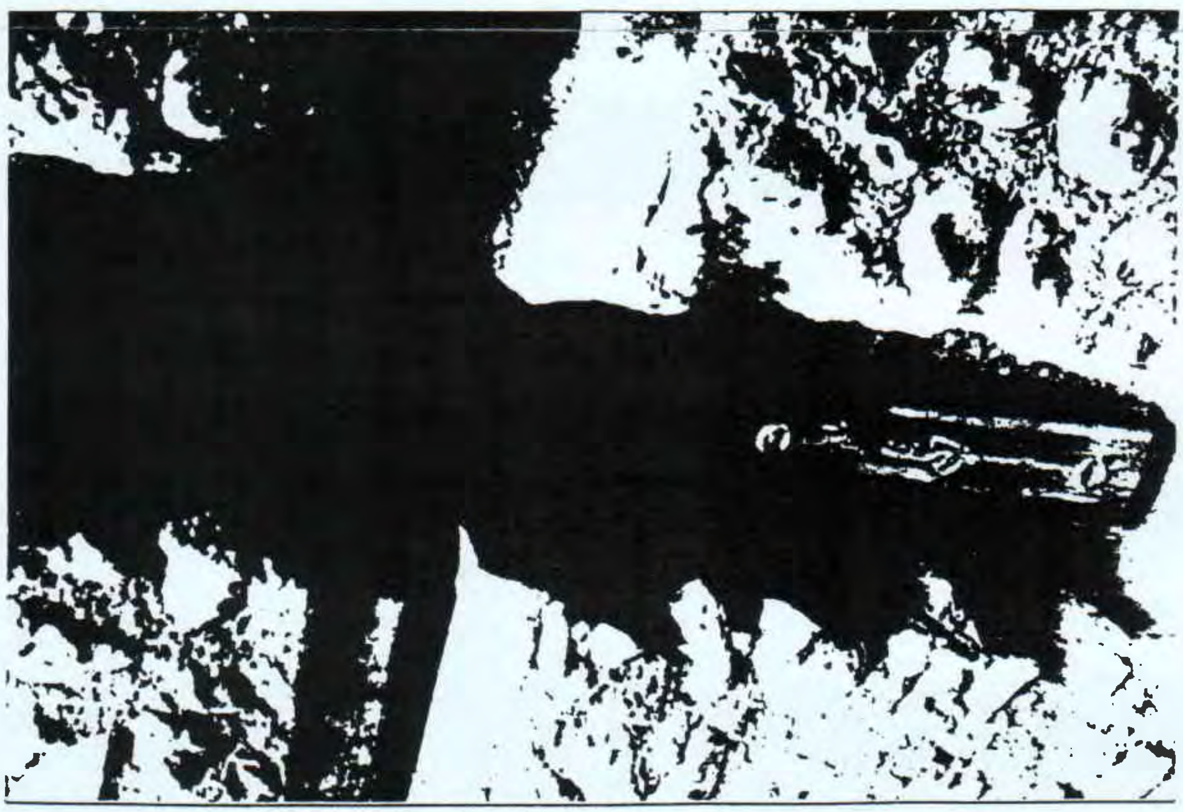


FIG 2.0 : LOAD /DEFLECTION CURVE – ANCHOR LENGTH 500mm
400mm INTO CONCRETE 100mm PROTRUDING AT FRONT



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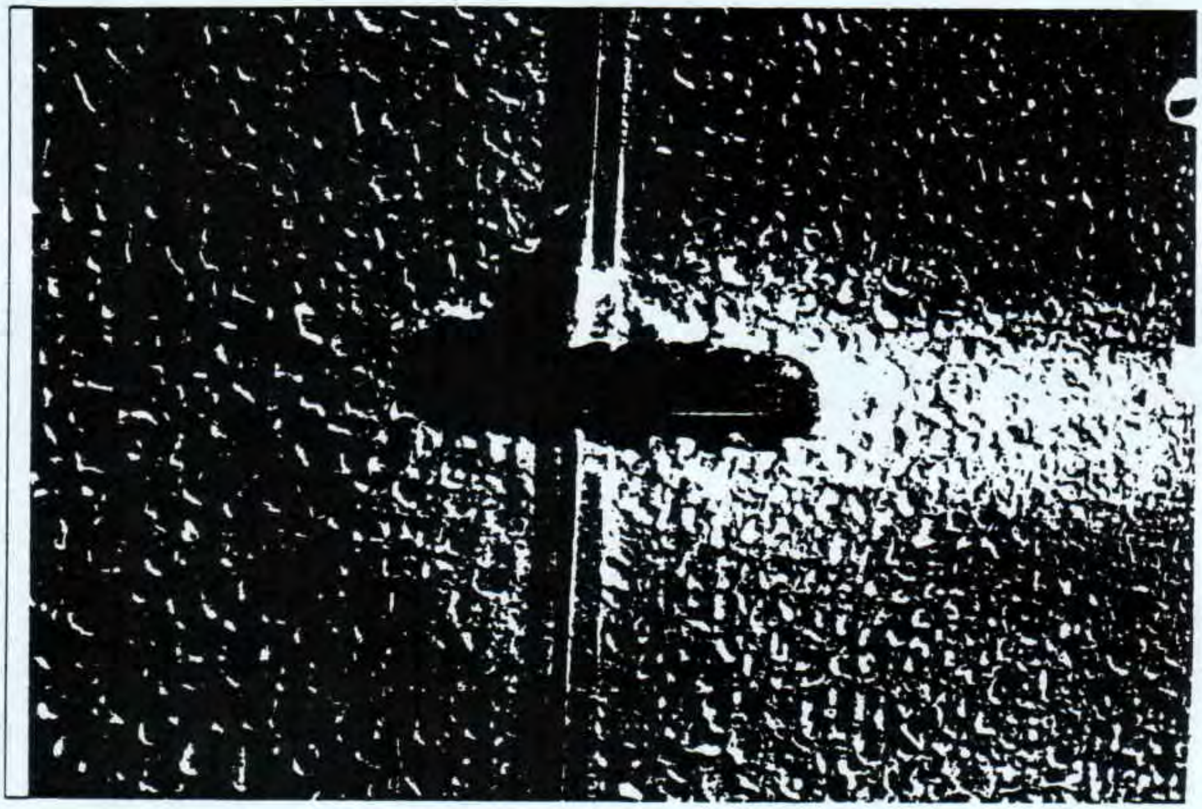

HOUSES AND APARTMENTS, BASILDON

The Commission of New Town wished to enhance the robustness of 400 No. houses and 200 No. 3-4 storey apartments, particularly against the effects of accidental damage. The Cintec anchor was adopted primarily on the basis of cost-effectiveness and least disturbance to the tenants.

Enhanced robustness was achieved by the installation of stitching anchors 6m long tying the front and rear elevations. It required the development of dry drilling techniques and carefully co-ordinated management so that tenants were only required to absent from their properties for one day between 8.00am and 6.00pm. The anchors passed through concrete hollow floors with careful control of level. Special socks and grout pressures were designed for the particular application. Particular attention was paid to keeping tenants informed and to meeting their individual requirements. As a result all the work was completed within cost and programme to the satisfaction of the clients and tenants.

Section through hollow floor

Technical
Consultancy

Structural Performance Division

Onsite Structural Testing and Assessment of the CINTEC Anchor Reinforcement System at Basildon, Essex.

By R J Currie
February 1993

The Building Research Establishment
Bucknalls Lane, Garston, Watford.
Hertfordshire. WD2 7JR.

Technical Consultancy

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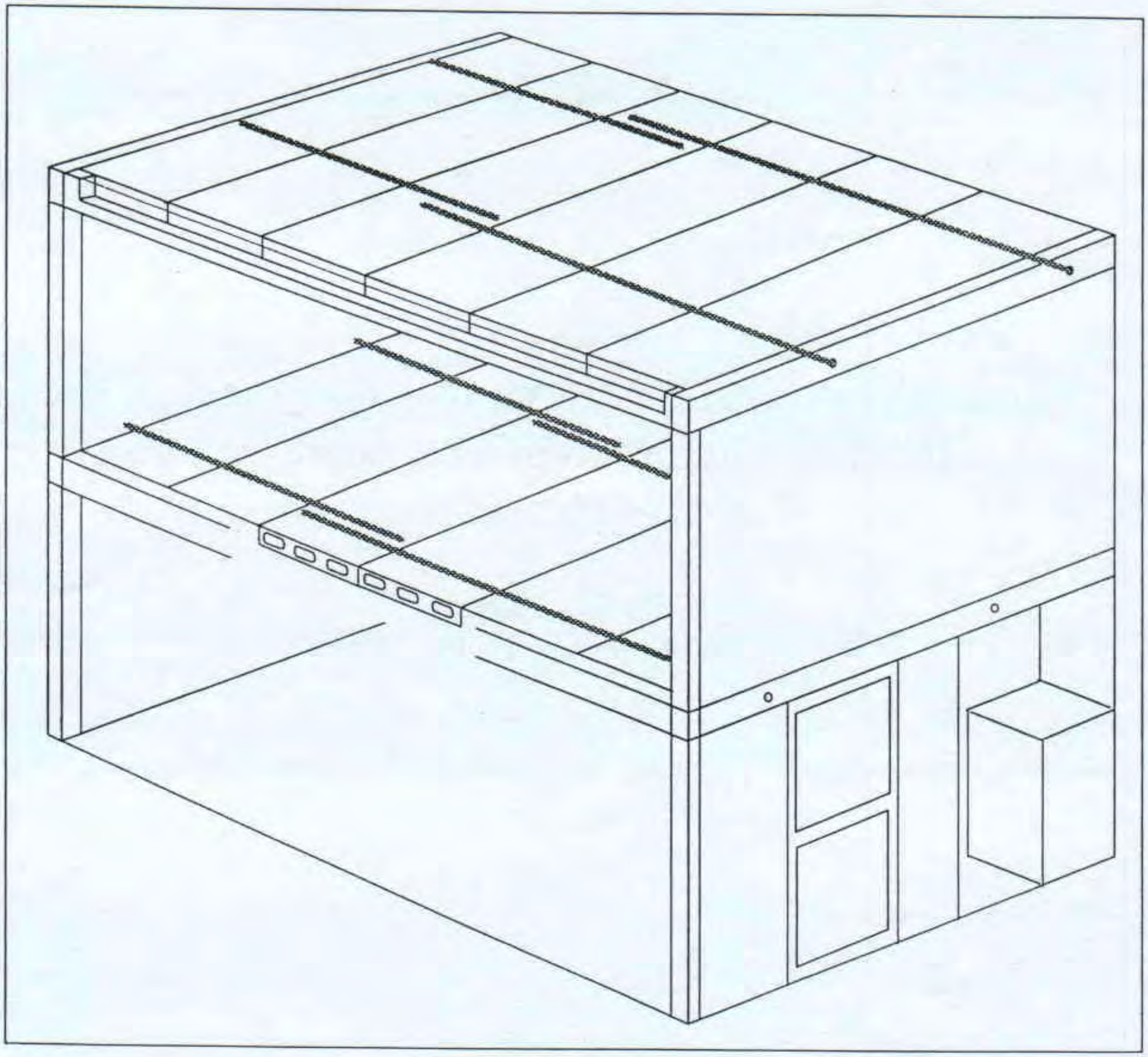
Onsite Structural Testing and Assessment of a CINTEC Anchor Installation at Basildon, Essex.

By R. J. Currie



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Bucknalls Lane
Garston, Watford WD2 7JR

Executive Summary

The strengthening system was developed by Scott, Wilson, Kirkpatrick & Partners using the Cintec anchor system supplied by Cintec International Ltd and was tested by BRE using dynamic and static techniques. The anchor system had been installed in the floors and roofs of an estate of large panel system dwellings at Basildon in Essex to improve the robustness of the construction.

A terrace of three dwellings was made available by the Commission for the New Towns for development and testing and the performance of the floors and roofs in two of the dwellings were assessed with and without the strengthening anchors.

Finally, collapse of the two end dwellings was initiated by removing the external ground floor load bearing walls and the influence of the anchors on the mechanism of collapse was recorded and analysed.

This work was undertaken by BRE as a collaborative project for Scott, Wilson, Kirkpatrick & Partners, the Commission for the New Towns, Building Regulations Division of DOE, the Housing Directorate of DOE and Cintec International Ltd, manufacturers and suppliers of Cintec Anchors.

Onsite Structural Testing and Assessment of a Cintec Anchor Installation at Basildon, Essex.

Background

During the course of an investigation into the condition of High Speed System Build (HSSB) dwellings in Basildon, Scott Wilson Kirkpatrick & Partners (SWK) were asked to assess the vulnerability of these concrete panel buildings to accidental loadings. It was concluded, as a result, that strengthening was required to increase the robustness to be equivalent to that of conventional domestic properties (BS8103: Part 1: 1986 Cl 4.1). An important part of the strengthening system would be to make the floors and roofs, which are constructed from precast concrete planks, act as a monolithic slab or plate. If this could be achieved, forces applied to the ends of the planks from the party walls of the dwellings could be transmitted to the shear walls which lie parallel to the direction of span of the planks and stability could be assured.

To achieve this SWK in collaboration with Cintec International Ltd developed an anchor arrangement which involved forming holes by dry diamond drilling horizontally through the floors and roofs at right angles to the direction of span of the planks and grouting in a Cintec anchor to lock the planks together and provide lateral continuity.

As the system was new and untried a block of three, two storey dwellings was selected for installation trials to establish the practicability of the system.

Testing Conducted by BRE on behalf of SWK

BRE was initially commissioned by SWK to establish if the trial installation of the anchors induced effective lateral connectivity between the planks.

To establish this BRE conducted dynamic tests on two sections of floors in areas A and B (Fig. 1) since this is the most economic and quickest form of full scale testing available.

Before the anchors were installed dynamic impact tests were conducted in rooms in areas A and B to identify the most suitable sections to test. Then a controlled forced vibration test was conducted on floors in areas A and B before the anchors were installed and the same tests repeated after installation of the anchors.

These tests established the following:-

1. The floors in end B were considerably stiffer than in end A, both before and after the anchors were installed. This may have been due to initial curvature induced in the floors in area B as a result of clay heave acting on the walls of the structure.

2. The stiffness of the floors in both areas A and B was increased by the addition of the anchors.

3. Before the installation of the anchors the deflected shape of the floors in both ends A and B was consistent with a system in which little lateral connectivity between planks was present (Fig. 2).

4. With the anchor system installed the deflected shape of the floors in areas A and B was consistent with a system with lateral continuity such as a plate (Fig. 3).

The dynamic tests therefore demonstrated that the anchors induced plate action in the plank floors which supported the presence of the lateral connectivity assumed by SWK in justifying the stability of the dwellings with the anchors installed.

As a result of these tests, and trials conducted by SWK to assess the reliability of the installation method, the whole estate of HSSB housing was strengthened using the anchor system without the need to decant the occupants.

Further Collaborative Test Programme Conducted by BRE

Background

When the strengthening work was completed on the estate an opportunity was available to conduct further testing and research on the anchor system which would enable its effectiveness to be measured right up to failure of the floors and roofs and its influence on the structures' post failure behaviour to be evaluated qualitatively.

The dynamic testing had not used all the resources set aside for testing the effectiveness of the strengthening system and as a result the Commission for the New Towns, who have responsibility for the dwellings, gave permission and support for SWK to conduct some additional testing. This was to be undertaken as part of a demolition scheme for the three dwellings which were no longer required.

SWK approached BRE with proposals with a view to commissioning a test programme.

On consideration of the SWK proposals it was clear that although BRE could devise and execute a test programme to provide the assessment an opportunity existed to demonstrate a much wider capability of the anchor system. If this proved successful it would provide building owners with an effective additional method of strengthening against loading conditions such as gas explosions, progressive collapse etc., in high rise construction.

In view of this potential the DOE (Building Regulation Division and Housing Directorates) agreed to fund some additional testing to investigate the possible wider applications. With the additional DOE funding, the SWK commission and a contribution from Cintec International Ltd, BRE put forward a proposal for a range of tests to evaluate the influence of the anchors on the structural behaviour of the dwellings from the elastic range through to collapse.

Summary of Test Programme and Findings

Series 1 - Static Load Testing

There were two motives for conducting these tests. The first was to establish if the plate action induced by the anchors, indicated earlier by the dynamic testing was maintained beyond full service loads and the second was to provide additional information to complement a large programme of full scale floor testing previously conducted by BRE.

A number of load tests were conducted on floors A and B with the anchors in place which investigated; lateral load transfer under different concentrations of load, the deflected forms of the individual planks and the transverse deflected profile of the floors.

The anchors in floors in area B were then cut and the load tests repeated on the same areas. Although the absolute deflections were still very small under these serviceability load levels the tests showed that the deflections of the planks increased when the anchors were cut and the floors reverted to a predominantly one way spanning system.

Therefore these static tests demonstrated that (a) two way spanning action was maintained up to full service loads and (b) confirmed that by cutting the anchors the floors reverted to a predominantly one way spanning system in accordance with conclusions reached from the dynamic testing.

Series 2 - Lateral Plate Action

This test was designed to check the assumption that the anchors produce plate action by connecting individual precast planks so that they may transfer "in plane" forces to the parallel shear walls.

This load testing was conducted on both ends A (anchors intact) and B (anchors cut) on the roof components with the waterproofing screed having first been cut along the lines of the plank junctions to avoid the complications of composite action with the roof finishes.

The test method employed was to apply a direct tensile force to the ends of one of the planks (Fig. 4) near the centre of the roof area and to monitor the deflected profile along the edge of the roof.

If the planks were effectively connected the end profile would be in the form of a smooth curve with evenly distributed deflections from a maximum at the point of load application to a minimum adjacent to the side walls. If unconnected, one would expect most of the movement to be concentrated at the plank subject to load application with a much poorer distribution of movement across this width of the roof.

Test 1 - End A (anchors intact).

Using the edge beams as reaction points for a deep beam a tensile load was applied incrementally to one of the central planks and the movement of all the floor planks recorded. When the applied load reached approximately 16 tonnes (T) the lateral movement of the loaded plank was 4.4mm and the profile across the end of the roof was as indicated in Fig. 5 position 1.

At this point the whole roof rapidly moved bodily on its supports a few millimetres and the load was therefore released.

Inspection revealed that the bonding between the roof and the walls underneath had been broken which allowed the central portion of the roof to move laterally relative to edge beams due to the 16 T.

To see if the integrity of the ties had been maintained under this shock loading and to examine if it was mainly frictional resistance between the roof and its supports which had been overcome when the roof moved it was decided to re-test the roof.

The position in which the roof planks came to rest is given in Fig. 5 position 2 which was the new datum position of the roof for the start of the second load cycle. This shows that approximately half the deflections recorded previously were recovered when the load was removed with a permanent 'set' of about 3mm between the centre and edge planks.

When the load was applied again it was clear that the roof was behaving in a similar way to the first load cycle with deformations being shared across the roof width.

The load was increased up to the previously recorded maximum of 16 T and the deformation profile was recorded Fig. 5 position 3. Position 4 (Fig. 5) shows the measurements in position 3 adjusted to allow for the initial datum position at the outset of the second load test.

As the load was increased to 16.4T, one of the threads of the rods used to apply the tensile load failed releasing the load instantaneously and the test on end A was therefore brought to an end.

Test 2 - End B (anchors cut)

Exactly the same procedure of loading and deformation recording was carried out on the roof at End B. When load was applied it was immediately apparent that greater movements than those recorded at end A were occurring.

When the maximum deflection at End B was similar to the maximum deflection recorded under the 16 T load in the previous test on End A the load was held constant. At this point the load applied to End B was 5 tons and the deformed profile at this load is given in Fig. 6 position 1.

When the loading was increased to 6 T the deformation increased dramatically as indicated in Fig. 6 position 2. At this point the load was released and the structure inspected. No recovery of the deformations occurred on removal of the load, the deformed profile being maintained under zero load.

There was no visual evidence of the floor moving on its supporting walls and to confirm the deformations being recorded by the instrumentation, a line was drawn on the underside of the roof to record the relative movements between the planks.

The 6 T load was reapplied and the loaded plank continued to pull out of the floor at this constant load. The test was terminated when the maximum lateral deflection reached was 18.4mm and the final deformed profile under the 6 T load is given in Fig. 6 position 3.

Summary of Findings from Series 2 Tests

End A - (anchors in place)

1. The deformed profile of the ends of the planks at End A (anchors in place) showed that throughout the loading cycle, movement was fairly evenly transferred from the loaded plank to the adjacent planks. This behaviour was maintained up to a load level of 16 T the limiting capacity being due in the first test to the friction between the roof and its supports and in the second test the capacity of the test rig.

2. At the maximum load of 16 T approximately half of the deformation present was recoverable and deformation under the repeated 16 T load was similar to those recorded during the first load cycle.

End B - (anchors cut)

1. The deformation at End B corresponding to the maximum deformation recorded at End A occurred under approximately one third of the load applied to End A.

2. From the beginning of the load cycle the loaded plank moved out from the adjacent planks and the limited movement of the other planks was heavily skewed to one side of the roof.

3. The movement of the loaded plank due to the 6 T load was non-elastic and continued to 18.4mm when the test was terminated.

4. The limitation on the capacity to resist and transmit inplane loads was the failure of the loaded plank to act in conjunction with the adjacent planks.

Series 3 - Collapse Modes

Although the anchor installation was not intended to provide any flexural capacity in the transverse direction to the planks and had only been designed to transmit in plane shear forces to the side walls it was thought it would be interesting to see if the mode of collapse of the structure was influenced by the anchor installation.

To do this some controlled demolition was undertaken and recorded using time lapse photography at five frames a second and video recorders.

The same procedures were used on both Ends A and B and were as follows:-

1. - The end ground floor concrete wall panels which support the ends of the first floor planks, the first floor end wall panels and the ends of the roof planks were cut away to leave a minimal support at the centre and the edges of the end wall (Photo 1).

2. The remaining central support section was then carefully removed using an excavator bucket.

3. The remaining side supports were then progressively taken away leaving the end of the first floor unsupported.

4. The ensuing collapse was filmed and recorded.

Findings

The only way to fully appreciate the mechanisms of collapse which were involved is to study the video recordings and the time lapse photographs. For the purpose of this report a number of the time lapse photographs are referred to which illustrate critical points in the collapse mechanisms.

End A - (anchors in place)

Photograph A4 - Shows the stage were virtually all the load bearing support has been removed from the end of the structure. At this point there is no significant disruption to the structure and the first floor planks have just started to deflect under the load.

Photograph A5 - The last remnants of the supports are removed and the first floor starts to fail slowly under the weight of the panels above.

Photograph A8 - The first floor continues to fail as a unit leaving behind the roof which is unsupported but still intact. The side walls start to buckle out at first floor level under rotational forces induced by the first floor wall panels.

Photograph A10 - The first floor collapse is complete, the side walls are now buckling badly but the unsupported roof remains in place and intact.

Photograph A12 - There is complete loss of stability of the side walls due to rotation at first floor level forcing down the roof structure as a unit.

Photograph A14 - Collapse of the complete end of the structure. Note the upper storey side walls being pulled inwards and roof now in free fall but still continuous.

Photograph A35 - Final condition of collapsed structure. Note most of the debris lies within the original dimensions of the building.

End B - (anchors cut)

Photograph B5 - This shows a stage where the supporting wall is being removed. It can be seen that the first floor has slipped down at the side nearest the camera and the end first floor wall panels and the side walls have started to rotate.

Photograph B6 - The first floor is failing rapidly as the excavator bucket attempts to remove the remaining supports and the side walls have commenced buckling at first floor level.

Photograph B7 - As the failure progresses rapidly the roof breaks up and fails following the first floor down. Note the first floor centre wall panel is sandwiched between the roof and floor and the end is falling as a whole.

Photograph B8 - The roof continues to follow the first floor in free fall.

Photographs B9/11 - Complete collapse of the end of the building.

Photograph B36 - Fragmented side walls had been contained by the safety scaffolding.

Summary of Findings

Apart from the rate of collapse being far greater at End B, there are a number of key differences in the sequence of events between the failure of Ends A and B these are:-

1. The roof structure at End A remained intact spanning laterally the width of the building throughout the collapse until the supporting side walls became unstable due mainly to impact from debris loading. Even at this point the roof collapsed as a continuous unit.

2. The roof structure at End B fragmented at an early stage of the test and followed immediately the failure of the first floor. Both roof and floor came down in free fall together.

3. The side walls at End A only buckled significantly when the first floor failure was complete and debris loading from the wall exerted its maximum lateral thrust. (Photo A12).

   At End B a similar degree of buckling to the side walls was present by the time the first floor had moved down approximately 300mm (Photo B6).

4. The lateral spread of debris was greater at End B than at End A and the construction immediately behind the collapsed portion of End B was vertically cracked indicating that lateral load had been transmitted back through the structure.

Discussion

In a progressive collapse energy is continually fed into the event and failure is transmitted through the structure via the connections. An important option in preventing local collapse progressing disproportionately is to ensure that locally damaged areas can be bridged over by the remaining structure and the resulting loads can be transmitted safely through alternative load paths.

Although the anchor system was not designed to improve post failure behaviour nor had their locations been chosen with this in mind, the test demonstrated two significant improvements in the post failure performance of the end strengthened with the anchors.

1. The roof structure was able to bridge the width of the building when its support had been removed and therefore its weight did not contribute to the energy input to the failure at first floor level.

2. The lateral tying delayed the buckling failure of the side walls until the impact of the debris loading from first floor forced the lower panels outwards.

   At End B the buckling failure of the side walls occurred almost at the same time as the first floor failure started.

Conclusion

1. The anchors clearly have the potential to modify the mode of failure of a structure and therefore offer a valuable tool to structural engineers in designing against the collapse mechanisms in existing structures.

2. Correctly designed and installed this form of anchor can effectively induce a high degree of plate action in floors and roofs comprising of individual hollow precast concrete components, enabling them to span laterally.

3. The introduction of plate action can stiffen existing precast floors and roofs.

4. The ability of the strengthened floors and roofs to transmit vertical and inplane forces through plate action was maintained, even in the presence of extreme deformations.

5. Repeat loading did not indicate any degredation in the anchors performance.

6. The ability of the roof system to transmit horizontal loads to the front and rear edge beams was increased at least $2^3/_4$ times by the addition of the anchors; the limiting factor during the test being the capacity of the test equipment.

7. Ultimately the total collapse of the end dwellings was initiated by lateral instability of the side walls. In the dwelling with the anchors cut this instability occurred almost immediately the first floor lost support, but only occurred when the first floor had completely collapsed in the dwelling where the anchors were intact.

8. The strengthening system does not introduce additional stresses into the structure and requires small deformations before it becomes active.

9. The structural model indicated by the dynamic testing was confirmed by the subsequent static load tests.

CRITICAL STAGES IN THE COLLAPSE OF DWELLINGS AT

END A

A4 Shows the stage were virtually all the load bearing support has been removed from the end of the structure. At this point there is no significant disruption to the structure and the first floor planks have just startedto deflect under the load.

A5 The last remnants of the supports are removed and the first floor starts to fail slowly under the weight of the panels above.

A8 The first floor continues to fail as a unit leaving behind the roof which is unsupported but still intact. The side walls start to buckle out at first floor level under rotational forces induced by the first floor wall panels.

A10 The first floor collapse is complete, the side walls are now buckling badly but the unsupported roof remains in place and intact.

A12 There is complete loss of stability of the side walls due to rotation at first floor level forcing down the roof structure as a unit.

A14 Collapse of the complete end of the structure. Note the upper storey side walls being pulled inwards and roof now in free fall but still continuous.

A35 Final condition of collapsed structure. Note most of the debris lies within the original dimensions of the building.

CRITICAL STAGES IN THE COLLAPSE OF DWELLINGS AT END B

B5 This shows a stage where the support wall is being removed. It can be seen that the first floor has slipped down at the side nearest the camera and the end first floor wall panels and the side walls have started to rotate.

B6 The first floor is failing rapidly as the excavator bucket attempts to remove the remaining supports and the side walls have commenced buckling at first floor level.

B7 As the failure progresses rapidly the roof breaks up and fails following the first floor down. Note the first floor centre wall panel is sandwiched between the roof and floor and the end is falling as a whole.

B8 The roof continues to follow the first floor in free fall.

B9/11 Complete collapse of the end of the building.

B36 Fragmented side walls had been contained by the safety scaffolding.

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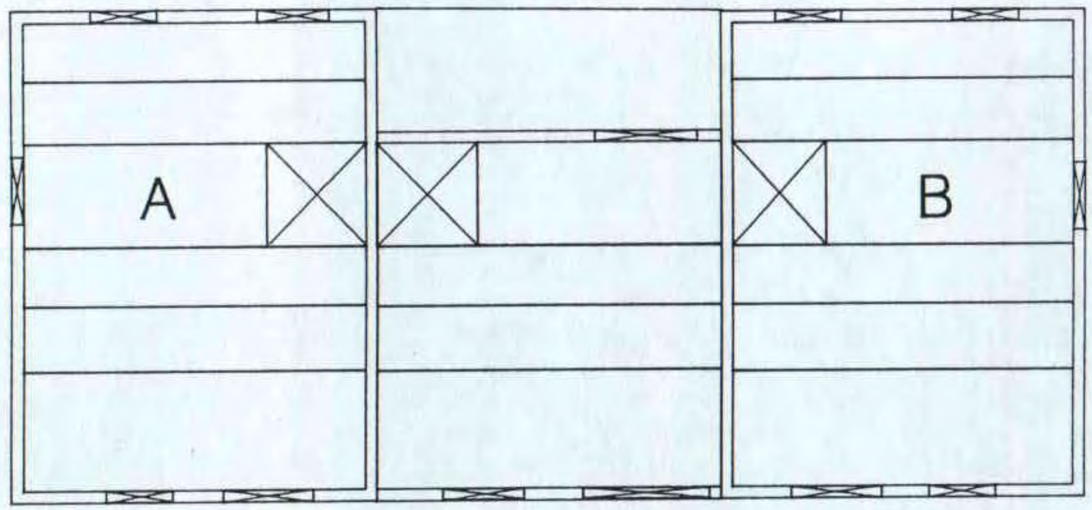
<u>Photograph 1</u>

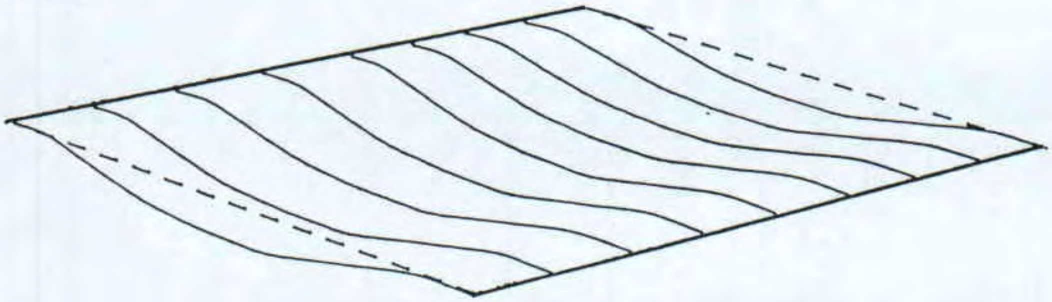
Reduced support prior to removal of ground floor wall.

Fig. 1



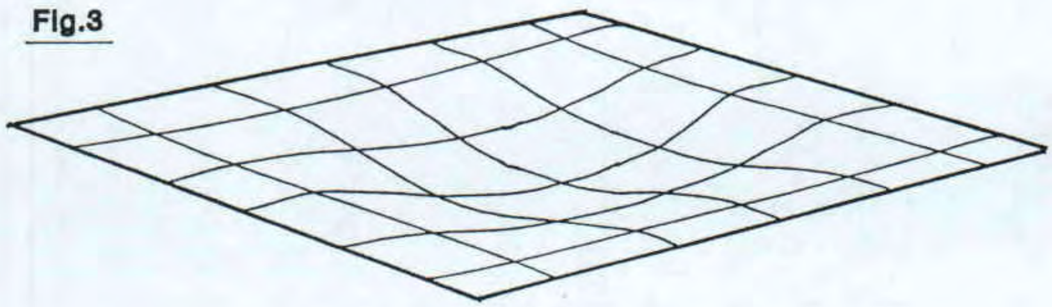
Plan at first floor level of terrace of 3 dwellings.

Fig.2



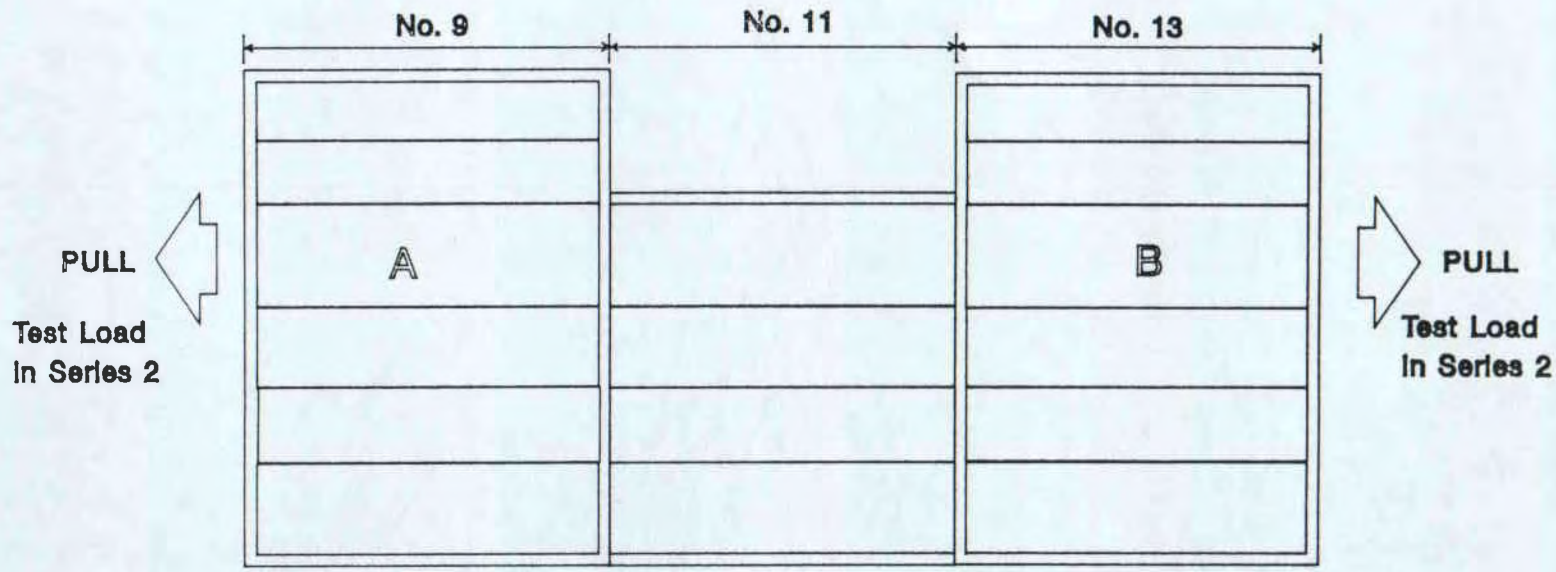
Floor spanning in one direction.

Fig.3



Floor spanning in two directions.

Fig. 4



Plan at roof level of terrace of 3 dwellings

Fig. 5

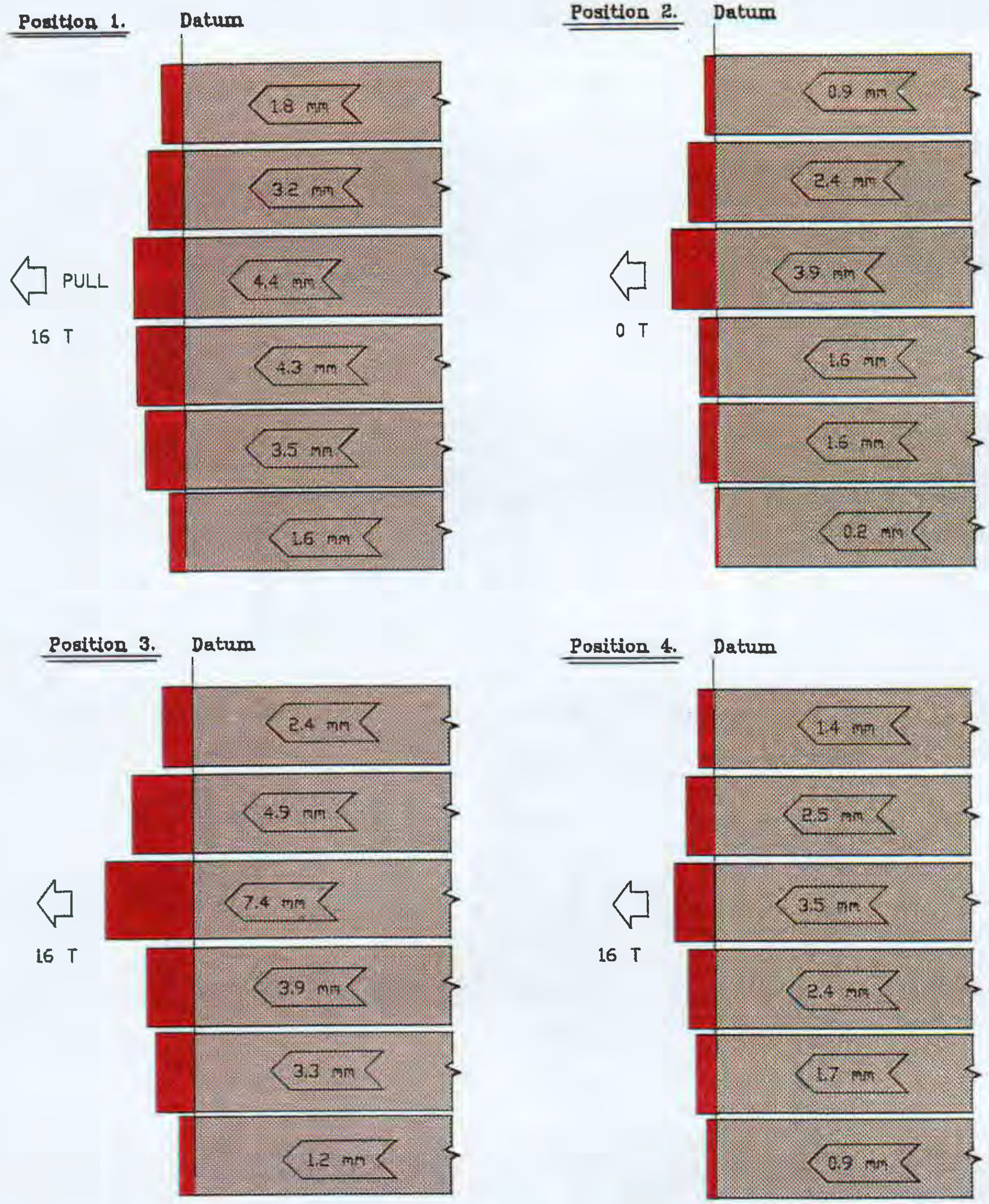
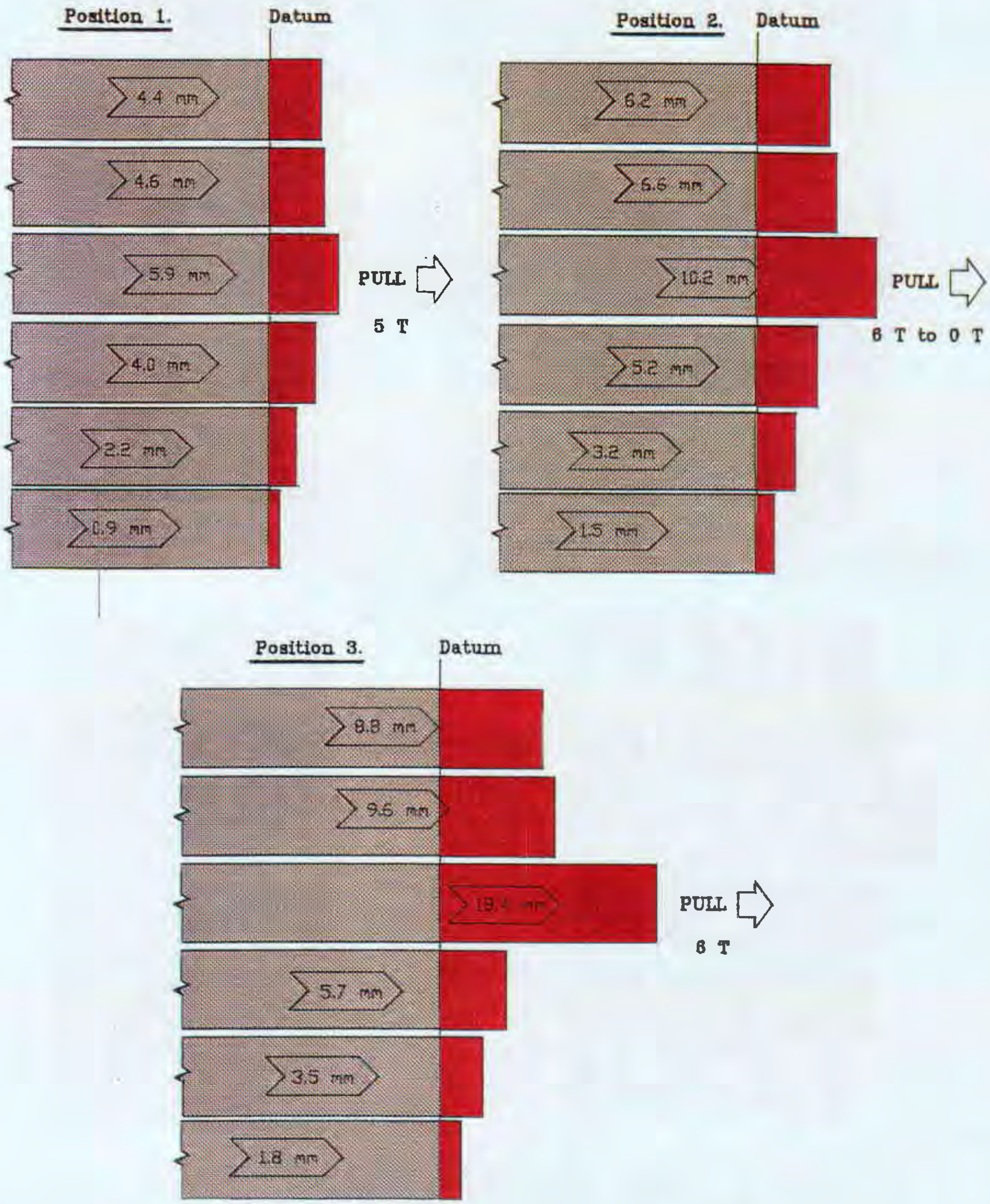

Position 1.
Datum
1.8 mm
3.2 mm
4.4 mm
4.3 mm
3.5 mm
1.6 mm
PULL
16 T
Position 2.
Datum
0.9 mm
2.4 mm
3.9 mm
1.6 mm
1.6 mm
0.2 mm
0 T
Position 3.
Datum
2.4 mm
4.9 mm
7.4 mm
3.9 mm
3.3 mm
1.2 mm
16 T
Position 4.
Datum
1.4 mm
2.5 mm
3.5 mm
2.4 mm
1.7 mm
0.9 mm
16 T

Fig. 6



Position 1.
Datum
4.4 mm
4.6 mm
5.9 mm
4.0 mm
2.2 mm
0.9 mm
PULL
5 T
Position 2.
Datum
6.2 mm
6.6 mm
10.2 mm
5.2 mm
3.2 mm
1.5 mm
PULL
6 T to 0 T
Position 3.
Datum
8.8 mm
9.6 mm
19.4 mm
5.7 mm
3.5 mm
1.8 mm
PULL
6 T

Commercial Blast Testing – ComBlast 2001

Post Test Report

Cintec Wall Targets

Spadeadam, Cumbria – 5 June 2001

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Annex A - Structural Details of Test Targets

A REPORT ON THE CINTEC RETRO REINFORCEMENT SYSTEM TO RESIST THE EFFECTS OF BLAST LOADS ON MASONRY STRUCTURES

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Division Manager, Cintec International Ltd

Abstract

This paper reports on the computer simulation and testing undertaken as part of a full-scale blast load test on masonry wall panels strengthened using a proprietary reinforcement system. Two masonry wall panels were constructed within structural steel frames and retro-fitted with masonry reinforcement anchors before being subjected to a blast load from 200kg of high explosive detonated at distances of between 12.5-15 metres. In preparation for the test, a number of finite/discrete element computer simulations were studied in order to evaluate the most efficient retro-reinforcement pattern. Both walls survived the test proving conclusively that the performance of masonry wall panels can be accurately simulated and strengthened to resist the effects of large blast loads.

Newport, South Wales January 02

1. <u>Background</u>. Cintec has a long history in repairing and strengthening buildings around the world, many of a historic nature. Since the mid 1980's, Cintec has been involved in a program to strengthen buildings in the event of accidental explosions. Over 32 miles of masonry anchoring has been installed in buildings in the United Kingdom to prevent disproportionate collapse in the event of an accidental gas explosion. Since then, Cintec has developed the system further to protect buildings from the damage caused by bomb and forced entry attacks. The Cintec system is capable of providing blast protection all types of masonry buildings from the historical to the modern including those with masonry panels and CMU or terracotta block walls. Cintec's professional blast engineers specially design the system for each application.

2. <u>Technique.</u> Cintec's technique is to drill a number of holes into the parent structure of varying length and diameter, the location of which is determined by detailed structural analysis. The holes are diamond drilled with or without the use of water depending on the sensitivity of the original structure. Once the holes have been completed, the anchors consisting of a structural steel member surrounded by a sock, are inserted in the hole. The sock is then pumped full of non-shrinking cementitious grout. The sock retains the grout around the steel providing a surface where the grout laitance can adhere to the parent masonry. It also prevents the grout flowing indiscriminately throughout delicate structures creating a retrofitted system of reinforcement within a masonry panel. The anchors have been age tested for 40 years and can be used in weak or friable materials. They have been manufactured over 60m (200') in length and installation is carried out only by approved contractors.

3. <u>Commercial Blast Trails</u>. Last summer, Cintec International Ltd was invited to take part in a series of commercial blast trials. These were known as ComBlast 2001; a joint initiative between the British Government and private sector companies run by the UK Police Scientific Development Branch. The trials took place at the Advantica[1] test site at Spadeadam, Cumbria, UK during the week 4-8 June 2001. A number of area tests were conducted with charges of 100kg and 200kg TNT NEQ[2]. Cintec agreed to take part particularly as this series of tests represented a valuable opportunity to test the Cintec retrofitted masonry wall reinforcement system against high blast loads.

4. <u>Previous Tests</u>. During the previous two years, Cintec International Ltd has been perfecting a system of masonry wall reinforcement with a view to protecting existing masonry walls from damage caused by high blast loads. The system is known commercially as *Blastec* and is capable of providing blast protection to all types of masonry buildings from the historical to the modern, including those with masonry panels and CMU[3] or terracotta block walls. A number of field trials have been successfully completed at COTEC[4]. These trials included the retrofit installation of reinforcing anchors and subsequent blast load testing of traditional solid and cavity masonry walls, hollow concrete block (CMU) and walls with window apertures – see Figure 1. Due to range restrictions, charge weights have had to be lower than those likely to be encountered in a terrorist or accidental incident. To compensate

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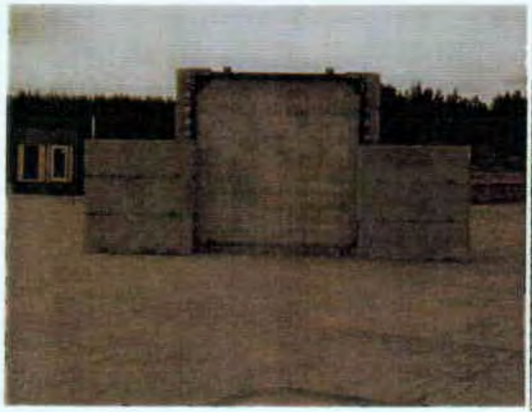
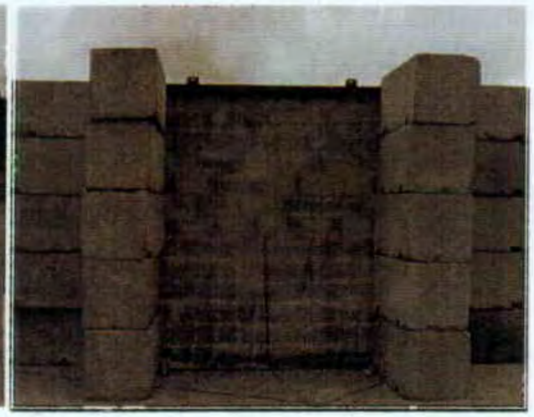
[1] Advantica – formerly BG Technology, the research division of British Gas.
[2] 100kg = 220lbs, 200kg = 440lbs; TNT - Trinitrotoluene; NEQ – Net Explosive Quantity; explosive used – nitro methane.
[3] CMU – concrete masonry unit.
[4] COTEC - Cranfield University Ordnance Testing and Evaluation Centre at West Lavington Down, Wiltshire, UK

for this the stand off distances were reduced dramatically to preserve the high impulse energies associated with this type of event. Although the blast load wave fronts were not truly planar, the trials demonstrated that relatively weak and friable masonry walls, with and without windows, could be successfully strengthened to resists the damaging effects of a blast attack.

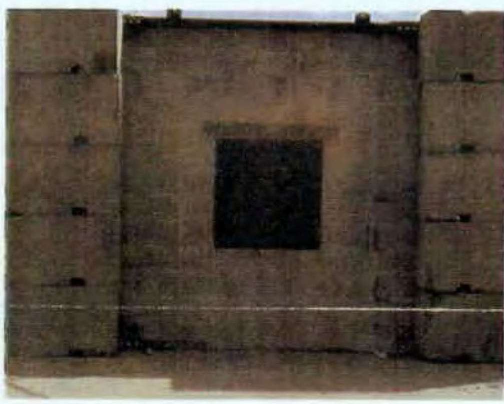
Fig. 1 – Post test results of the first Blastec Wall Trial – 8 Nov 99. Two identical 225mm (9") thick masonry walls each subjected to a blast load of 7.8kg (18.1lbs) TNT NEQ at 1m (3.25'). The wall on the left was unreinforced; the other reinforced by Cintec "Multibar Anchors" at 225mm (9") vertical centres.

5. <u>ComBlast 2001</u>. For the tests at Spadeadam, two similar 150mm (6") thick hollow block or CMU walls were constructed at the Advantica test site in April 2001. Each wall measured 3m x 3m (9'10" x 9'10") and was retained within a structural steel reaction frame. In preparation for the test, the frames were supported at the front and back by 'Pendine' concrete blocks. The first wall consisted of a plain retrofitted reinforced masonry panel; the *Blastec* reinforcing anchors were inserted horizontally through the reaction frame in the centre of each course of blockwork. In the second wall, a 10mm steel plate measuring 1m x 1m (39" x 39") welded to structural steel angles was fixed in position using *Blastec* window anchors. The purpose of this window was to transfer the blast loads into window retention anchors without the added complication of the window breaking. The remainder of the blockwork was reinforced in a manner similar to the plain wall. During the anchor insertion and inflation sequence, each layer of hollow blocks was filled with a cementitious grout identical to that inflating each anchor sock. This effectively created a reinforced concrete wall inside the existing blockwork wall. Structural details of each target appear at Annex A and the walls are shown in Figures 2-5.

6. <u>Test Details</u>. Each wall was subjected to a separate blast load from 200kg (440lbs) of TNT NEQ in an arena test alongside other commercial test targets. In the case of the plain hollow block wall, the charge to target distance was 12.5m (41'). For the window wall, the charge to target distance was increased to 15m (49'2"). In addition to still photography taken before and after each firing, normal video and high-speed video was taken of the rear of each wall during the loading event.

Figs 2 & 3 - Pre-test photographs of the plain reinforced wall before the test. The front view is on the left.




Figs 4 & 5 -- Pre-test photographs of the reinforced window wall before the test. The front view is on the left.

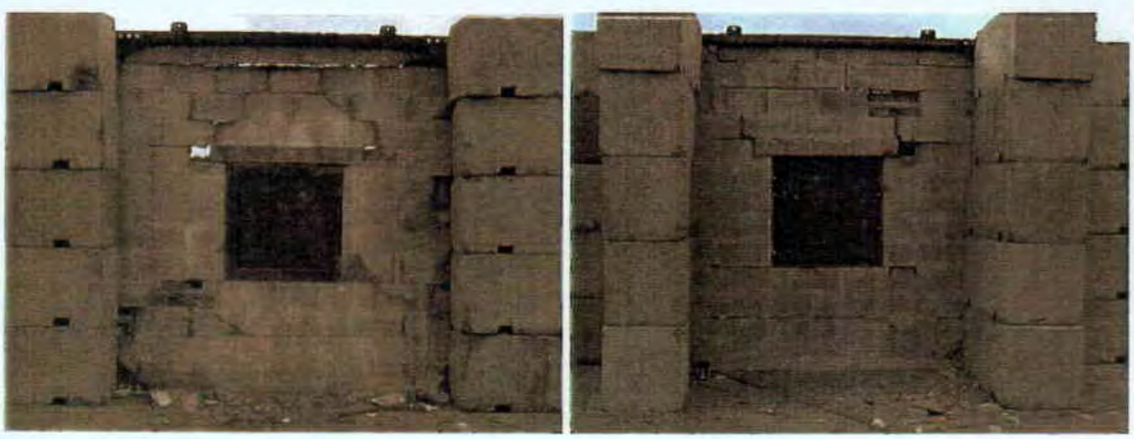
7. <u>Test Results</u>.

a. Each wall survived its test and remained largely intact – see Figures 6-9. The onset of bending failure was clearly evident and that each face was heavily cracked. Although each wall suffered from some spalling, the velocity of the fragments leaving the rear face of each wall was low. There was no evidence of any fragment strike marks on the ground and the maximum debris throw for the larger pieces of spalled material behind each wall was 3.9m. This indicates that the fragments fell from the back of the wall as opposed to being blown off it and would not constitute a hazard to building occupants.

b. The permanent deflection of the front of the plain wall was 320mm, whilst that for the window wall was 310mm leading to a support rotation for both of approximately 12°. It was clear following the test, that a few parts of each wall had not been wholly grouted and that where this occurred, the blockwork showed an increased tendency to fracture and spall. This is an important observation as it demonstrates that the anchors must act compositely with the masonry fabric if they are to remain effective during the loading process. Walls that are particularly brittle, for example terracotta block must be thoroughly grouted and any voids filled if they are to survive[5]

c. Most of the anchor bonds, where they emerged from the reaction frame, survived well although there were a few that had split longitudinally indicating partial shear failure of the 'external' part of the anchor. However, in the case of the window anchors where the 'external part' of the anchor plug was more than twice the standard length, no shear failure was observed. The reaction frames survived well in both tests and can be reused in the future. Damage to the supporting 'Pendine blocks' was minimal

Figs 6 & 7 Post-test photographs of the plain reinforced wall after the test: 200kg TNT NEQ @ 12.5m; 534kPa, 1274kPa-ms (440lbs TNT NEQ @ 41'; 77psi, 185psi-ms). The front view is on the left.



Figs 8 & 9 Post-test photographs of the reinforced window wall after the test: 200kg TNT NEQ @ 15m; 323kPa, 1030kPa-ms (440lbs TNT NEQ @ 41'; 47psi, 149psi-ms). The front view is on the left.

[5] To ensure that the grout fills all parts of the wall, a thermal image survey can be used to detect the remaining holidays or voids

8. Computer Simulation

a. In preparation for the tests at Spadeadam, both types of wall were modelled in 3-D space using a finite/discrete element program provided by Rockfield Software. Loading data was obtained from the Engineering Systems Department, Cranfield University, RMCS Shrivenham using the program Air3d[6]. The data was in the form of a series of pressure time histories taken on a 200mm x 200mm grid over the target face. The representation of the interaction of the blast wave with the masonry structure was accomplished using a semi-coupled approach assuming that the masonry structure was stationary.

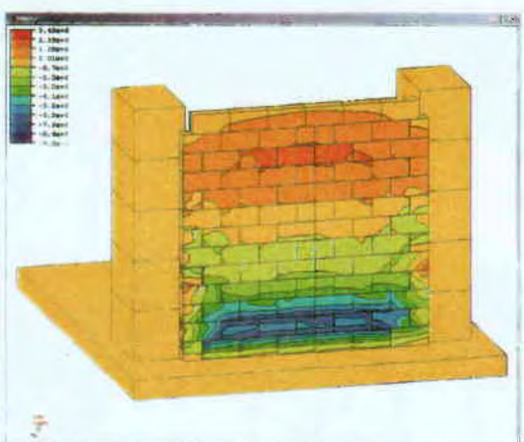
b. The finite/discrete element (f/de) model comprised the masonry wall, a steel reaction frame, concrete support blocks and the reinforcing anchors. Each brick was modelled as a discrete elastic solid, with a mortar interface employed between the brick surfaces to account for the behaviour of the grout. The steel frame utilised an elasto-plastic material model to enable plastic deformation to be captured.

c. In the case of the reinforced wall, non-linear anchor elements were used to represent the *Blastec* anchor reinforcement. These anchor elements account for elasto-plastic behaviour of the steel bars and the stiffness of the anchor grout in the direction along the bars. Gravity loads on the structure were applied first followed by the time-history pressure (obtained from the CFD results) defined for each brick face. This ensures that the initial static load conditions are met before the subsequent pressure loading.

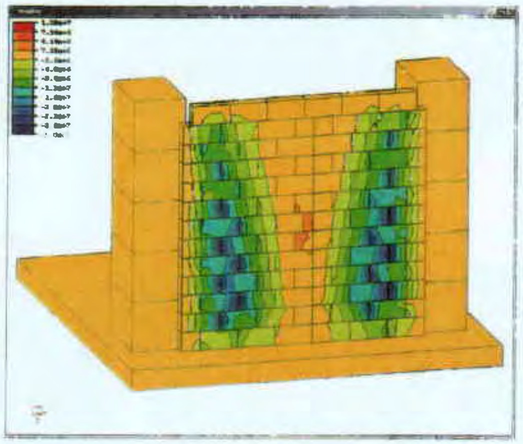
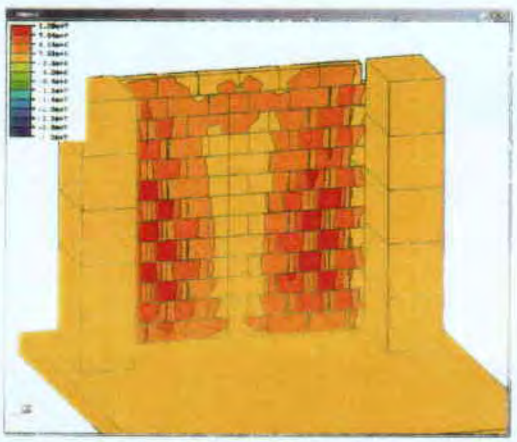
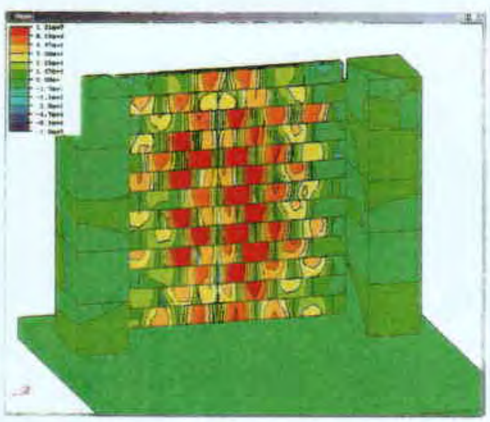
9. Discussion of Results

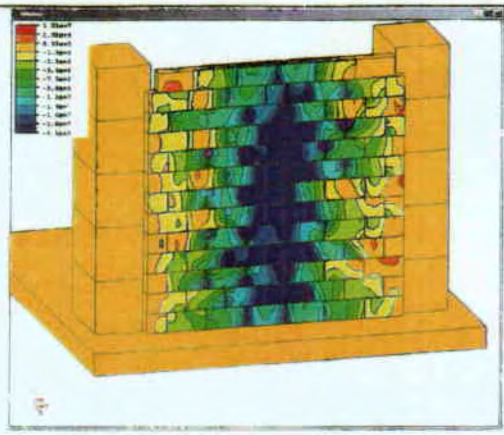
a. The plots of stress distribution on the reinforced wall taken at varying time intervals are shown in Table 1 below.

Table 1. Stress Distributions

	Plot of vertical stress contours at 1.0ms. The stress levels in the compressive zone at the base of wall are high enough to crush the bricks. This zone corresponds to an area on the test wall that exhibited front face brick failure.

[6] Air3d, developed by Dr Tim Rose, is a computational fluid dynamics blast simulation code using a variant advection upstream splitting method (AUSMDV) and MUSCL-Hancock integration producing solutions of the Euler equation that is second order accurate in space and time.

	After approximately 1.5ms, the wall starts to bend at the edges and compressive horizontal stresses form at the vertical brick joints on the front face. This compressive force is also large enough to crush of the brick edges as seen in the tests.
	Also at 1.5ms, a horizontal tensile stress zone develops on the back face, commensurate with the bending action. The maximum tensile stress occurs at the centre of each brick, reducing horizontally towards the mortar joints. The joints have already failed since stresses are excess of the mortar tensile strength
	After 2ms, the bending zones move inwards and meet at the centre of the wall. Significant horizontal stresses on the back face exceed the tensile capacity of the bricks and initiate vertical cracking.
	Also after 2ms, the compressive stress zone meets in the centre of the wall causing significant crushing of the bricks particularly along their vertical edges.



b. **The displacement plots for the unreinforced wall taken at varying time intervals are shown in Table 4 below.**

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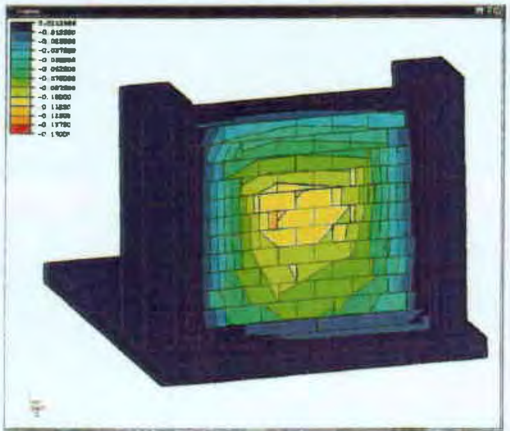
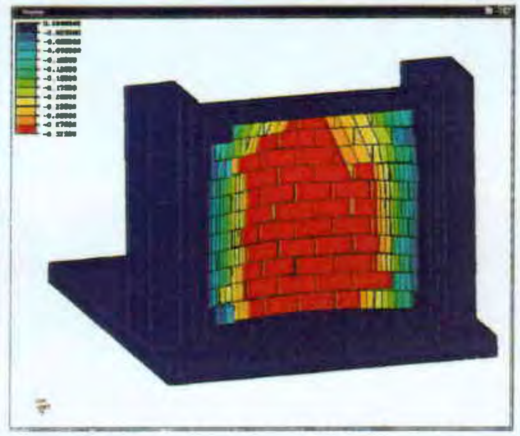
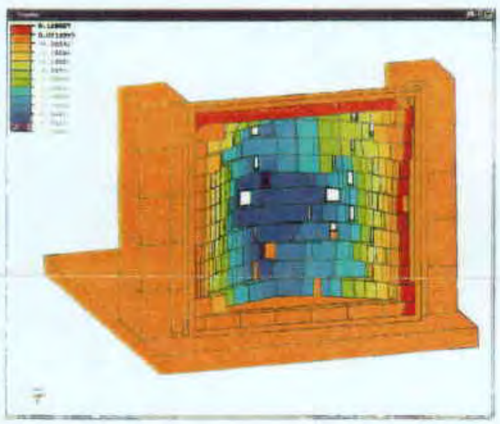
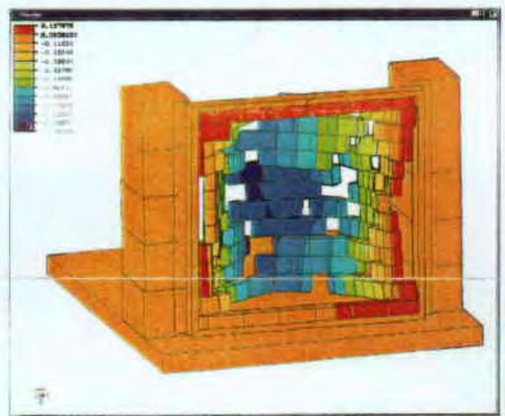
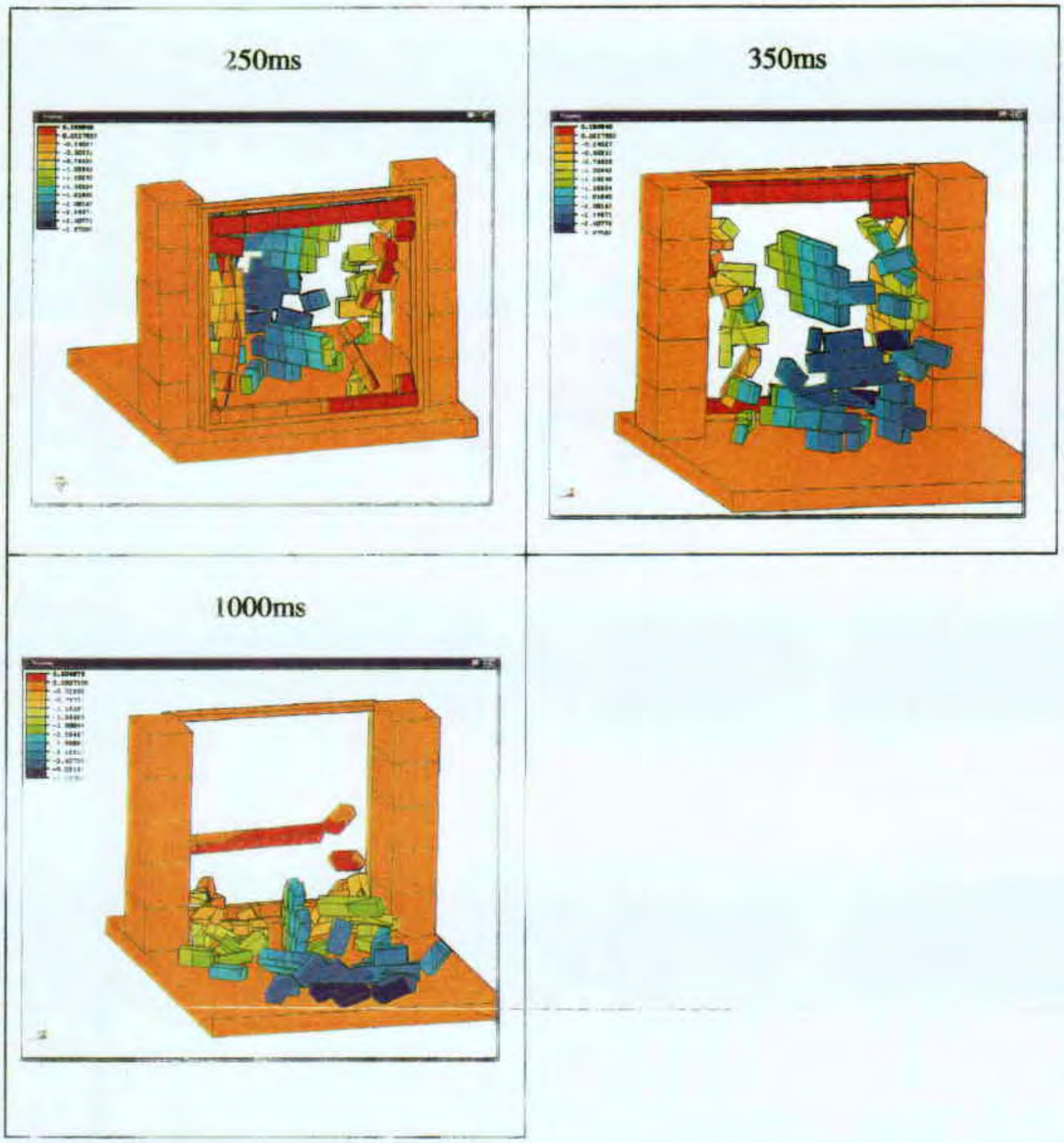
Table 4. Displacement Contours for the Unreinforced Wall

10ms 	40ms 
90ms 	140ms 



10. Conclusions. By combining the discrete element technique with a finite element formulation for reinforcement numerical models have been developed that have allowed rapid evaluation of the relative performance of reinforcement-based retrofitted strengthening. In particular:

a. The overall performance of masonry acting compositely with retrofitted reinforcement can be predicted.

b. A pattern of reinforcement has been investigated that improves the resistance to blast loads and that works equally well for walls with and without openings allowing the comparison of practically viable schemes.

c. The development of strengthening schemes would have been extremely difficult and costly using conventional analysis or testing. However, to be completely confident that the results obtained by these numerical simulations are correct, some further work may be required to verify the simulations against observed behaviour of masonry structures subjected to blast loading.

11. Summary. The use of *Blastec* as a retrofitted protection system significantly enhances the strength of existing hollow block and other types of masonry walls against high levels of blast attack. Structural integrity is maintained and levels of robustness increased. Spalling is minimised to well below the acceptable limits for building occupants. Moreover, the *Blastec* system is flexible and can accommodate a diverse variety of building designs and construction methods without the need to expose the rear of the masonry surface. Blast proof windows, doors and other components can also be successfully retained within the building envelope. Furthermore, using the finite/discrete element model it is now possible to predict accurately the effects of a blast load on a strengthened wall and design the reinforcement pattern accordingly.

Annex:

A. Structural Details of Test Targets.

References:

Cintec International Limited. 2000. *The Cintec anchor system.* Newport, Wales, UK.

Cundall, P.A. 1971. A computer model for simulating progressive, large scale movement in blocky systems. *Proceedings: Symp. ISRM, Nancy, France, Vol. 2, 129-136.*

Engineering and Mechanics Research Corporation 1999. *DISPLAY3 version 9.0 production.* Troy, Michigan, USA.

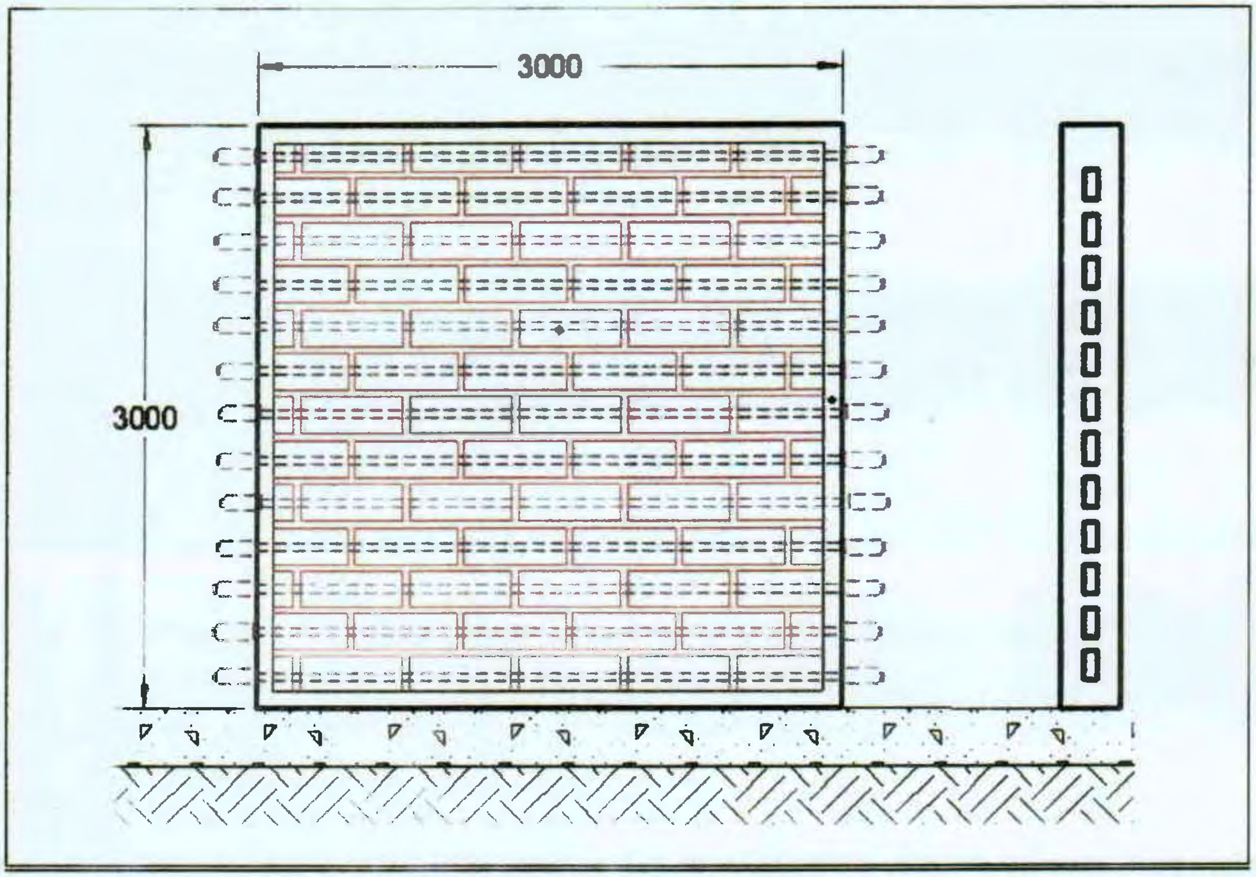
Munjiza A., Owen, D.R.J., Bicanic, N. 1995. A combined finite/discrete element method in transient dynamics of fracturing solids. *Engineering computations, 12, 145-174.*

Rockfield Software Limited 1998. *ELFEN version 2.8.0a_MT Archtec version.* University of Wales Swansea.

Roberts, D.P. 1999. Finite element modelling of rockbolts and reinforcing elements. *PhD Thesis, University of Wales Swansea.*

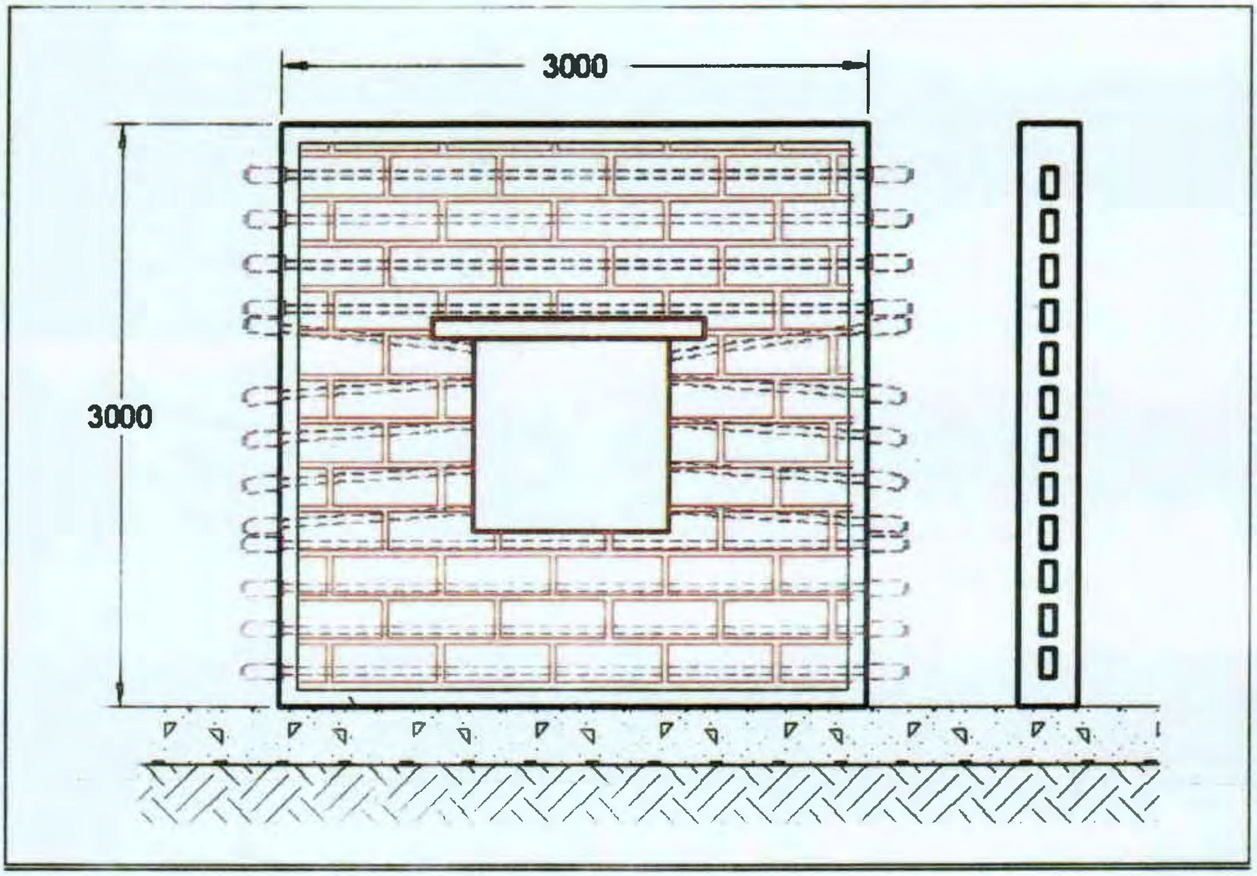
STRUCTURAL DETAILS OF TEST TARGETS

Plain Reinforced CMU Wall



STRUCTURAL DETAILS OF TEST TARGETS

Reinforced CMU Window Wall



Report No. DEOS/AAS/466/99

CINTEC WALL TRIAL

A A SMITH MBE

8th NOVEMBER 1999

Ballistics Group
Department of Environmental and Ordnance Systems
Cranfield University

at

Cranfield University Ordnance Test and Evaluation Centre
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PHOTOGRAPHS	4 to 17
8 GRAPHS OF PRESSURE VERSUS TIME	A - 1 to A - 4

Authorised by	Professor A B Crowley
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Signature	A Crowley
Date	23/11/99

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Telephone 01793-785277

Reference: A. Trials Request dated 30th June 1999.

1. BACKGROUND

There is a requirement to make buildings to stand up to explosive charges. This is one such system that can be retrofitted to existing buildings. The objective of this trial was to prove the difference between a strengthened wall and an unstrengthened wall. This trial was carried out at the Cranfield University Ordnance Test & Evaluation Centre on 8th November 1999, in accordance with Reference A . The Trial Conducting Officer was Mr A.A. Smith MBE. Mr P. James, Mr D. Lee, Mr S. Daves, Mr S. Ward (CINTEC) Mr J. Cain (Rockfield) Mr P. Mullett and Mr C. Bonner (Giffords) attended the trial.

2. OBSERVATIONS

The following observations were recorded:

(a) Video recording of the trial.

(b) High speed video recording of the trial.

(c) Still photographs before, during and after each test.

(d) Blast overpressures.

3. STORES

(a) 38 kg Plastic explosive PE4.

(b) 5 No 8 detonators.

4. EQUIPMENT

(a) Digital video camera AG-EZ1 serial No. K6HY00186.

(b) High Speed Video Recorder Serial No 117.

(c) 35mm camera.

(d) Kistler type 603B pressure gauges serial No's 65608 and 442944.

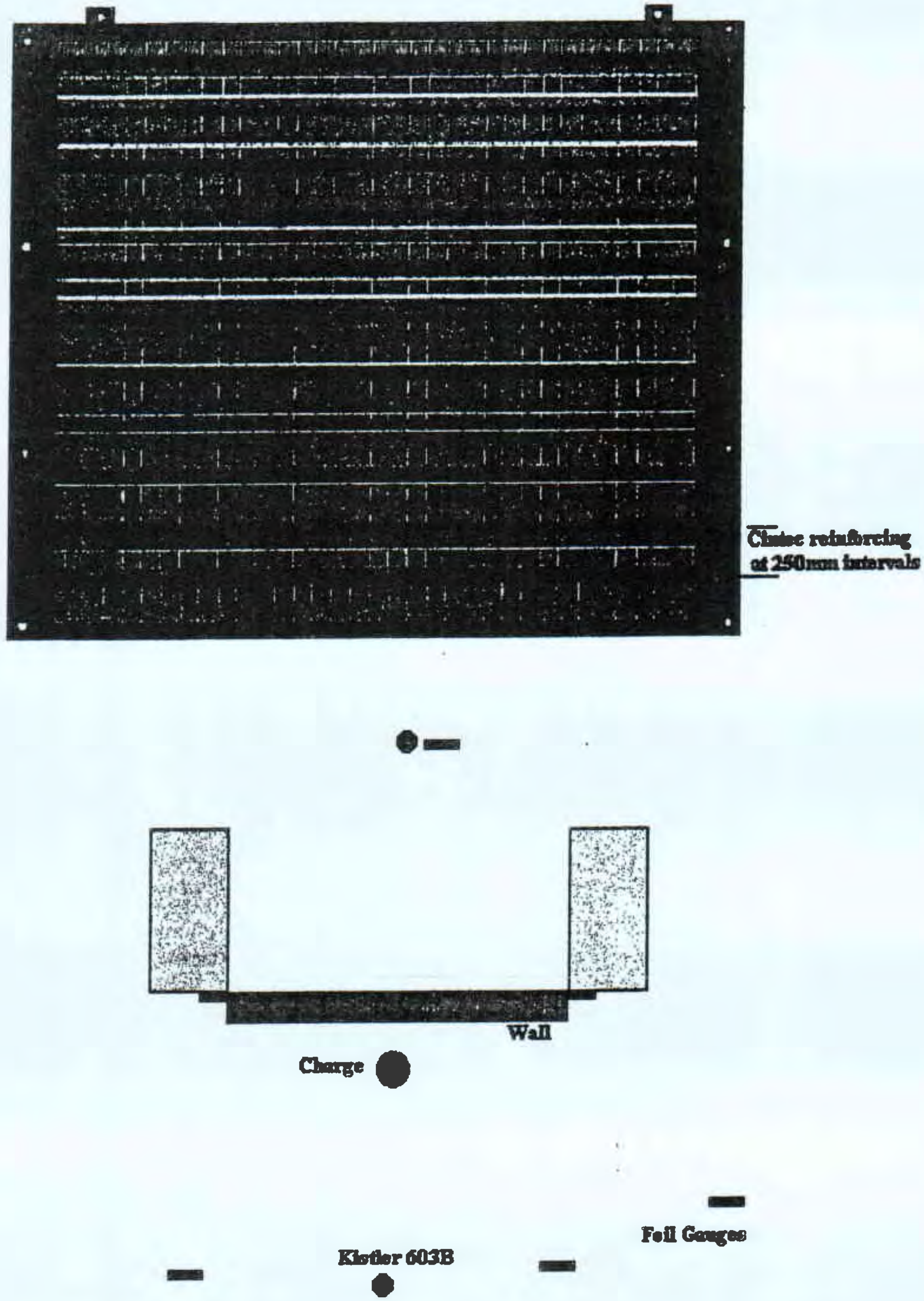
(e) Kistler type 5011 charge amplifiers serial No's. 36191 and 12789.

(f) Gould type 2608 digital storage oscilloscope serial No. 1507207.

(g) Tape measure serial No Lav 3.

(h) 4 Foil gauges.

5. DIAGRAM OF THE TRIAL SET-UP.



6. RESULTS

The video recordings were given to the sponsor on completion of the trial.

Test No	Charge Size	Pressure Gauges	Results	Remarks
1	6 Kg	Charge Side Foil gauge at 9.144m Charge Side Foil gauge at 10.668m Charge Side Foil gauge at 12.192m Blind Side Foil gauge at 10m Charge Side 603B at 10m Blind Side 603B at 10m	8.971<p<15.074 psi 4.517 <p <8.971 psi 4.517 <p <8.971 psi 4.517<p <8.971 psi 9.185 psi 5.738 psi	Test 1 was on the un reinforced wall. The charge was set-up 1m from the centre of the wall. The charge blew a large hole in the wall.
2	6 Kg	Charge Side Foil gauge at 9.144m Charge Side Foil gauge at 10.668m Charge Side Foil gauge at 12.192m Blind Side Foil gauge at 10m Charge Side 603B at 10m Blind Side 603B at 10m	8.971<p<15.074 psi 4.517 <p <8.971 psi 4.517 <p <8.971 psi 4.517 <p <8.971 psi 8.801 psi 6.260 psi	Test 2 was on the reinforced wall. The charge was set-up 1m from the centre of the wall. The rear of the wall was cracked.
3	6 Kg	Nil	NA	Test 3 was on the reinforced wall. The charge was set-up 1m from the centre of the wall. The rear of the wall had a few bricks blown out.
4	10 Kg	Nil	NA	Test 4 was on the reinforced wall. The charge was set-up 1m from the centre of the wall. The rear of the wall had a few bricks blown out.
5	10 Kg	Nil	NA	Test 5 was on the reinforced wall. The charge was set-up 1m from the centre of the wall. The rear of the wall had a few more bricks blown out and 1 brick had been blown out from the front of the wall.

Tests 2 - 5 were carried out on the same wall.

7. LIST OF PHOTOGRAPHS

PHOTOGRAPH NUMBER	Test No	REMARKS
1& 2	1	Showing the set up of the test.
Photo set No 1	1	Showing detonation.
3 & 4	1	Showing the damage to the wall after the test.
5 & 6	2	Showing the set up of the test.
Photo set No 2	2	Showing detonation.
7 & 8	2	Showing the damage to the wall after the test.
9	3	Showing the set up of the test.
Photo set No 3	3	Showing detonation.
10	3	Showing the damage to the wall after the test.
11	4	Showing the set up of the test.
Photo set No 4	4	Showing detonation.
12	4	Showing the damage to the wall after the test.
13	5	Showing the set up of the test.
Photo set No 5	5	Showing detonation.
14 - 17	5	Showing the damage to the wall after the test.

Photo No 1

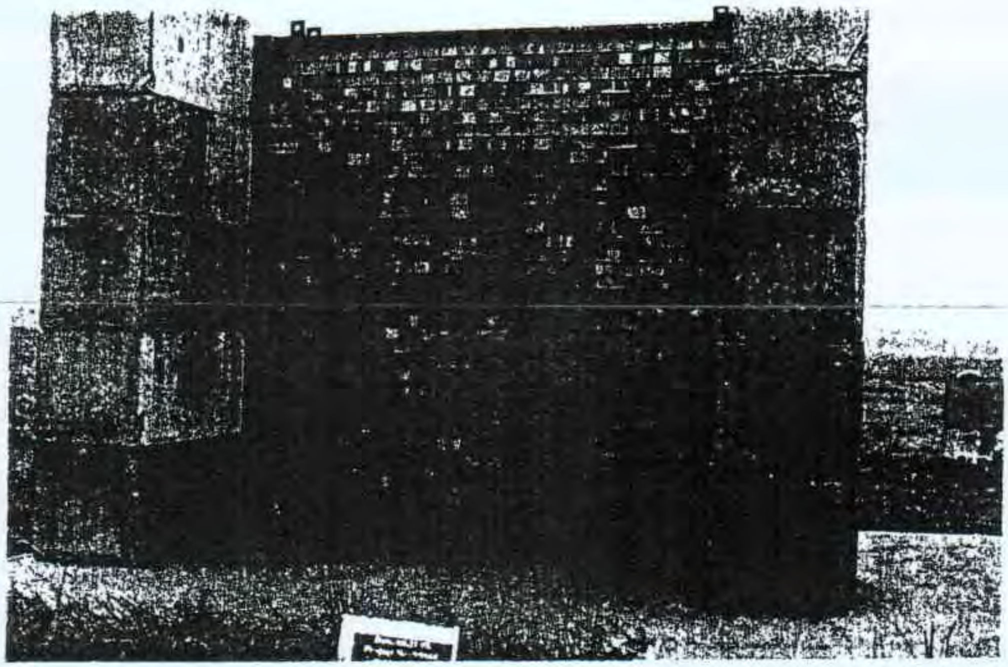


Photo No 2

Photo Set No 1

Photo No 1

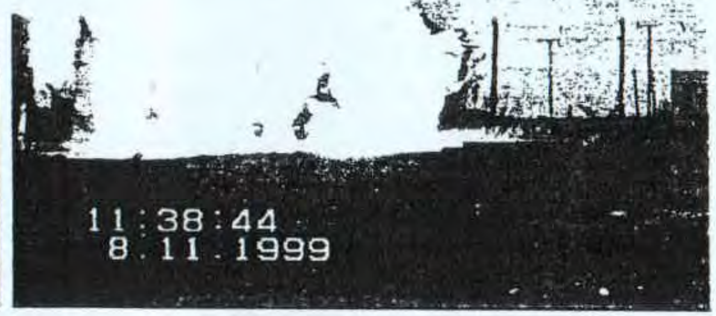
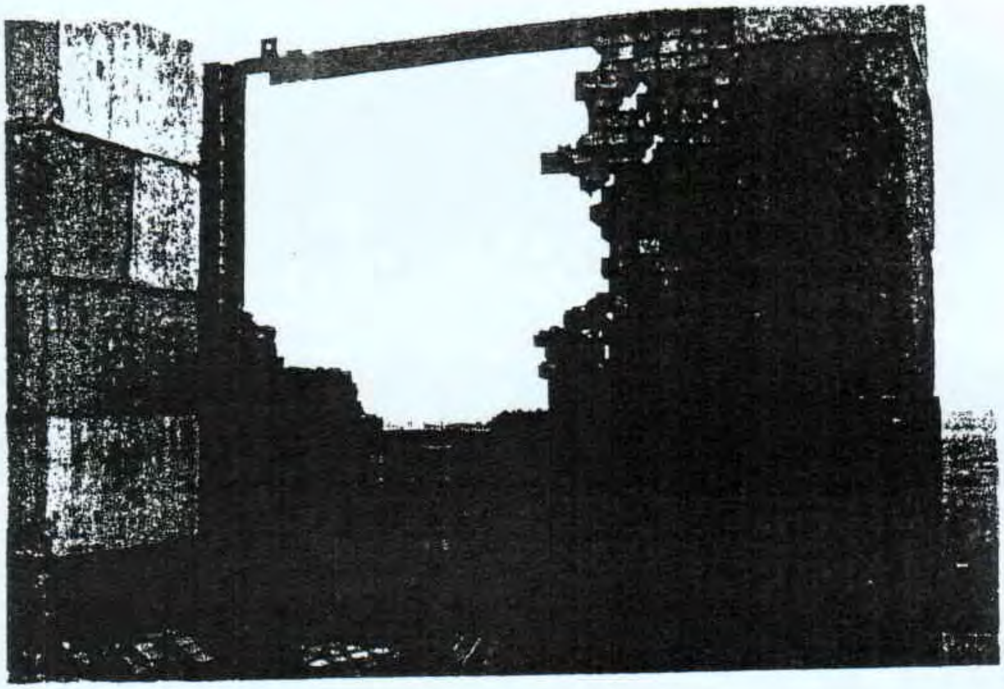


Photo No 2

Photo No 3

Photo No 4



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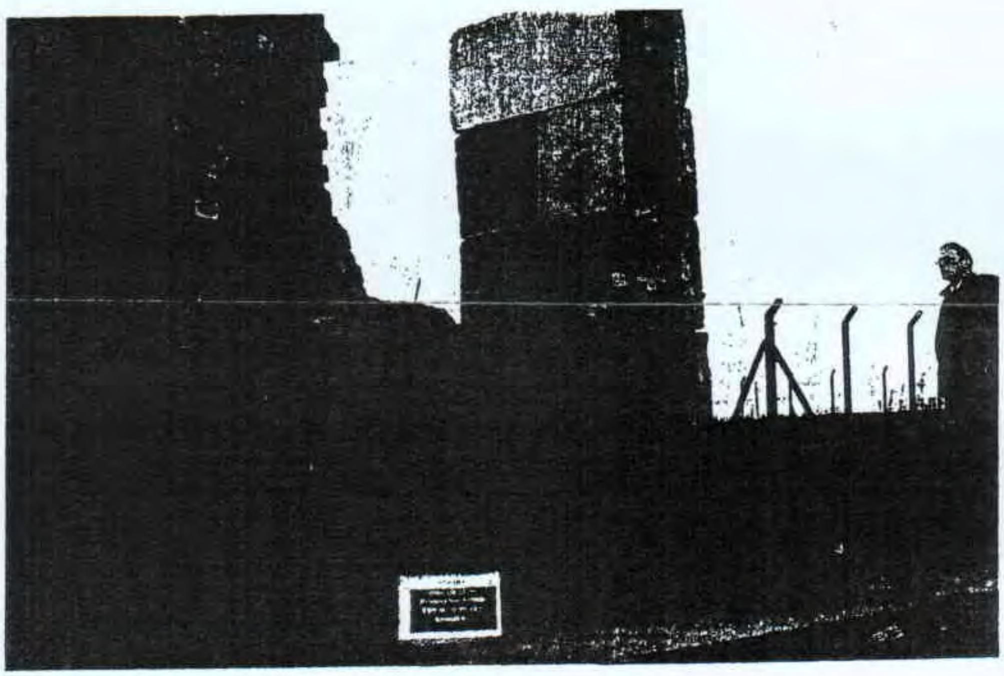

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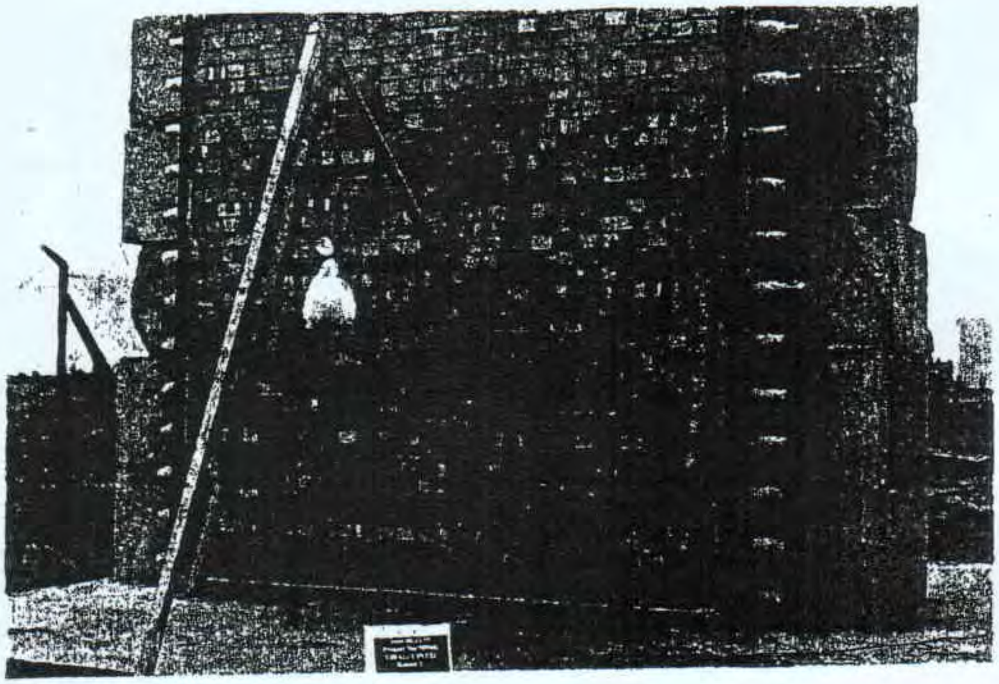

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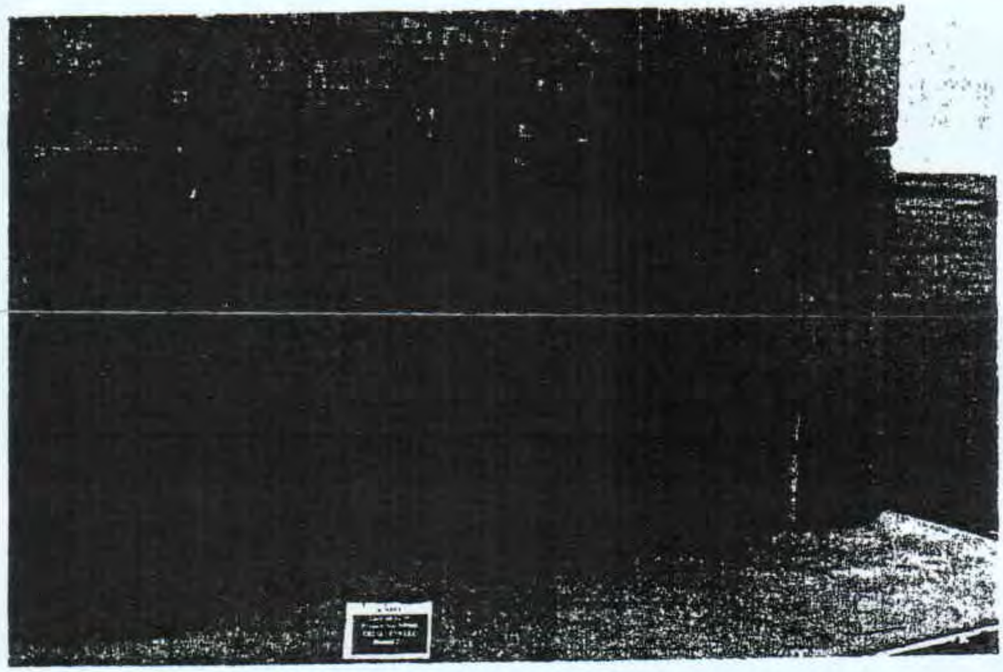


Photo No 6

Photo Set No 2

Photo No 1

Photo No 2

Photo No 3

Photo No 4

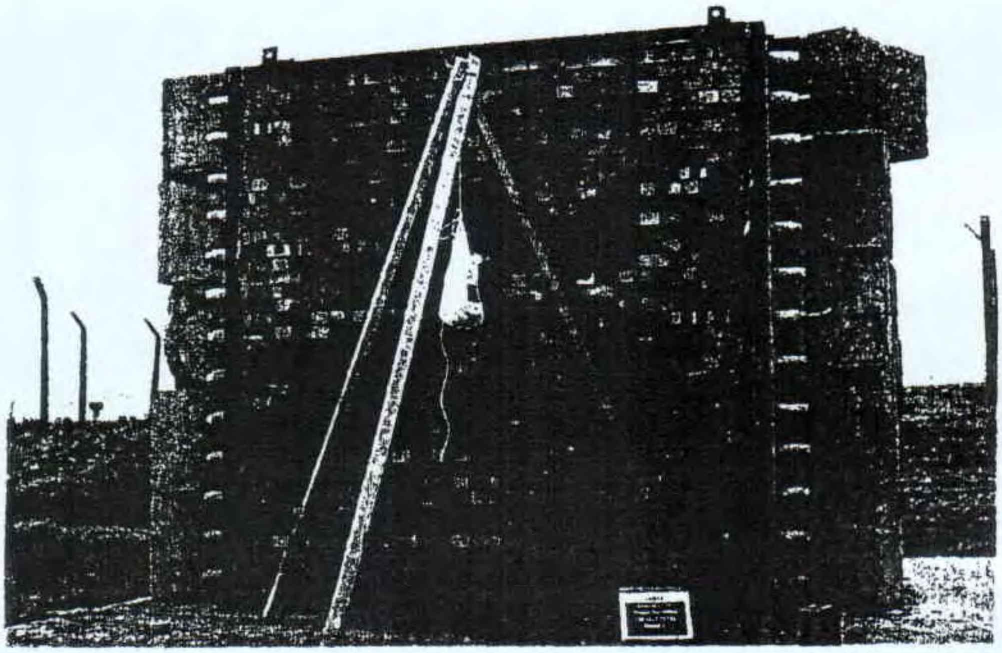


Photo No 9

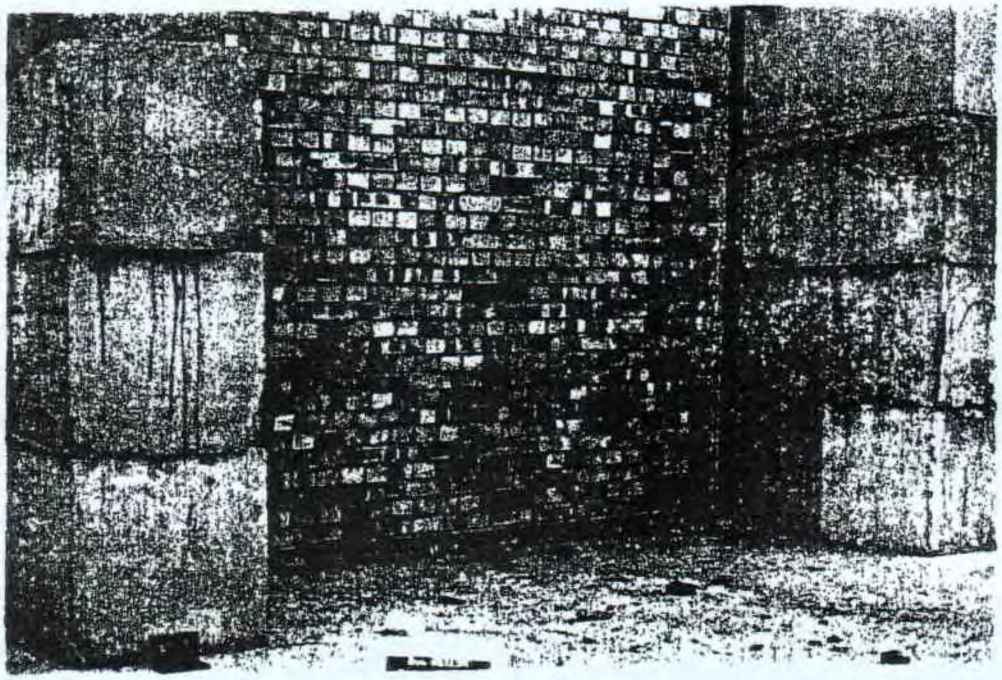
Photo Set No 3

Photo No 1

Photo No 2

Photo No 3

Photo No 4



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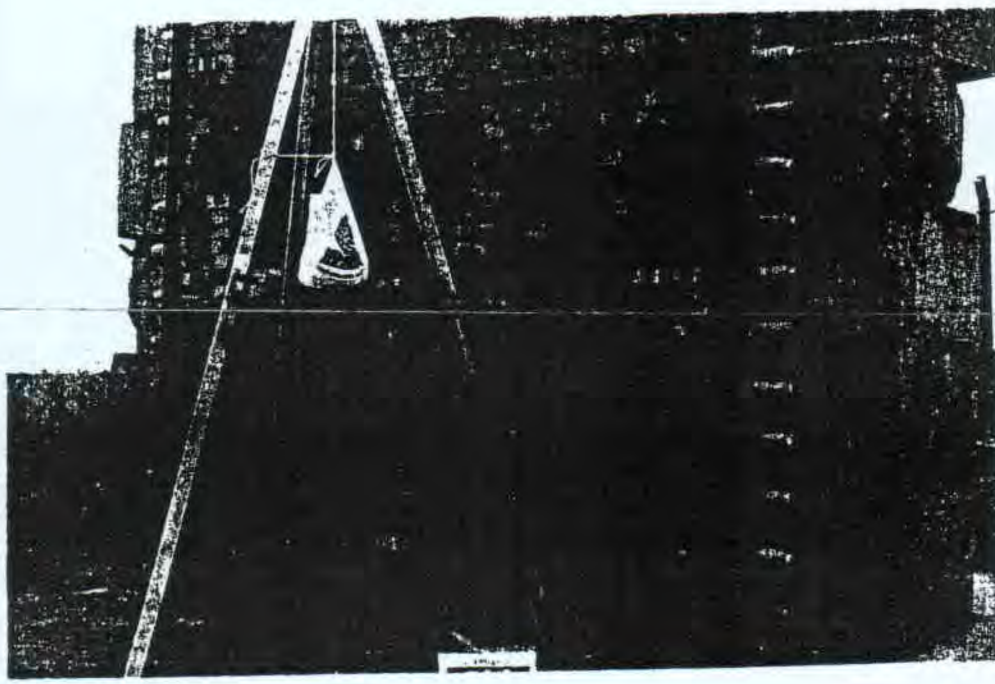


Photo No 11

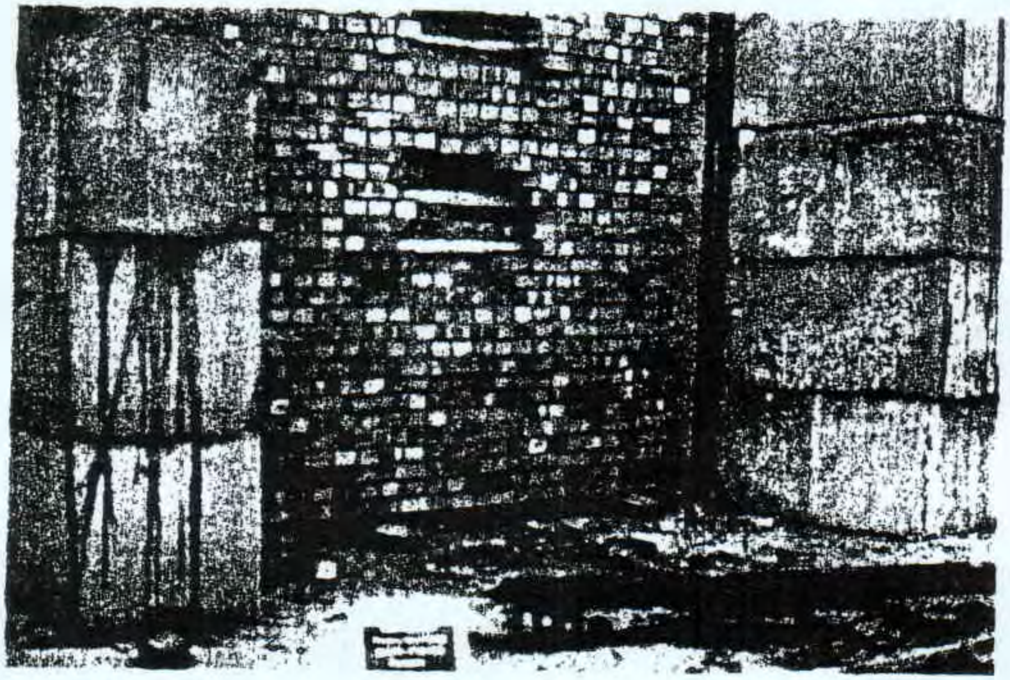
Photo Set No 4

Photo No 1

Photo No 2

Photo No 3

Photo No 4



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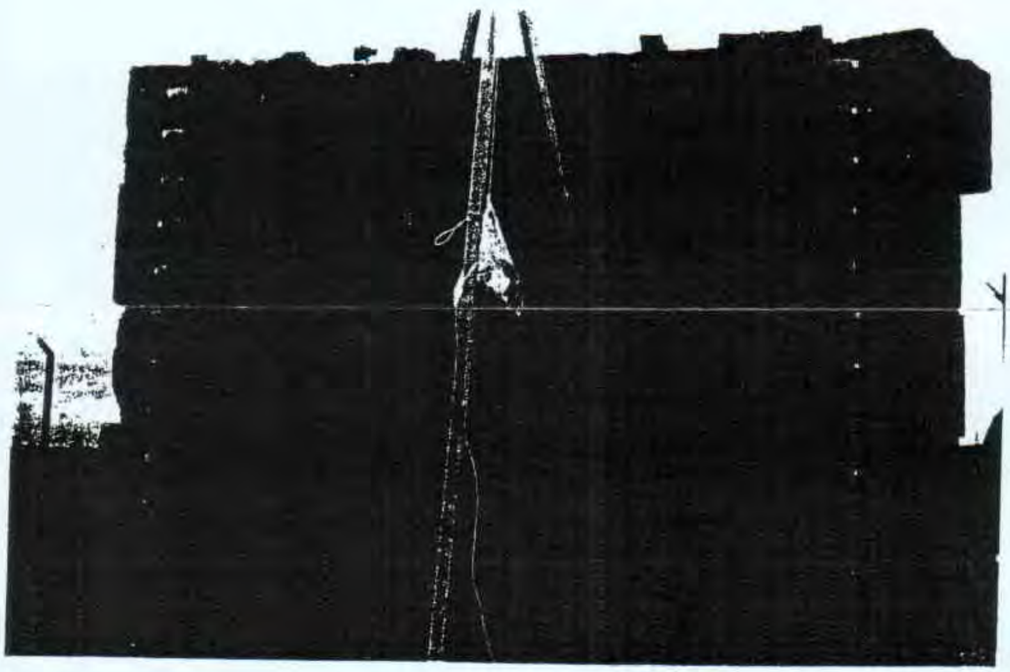


Photo No 13

Photo Set No 5

Photo No 1

Photo No 2

Photo No 3

Photo No 4

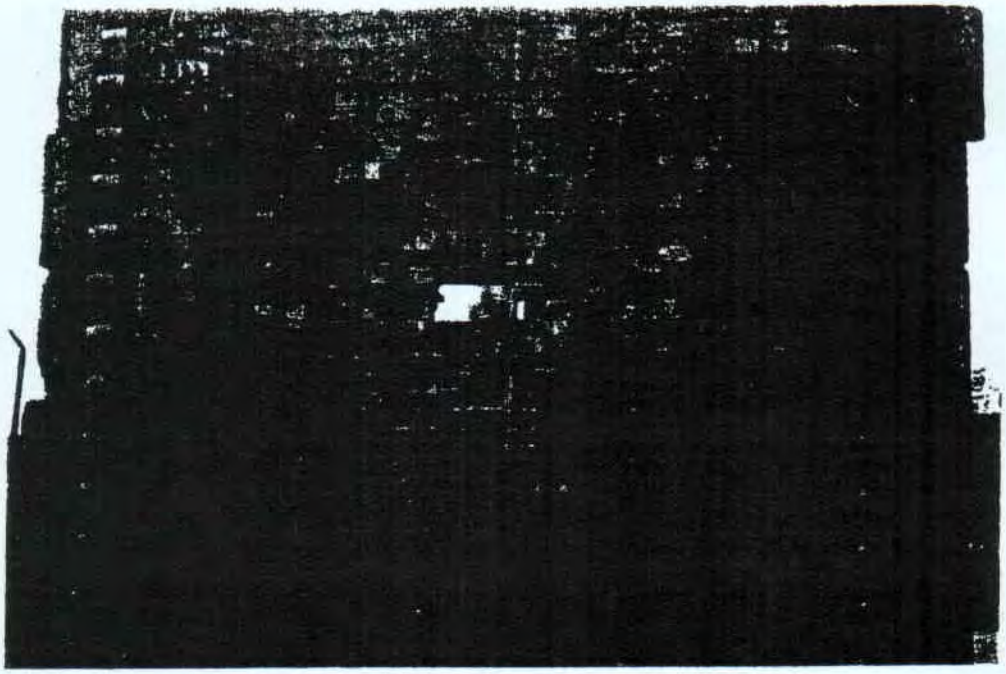


Photo No 14

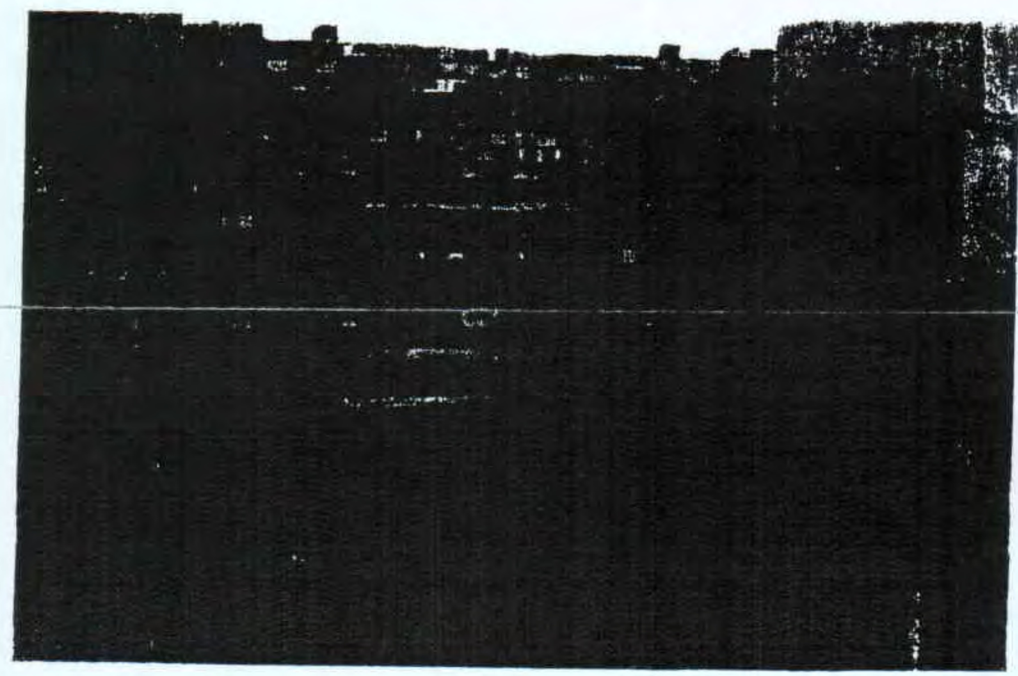


Photo No 15

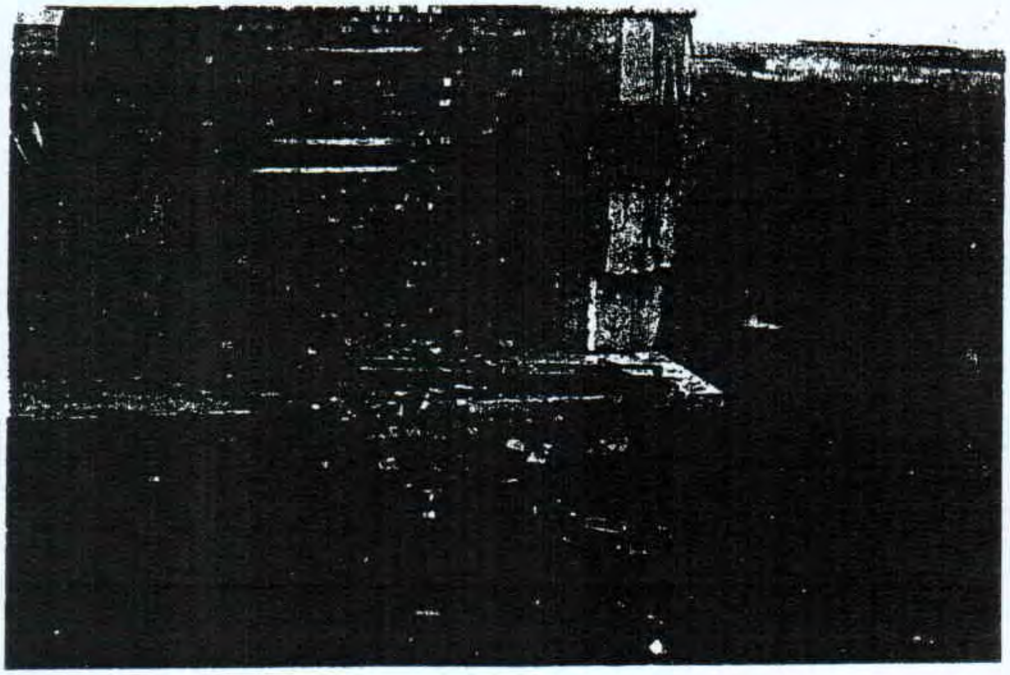


Photo No 16

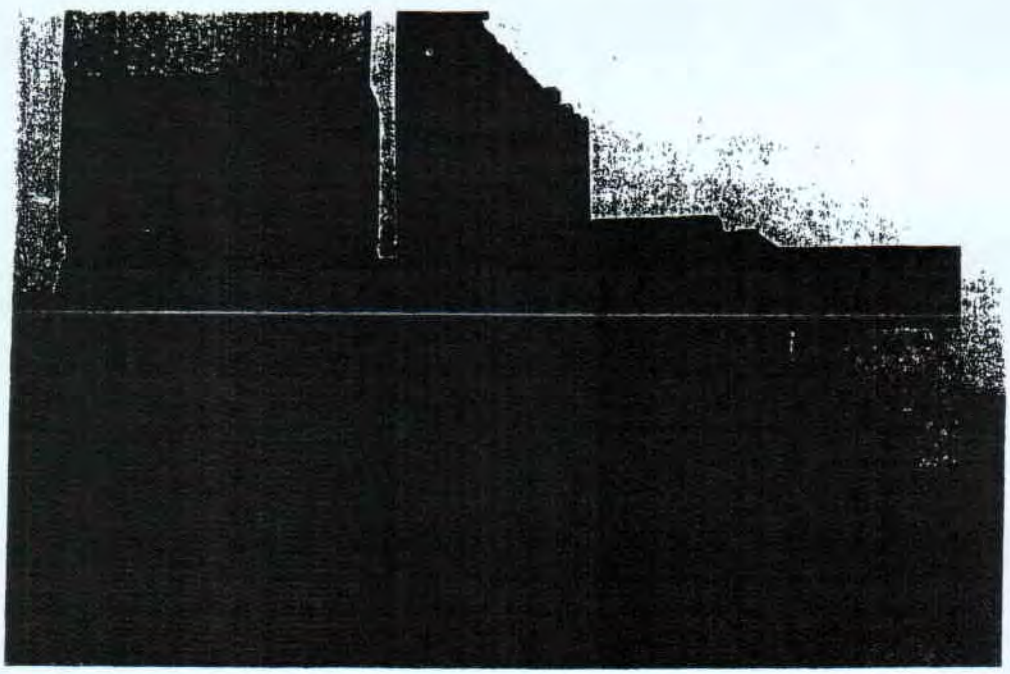
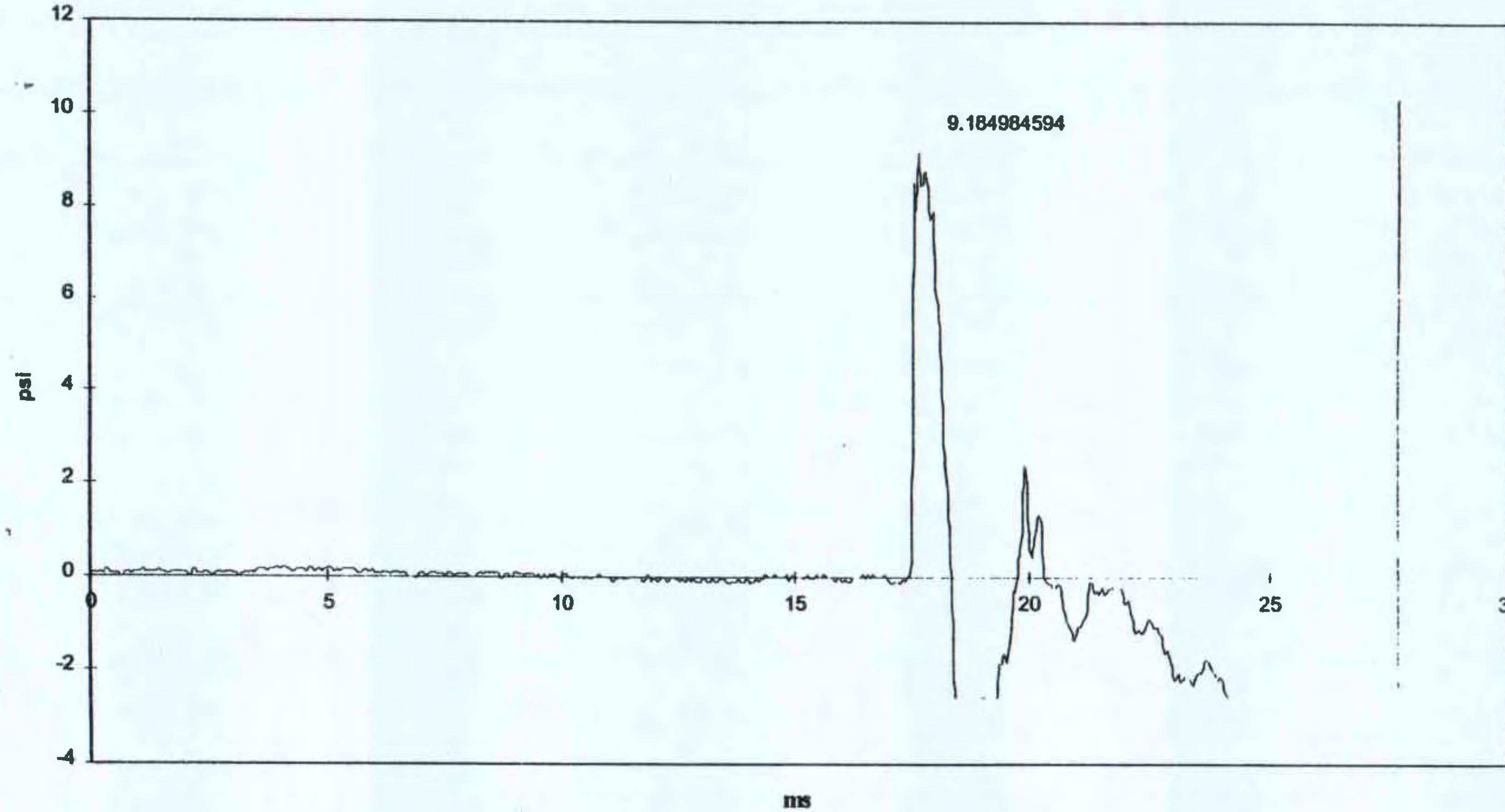
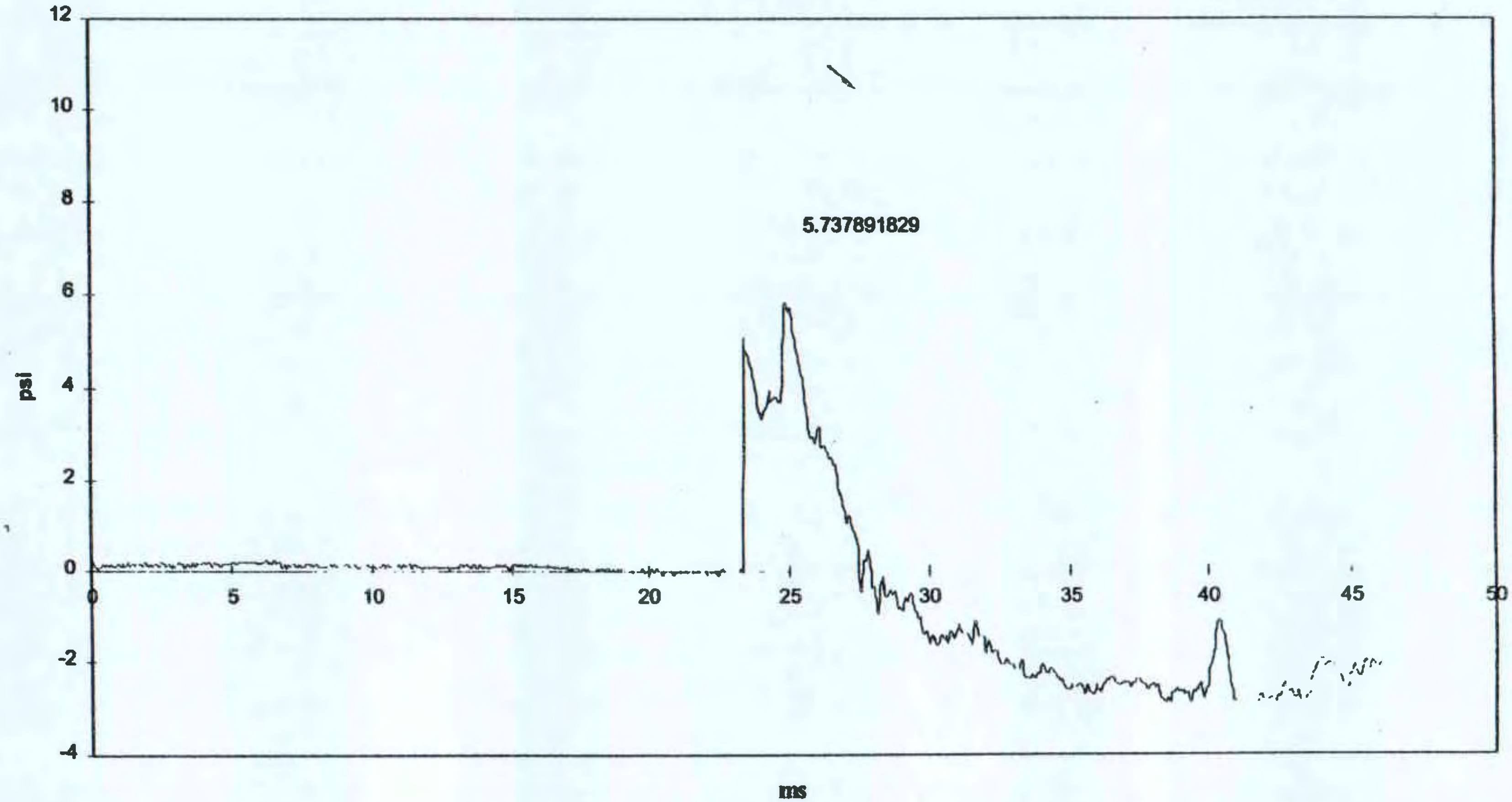


Photo No 17

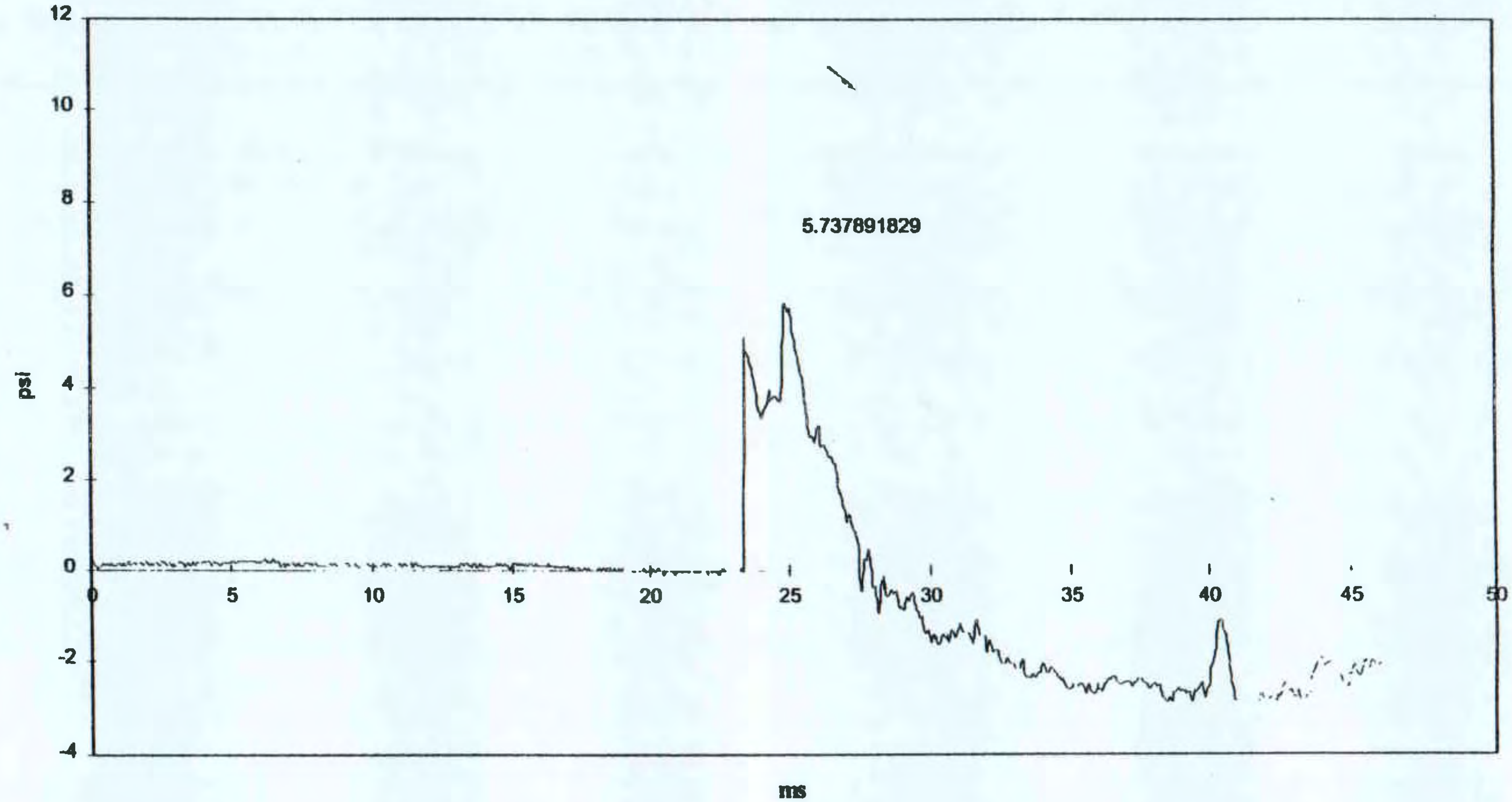
Test 1 Pressure 10m in Front of the Wall



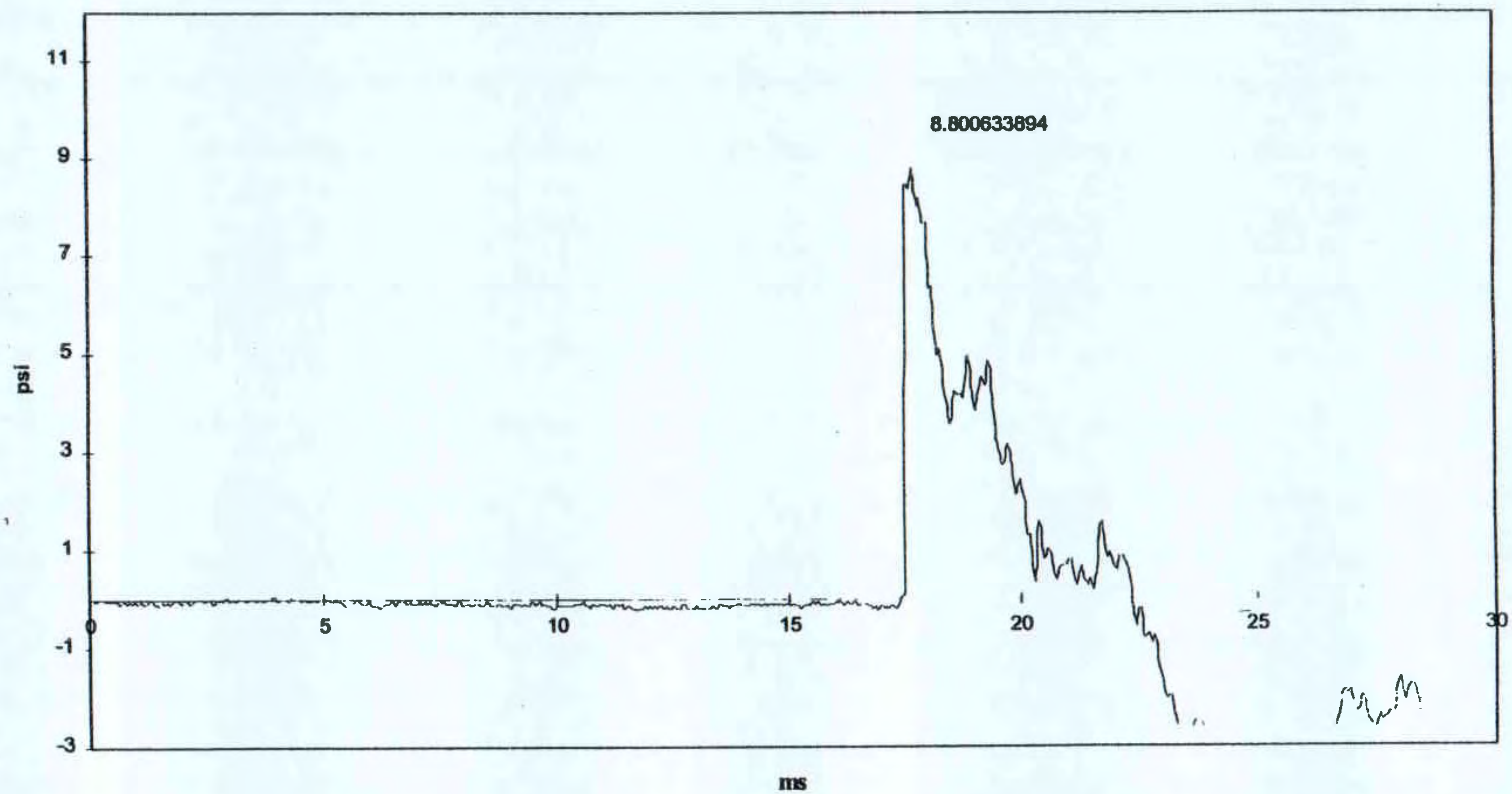
Test 1 Pressure 10m Behind the Wall



Test 1 Pressure 10m Behind the Wall



Test 2 Pressure 10m in Front of the Wall



Test 2 Pressure 10m Behind the Wall