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INVESTIGATION ON THE STRUCTURAL INTERACTION BETWEEN TRANSMISSION LINE TOWERS AND FOUNDATIONS

Working Group
B2.08

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WG B2.08

Investigation on the structural interaction between Transmission line towers and foundations

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1. INTRODUCTION

This paper reports the studies made by the joint task force between the working groups Cigre SCB2.07 and B2.08 on the structural interaction between transmission line tower and foundations. The main purpose of this task force is the study of the tolerances of the foundation displacements and their impact on the tower capacity. One of the main motivations of this research is the recurring issue about which foundation displacements could be tolerated in order not to compromise the performance of the towers, from the point of view of the design methodology based on the Limit States (ultimate and serviceability). Based on such motivation, WG B2.08 developed a research project to answer some questions often present in the discussions agenda on this issue. Within this context, this paper approaches the tower-foundation interaction through a discussion that can be summarized as follows: firstly, a basis is established by discussing the usual limits, the problem under the point of view of the limit state methodology of IEC 60826, and a comparison between theoretical and experimental data on the foundations response. After that, the experiments performed by WG B2.08 in South Africa in 2001 are presented, when vertical displacements (uplift and settlement) were applied to a tower whose structural response was carefully monitored. This paper presents a description of the prototype, of the displacement application system, of the monitoring system, and of the estimated structural response, performed by several designers. Additionally, this paper reports the research efforts to determine the structural response by using more complex structural models, aiming at improving the estimation capacity of the experimental results. The research adopted structural analysis models varying within a wide range of complexity, from a simple static linear analysis of truss elements to an analysis with slippage on the joints represented by non-linear relations between strength and displacement. At the end, a report is made on the achievements of the research, presenting some important conclusions such as, for example, evidence that the slippage in the bolted joints of the towers plays an essential role of "relieving/dissipating" the impact of the foundation displacements on the structural performance of the towers. This aspect, when accounted for in the structural analysis models, provides for an excellent correlation between predictions and experimental results.

2. MOTIVATION

Transmission Line Design Standards usually make no reference to allowable foundation movements due to their impact on the performance of the structure.

During the past 25 years, CIGRÉ's Working Groups B2.07 and B2.08 have been continuously dealing with the issue on the interaction between towers and foundations. Particularly, the issue of greatest interest is "which foundation displacement can be allowed under the point of view of the ultimate and serviceability limit states". However, the approach of this issue includes other important questions, such as:

- Which limits are usually practiced by the industry?
- What is the magnitude of the displacements the foundation can withstand considering the soil response?
- What is the magnitude of the displacements the foundation can withstand considering the structural response of the foundation itself?
- Which are the tower characteristics that can cause the most important impacts on the establishment of the displacement limits (tower height, base width, structural configuration – whether isostatic, hyperstatic, etc.)?
- How can this interaction be accounted for?
- What is the Ultimate Limit State in this interaction?
- What is the Serviceability Limit State in this interaction?
- What is the most critical movement (vertical, horizontal, or a combination of both?)
- Within this context, the article presents a summarized report on the progress achieved as a result of contributions made by several professionals in the Cigré B2.07 and B2.08 groups.

3. TECHNICAL BASIS

3.1. Industry practical Construction Tolerances

As previously mentioned, Transmission Line Standards do not specifically refer to the foundation movement issue, relating it to the behavior of the structures. So, OHL experts have continuously discussed which limits to adopt. For example, B. Schmidt[1], based on investigations about the issue, stated that "the tower owns a certain intelligence to make up for part of the strength due to foundation movements caused by the bolts slippage."

Regarding erection, the foundations are allowed some tolerances, such as those suggested, for example, by F. Villa [2], (Table 1 and Figure 1):

Table 1: Allowable Construction Tolerances

Tolerance	Allowed values
Base width	$\Delta x \leq \pm$ 0,1 % length of width length B
Diagonal distance	$\Delta d \leq \pm$ 0,1 % length of diagonal length D
Stub level	$\Delta z \leq \pm$ 0,1 % length of width/2 (B/2)
Batter axis	$\Delta r \leq$ 0,5 % length of stub above the concrete top length L
Angular distortion	$\Delta \gamma \leq \pm 0{,}5^0$ (degree) on stub angle twist angle γ
Tower distortion	$\Delta \delta \leq \pm 3^0$ (degree) on centre line
Tower eccentricity	$\Delta e \leq \pm$ 0.5% length of diagonal length D

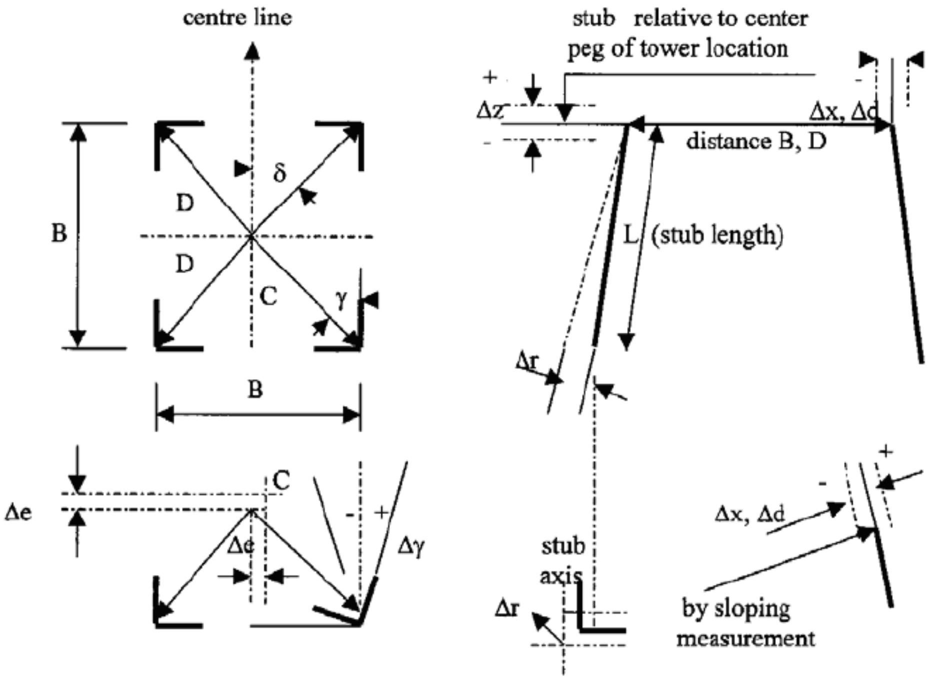
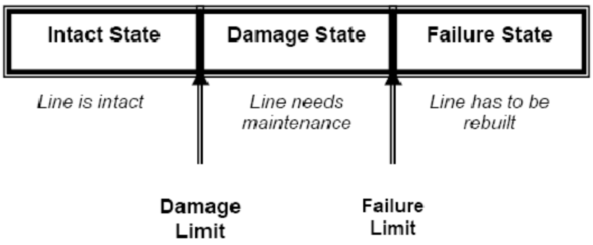


Figure 1: **Tolerances for tower erection**

3.2. Limit State Methodology

The limit state methodology allows an appropriate basis for decision making about different design alternatives in terms of reliability or its complement, the risk. Additionally, such basis also allows optimized designs without reduction in reliability. According to this concept, the transmission line is considered as a system whose "states" are illustrated as follows:



Where:

- The Ultimate Limit State is related to the ultimate capacity, that is, to the total and/or partial failure or collapse of the system and/or of a component, which, in this approach, means failure of the tower or the foundation;
- The Serviceability Limit State is related to the limit from which the tower or the foundation needs to be repaired.

The Ultimate and Serviceability Limit States are associated with the function of the Transmission Line and with the durability of its materials. Under this approach, the Serviceability Limit State (the Use Limit) is the limit from which the transmission line may be subject to a temporary interruption for repairs. On the other hand, the Ultimate Limit State is the limit from which the transmission line loses completely its power transmission capability.

Regarding the performance of the foundation, any displacement may cause a reduction in the ultimate capacity of the tower and/or in the ultimate capacity of the foundation itself. However, it should be assumed that, the magnitude of this reduction in terms of the tower strength capacity, will depend on its characteristics such as its stiffness.

Therefore, the establishment of these displacement limits shall be based on the interaction between tower and foundation, and it shall be based on the Limit State philosophy.
According to IEC 60826, the general design equation is as follows:

$$\mathbf{Q_T \leq \Phi_R R_C}$$

Where:

- Q_T is the effect of the load associated with a return period equal to T years;
- Φ_R is the strength factor that considers the (Φ_S) strength coordination (or preferential failure sequence), (Φ_N) number of components submitted to the maximum load intensity, (Φ_Q) difference

in quality between the test and the actual application, and (Φ_C) the difference between the actual exclusion limit and the limit e=10%, being $\Phi_R = \Phi_S \Phi_N \Phi_Q \Phi_C$,

- R_C: Nominal or characteristic strength

So, after the establishment of the general formulation, the calibration of models is required, together with the assessment of strength coefficients.

3.3. Comparisons between Field Tests and Estimates

Regarding the application of the IEC 60826 methodology, capacity estimation models have been calibrated through the comparison with test results. Examples of these efforts are those carried out by WG B2.07[3], reported for a shallow foundation submitted to uplift, which may be exemplified by figure 2.

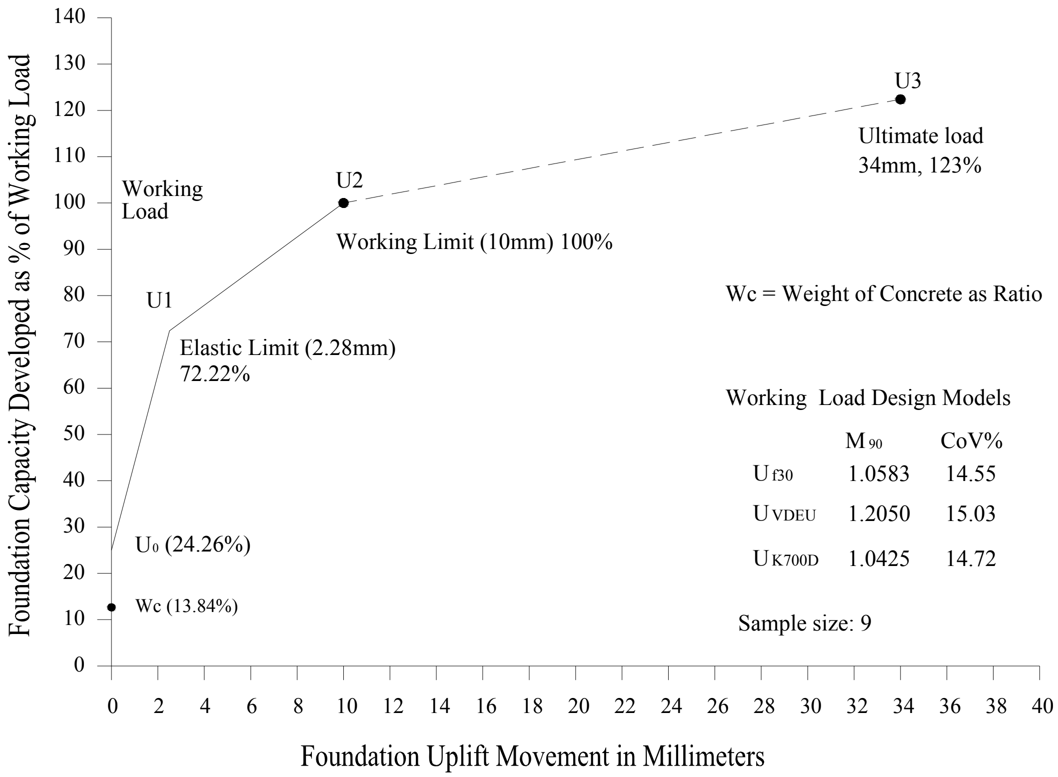


Figure 2: Foundation movement versus Foundation Capacity

From these studies, expressions similar to those shown below can be established. For example, supposing that the limit displacement for the service condition is about 10mm, the design equation for the foundation "serviceability limit state" would be:

$$\gamma_U \, Q_T < \Phi_R U_{f30}$$

$$\gamma_U \, Q_T < \Phi_S \text{ x } \Phi_N \text{ x } \Phi_Q \text{ x } \Phi_C \text{ x} U_{f30}$$

$$0.90 \, Q_T < 0.93 \text{ x } 0.98 \text{ x } 1.0 \text{ x } 0.9768 \, U_{f30}$$

that is,

$$Q_T < 0.9892 \, U_{f30}$$

For the "ultimate limit state", it would be as follows:

$$0.90 \, Q_T < 0.93 \text{ x } 0.98 \text{ x } 1.0 \text{ x } 1.3138 \, U_{f30}$$

$$Q_T < 1.33044 \, U_{f30}$$

Where:

- Q_T: is the effect of the load associated with a return period equal to T years;
- U_{f30}: the foundation capacity evaluated by the inverted cone methodology, with a 30^o angle.

These expressions already take into consideration the failure coordination, or preferential failure sequence, between tower and foundation. Additionally, it should be clear that these expressions are valid for this specific case. Similar expressions can be established for other cases with relative conceptual easiness, in view of the clear logic of the IEC 60826 philosophy.

4. THEORETICAL EXERCISE

In this context, the following numeral simulation has been proposed to be carried out within the scope of the interaction between SC B2.07 and B2.08:

a) Select a typical representative suspension structure;
b) Analyze the tower response in terms of strength due to representative loading cases, such as extreme transversal wind;
c) Simulate the foundation movements in some specific soils as result of the loading:

- vertical settlement;
- uplift;

- horizontal displacement.

Values such as 25mm for vertical displacements, uplift or settlement, and 10mm for horizontal displacement, should be taken into account as they represent "the maximum practical accepted limits" as reported by the WG07 and/or WG08 TF members [4].

d) Evaluate the influence of these movements on the tower and on the foundation structure;
e) Extend the conclusions of this exercise to other conditions: different characteristics for towers; types of soils and foundations.

Part of these studies has already been carried out, especially those regarding the theoretical and analytical evaluation of the problem, relating them to the tests performed hereafter described.

5. EXPERIMENTAL TESTS

This study was associated with another research project already conducted by WG B2.08 concerning the influence of towers modeling. Therefore, the theoretical study about the influence of the foundation movements was initially carried out on the three tower prototypes tested in South Africa (Figure 3).

5.1. Description of the Prototypes

The three tower prototypes tested in South Africa were especially designed to study the influence of modeling and the estimated structural response. Figure 3 shows those silhouettes, while Figure 4 shows the pictures of the towers on the test pad.

The three towers are visually almost identical, but with slight differences in the bracing. These differences represent typical bracing alternatives frequently adopted by industry practice. Prototypes 2 and 2A are solutions with a higher degree of internal hyperstaticity. This way, since balance can be established in several different load paths, the towers are expected to present different behaviors, as well as foundation reactions. Prototype 2 was chosen to be tested simulating foundations displacements due to its degree of hyperstaticity as compared with structure number 1, which is isostatic. It was also decided to simulate the foundation movements in just one tower leg, since this "differential movement" seemed to be more critical as result of the theoretical analysis.

The displacements ranged from 0 to 25mm of uplift and corresponding unloading to 0mm. From this point, a settlement down to 25mm was simulated, as well as the corresponding unloading up to 0mm again.

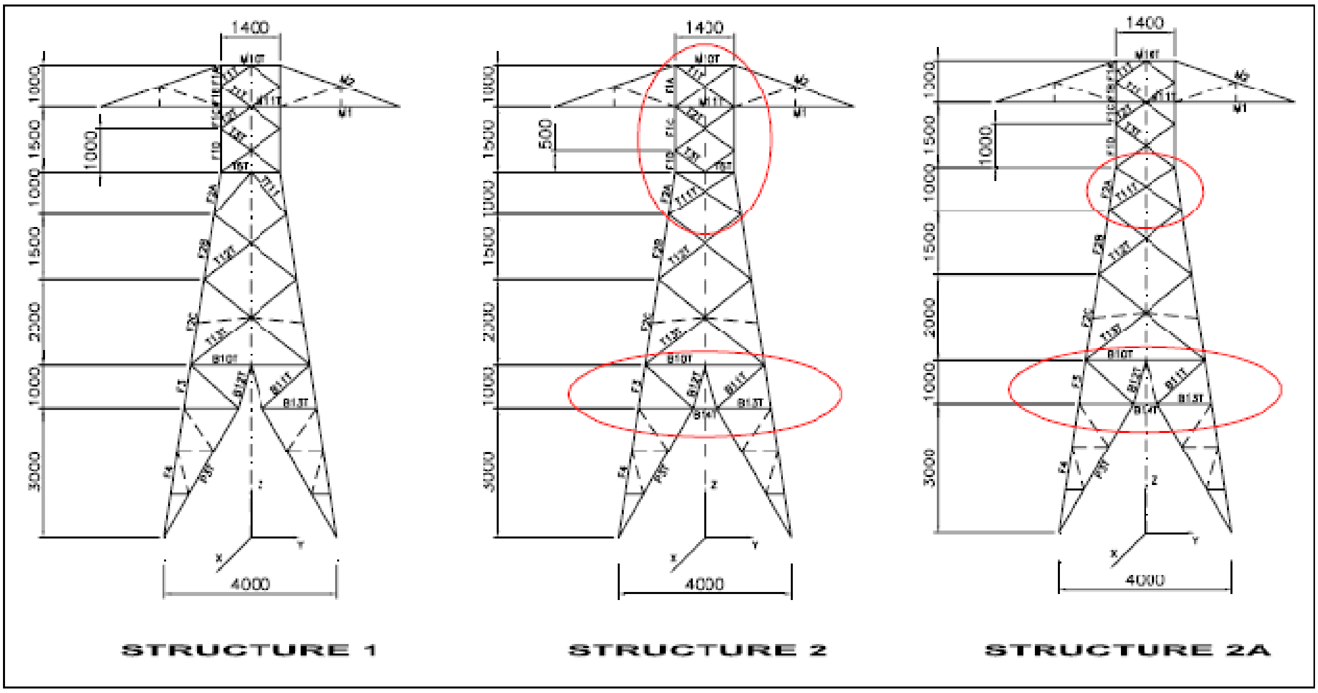


Figure 3: Configuration of the three towers tested in South Africa

Structure 1 Structure 2 Structure 2A

Figure 4: Tower prototypes on the Test Pad

5.2. Estimated structural response

The structural response was analyzed in order to establish a member force value to be imposed on the structure leg. The purpose of these calculations was to determine displacements that would allow a safety margin during the tests, considering the presence of workers operating devices during the application of the displacements. The results of these calculations can be summarized in figure 5 below. This figure shows that bar F4 (main member) achieves its ultimate capacity (that is, use factor equal to -1, in compression) close to 46mm of foot displacement. Considering this result, the limit of 25mm was established during the tests, as this value is adopted in many countries around the world as the maximum acceptable limit by industry practices.

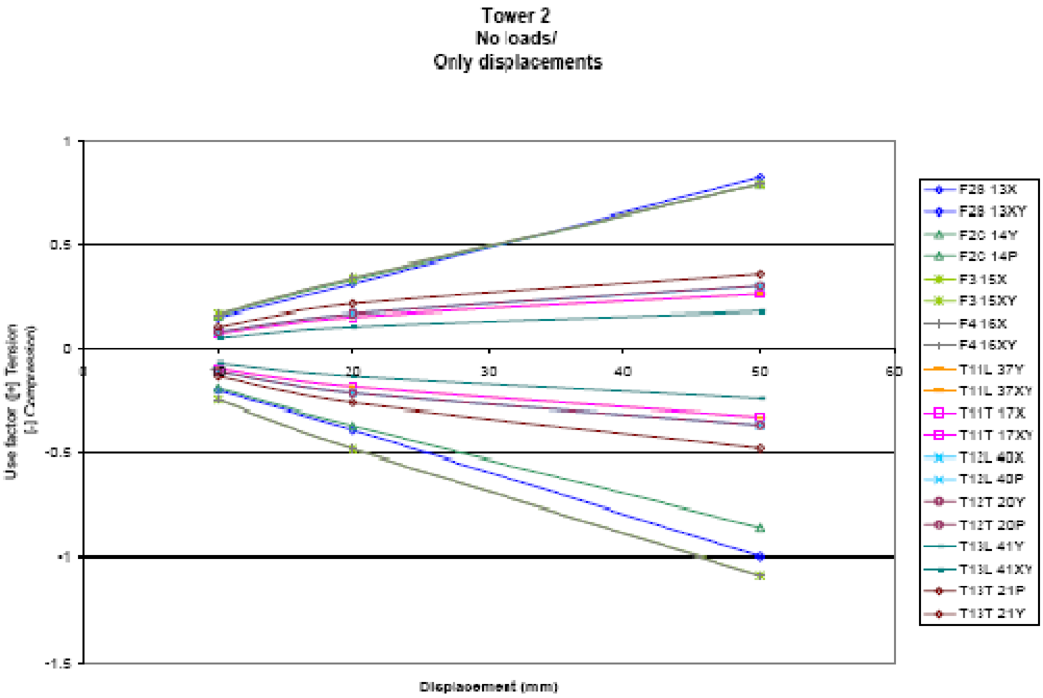


Figure 5: Study to determine test displacements

5.3. Tests

A strain-gage system was developed by TSI – *Technology Services International*, ESKOM *Transmission Group*, to monitor the forces on the bars. By this system, axial forces and bending moments could be measured and observed in real time. Figure 6 shows details of a strain gaged member.

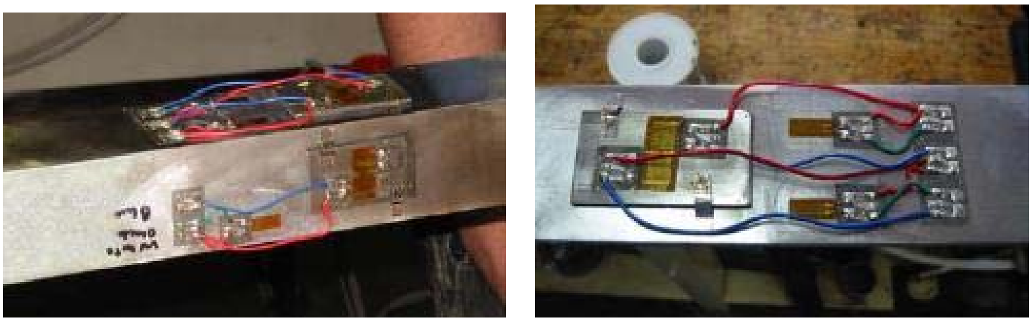


Figure 6: Detail of a strain gaged member

Figure 7: Device designed for application of displacements on tower

A special device was designed for the application of the displacement at the tower foot, as shown in figure 7.

The complete strain gaged member was calibrated under the unloaded bar condition (figure 8), establishing the proportionality constants for determining member strain (load) and then, after the erection of the tower, “zeroing” the values when the external loads were not acting. Annex A contains the complete test results.

5.4. Test Results

Some of the test results, as compared with the corresponding predictions, can be seen in the following figures (figure 9). In these figures, bars F4 is a main member, bar B11L and T12L are longitudinal diagonals, and bar P3L is a tower leg diagonal. In Annex B are shown the “estimated *versus* measured” loads on all monitored bars for the uplift test.

The test results show a general trend that the measured values are reasonably lower than those calculated using linear models, that are currently used in design practice.

Figure 8: Calibration of a strain gaged member

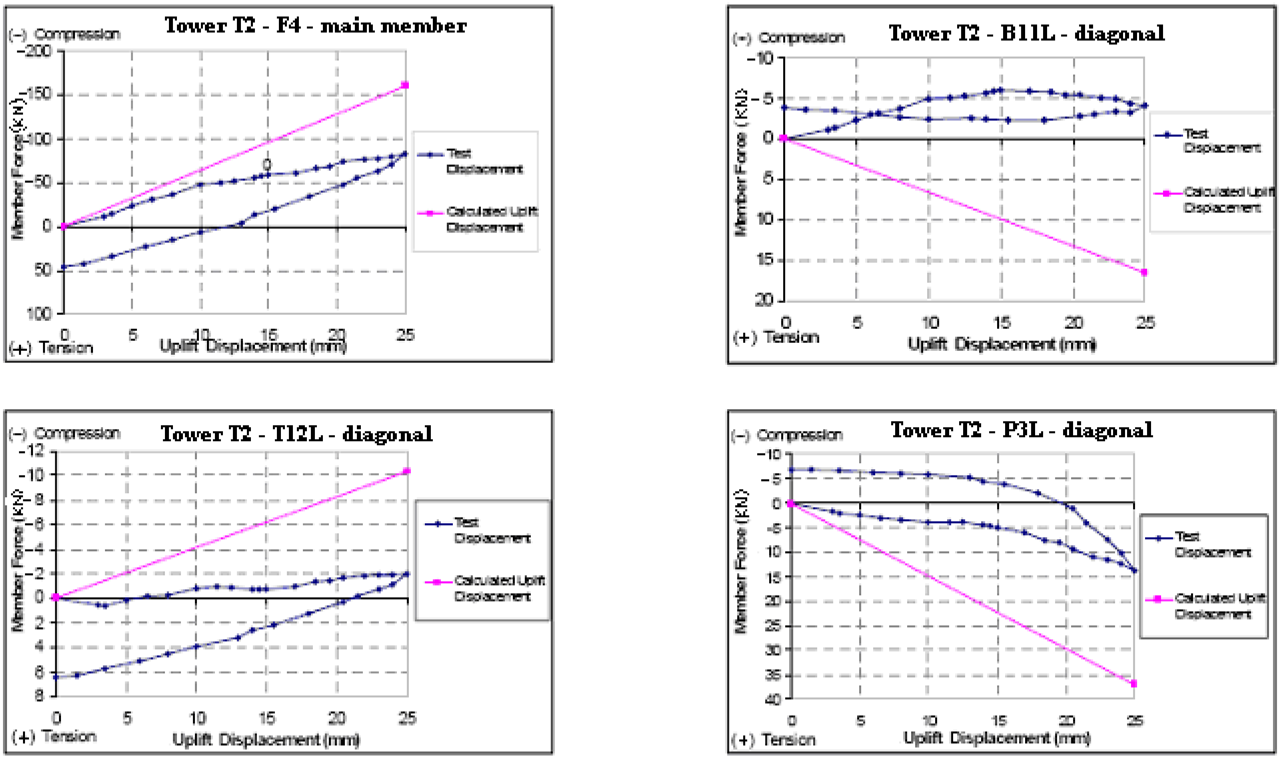


Figure 9: Test results *versus* estimate

6. ANALYSIS AND RESULTS

These test results lead to some conclusions, as follows:

• Regarding the structural response of the main members, the current structural analysis shows that, due to the uplift movement of the foundation, the corresponding main member would be under compression. Analogously, when the foundation is subject to a vertical settlement, the corresponding main member would be under tension.

• So, it can be inferred that when an uplift displacement occurs in the foundation, the phenomenon will be associated with a significant compression force. From this behavior shown in the above figures, it can be concluded that the displacement causes a "relief" on the butt joints. A similar and analogous behavior can be described for the opposite movement: it can be inferred that when the foundation experiences an uplift, the phenomenon will be associated with a significant tension. The foundation movement would cause a "relief" on the bar forces. Neglecting the reliefing affect results in conservative forces.

• During the tests, it could be observed that the bar loads were significantly lower than those calculated using a linear-elastic structural analysis. This experiment motivated the development of more sophisticated models to take bolt slippage into consideration. These more sophisticated models, developed by Kaminski Jr. [5], provided a better representation of the results, as shown in the figures below. In general, the bolt slippage occurred in the connections also acts as a "relief" of the member loads due to the foundation movement.

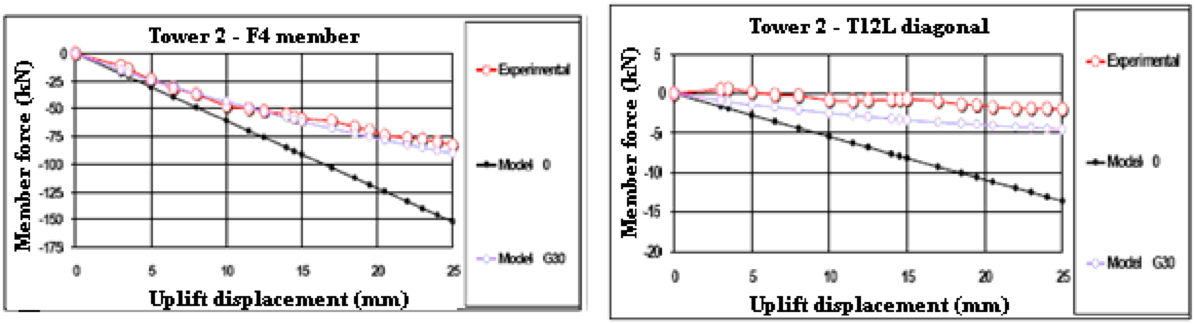


Figure 10: Estimates with bolt slippage models

With these more sophisticated models, like those shown in Figure 10, the bolt slippage allows better estimates of the tower response.

7. CONCLUSIONS

Important conclusions can be derived from this experience. Among the most important ones are:

- The calculations and practices currently adopted by the industry, even if proved to be a little conservative are confirmed as valid.

- The structural behavior, as well as the loads on bars due to those practical foundation displacements as accepted by the industry, can be understood and estimated.

- Using refined structural analysis models and calculations methods, for example, those that take into account the impact of the bolt slippage on joints, the calculated load values on bars approached very much those measured during the tests. Those analyses were done by Kaminski Jr.[5], and can be seen on the graphs of the figure 10.

- Looking at the graphs of the Figure 9, it can be seen that all the initially estimated values for bar loads were larger than those measured. The current industry calculation practices are, generally, on the conservative side. The real tower behavior and accommodation movements could be seen during the tests, and assessed by post-tests calculations using refined analysis models. The non-linear analysis that take into account bolts slippage on joints simulate well what happens when the structure experiences small foundations displacements. Those calculations can be seen on graphs of Figure 10. The bolts slippage acts as "relievers" on the main members loads and, so, the graphs show better calibrated values.

- This can be understood as if the possible slippage on joints (especially on main members) and the detailing clearances on all the holes (1/16" or 1.5mm) provide the structure a relieving condition here named "self internal relieving system". This could be one of the important reasons for explaining why measured values are always smaller than those estimated.

- Aiming to extrapolate the results to other towers and/or soils, it could be expected even better "relieving" effect. This is because soils are never so rigid like the test base used on South Africa tests and even due to the fact that Cigré TF4 Prototype 2 had only one main member splice-joint. It is expected that many main members' splice-joints (real towers), can increase the "self internal relieving" action.

- As said before, a better predictable structural behavior regarding foundation displacements is also expected by prototype 1, than the other towers which are less rigid and having smaller degree of hyperstaticity.

- The joint task SCB2.07 and B2.08 believes that those inedited results can contribute to the understanding of the real behavior of the structure during small foundations displacements and can also stimulate further studies.

8. ACKNOWLEDGEMENTS

WG08 acknowledges the support received from: Eskom, ABB, Roy Macey, Mark Newby, S.J. Madibu, M. Megale de Carvalho.
The convener also thanks K. Papailiou and E. O'Connor, SCB2 official reviewers, for their work and comments.

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[3] BUCKLEY,M., Reliability Based Design of OHL Foundations - Cigré Session 1994 nr. 22-203, and, Vertical Movement of Tower Foundation Under Load(draft), doc. 22-98(WG08)12.

[4] Foundation Installation – An Overview, Cigré SCB2.07 Technical Brochure nr. 308.

[5] Influence of the Hyperstatic Modeling on the Behavior of Transmission Line Lattice Structures, Cigré SCB2.08 Technical Brochure.

FOUNDATION DISPLACEMENT TESTS

ANNEX A: TEST REPORT

SC B2.08 TF: TOWER FOUNDATION INTERCONNECTION

LINE ENGINEERING SERVICES
TOWER TESTING STATION , ROSHERVILLE
JOHANNESBURG , GAUTENG PROVINCE
SOUTH AFRICA

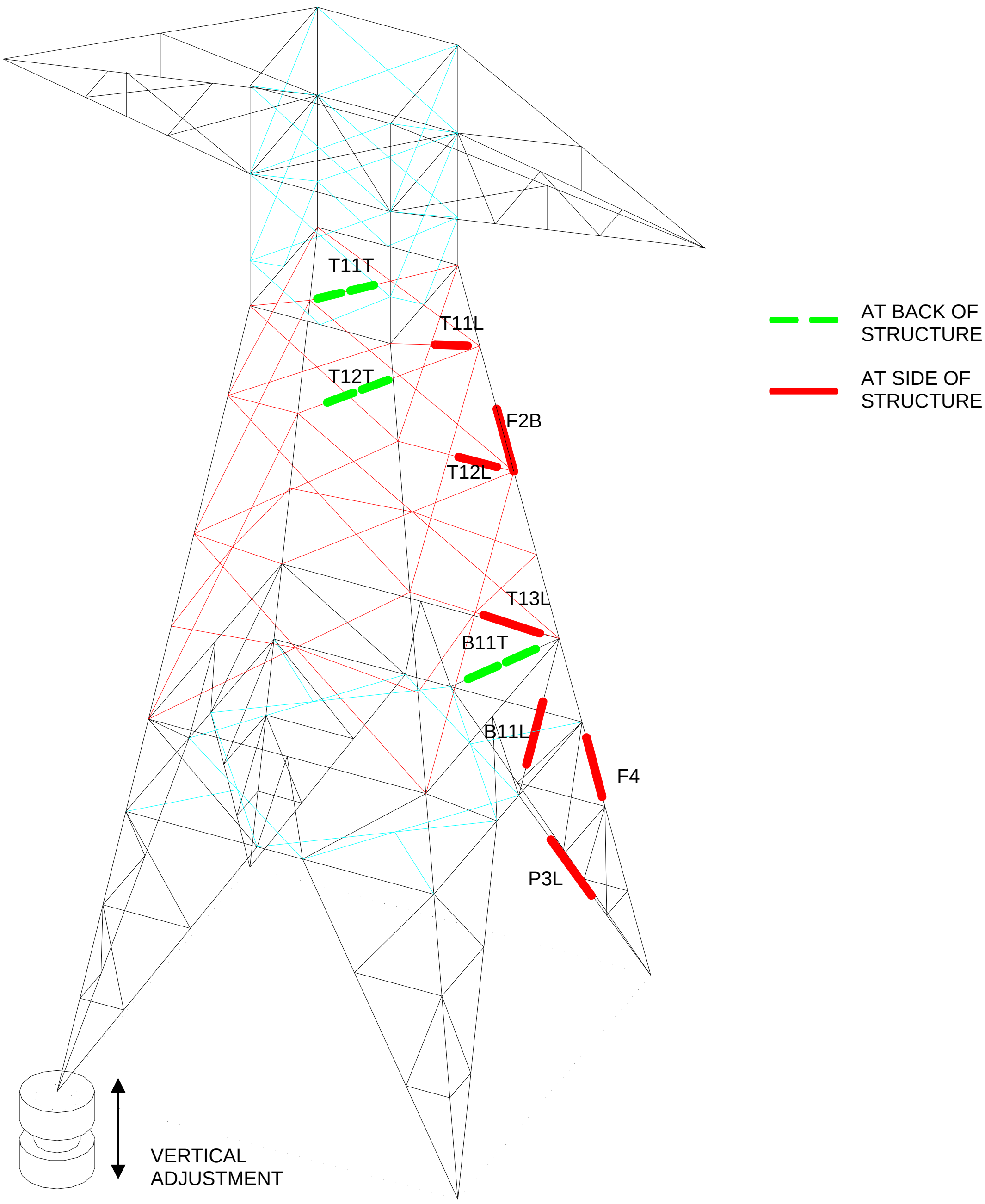
TEST REPORT NUMBER : SC 00 / 542
PROJECT NUMBER: RO - 423

TOWER TYPE :	**" STRUCTURE 2 "**		
TOWER DETAIL :	**ABB DRAWINGS:**	**(T - 47792** **(T - 47793**	**TRANSVERSAL FACE)** **LONGITUDINAL FACE)**
MANUFACTURER :	**ABB Asea Brown Boveri**		
CLIENT :	**CIGRÉ**		
PROJECT:	**WORKING GROUP 08 - TASK FORCE 7 INFLUENCE OF HYPERSTATIC MODELLING - FOUNDATION DISPLACEMENT**		
CONSULTANT :	**CIGRÉ**		

A.1 – TEST PROCEDURE

TEST 1	STRUCTURE 2	FOUNDATION DISPLACEMENT

The Foundation on leg No: 1 of the structure was raised in small increments from 0 to 25mm above ground level, and then, adjusted in the opposite direction to 25mm below ground level, and back to ground level. Then, the joints were loosened and tightened.
Throughout the whole test, the Strain Gages measured the strain in the structure members, which can be identified by the drawing below:



A.2 – FOUNDATION MOVEMENT AJUSTABLE DEVICE

ADJUSTABLE FOOT USED DURING THE FOUNDATION DISPLACEMENT TEST

A.3 – TEST RESULTS

FOUNDATION UPLIFT TESTS

(mm)	F2B	P3L	F4	T12L	T13L	T12T	B11L	T11L	B11T	T11T
0	0	0	0	0	0	0	0	0	0	0
3	-9074	1610	-11171	559	541	497	-1072	-904	893	-1741
3.5	-11925	2052	-14574	626	702	472	-1393	-1157	1173	-1779
5	-18182	2546	-23384	131	1233	306	-2295	-1334	1753	-1421
6.5	-24329	3058	-31272	-211	858	328	-3136	-1666	2286	-675
8	-28277	3360	-36534	-296	854	272	-3749	-1881	2539	-534
10	-36239	3781	-47509	-858	351	-31	-4962	-2175	2977	908
11.5	-37091	3761	-49737	-959	306	-146	-5110	-2324	2997	1435
12.5	-38633	3888	-51974	-877	326	-284	-5360	-2524	3078	1349
14	-40866	4492	-55298	-744	396	-407	-5684	-2673	3205	1512
14.5	-42317	4715	-57421	-772	568	-478	-5872	-2836	3392	1643
15	-42950	5010	-58969	-708	860	-482	-5998	-3429	3349	1474
17	-44123	5929	-60553	-977	819	-314	-5971	-2685	3110	1612
18.5	-48683	7508	-66819	-1364	699	-125	-5816	-1797	2429	1255
19.5	-50191	7891	-69236	-1437	443	-3	-5446	-1614	2948	978
20.5	-53564	9412	-73895	-1708	108	221	-5493	-1445	3103	506
22	-54641	11037	-76256	-1861	-105	759	-5138	-1315	3953	206
23	-55486	11542	-77379	-1889	-79	846	-4986	-1400	4045	113
24	-56930	12422	-79731	-1888	-257	1228	-4403	-1009	4563	-138
25	-58747	13624	-82703	-1989	-393	1270	-4066	-658	4947	-492
24	-49585	10205	-70949	-1125	440	363	-3306	-1661	5528	470
23	-43890	7314	-63691	-702	933	216	-3408	-2425	6311	1343
21.5	-37411	3989	-55335	-168	1267	-346	-3074	-2909	6991	2373
20.5	-31651	977	-48066	292	1751	-865	-2746	-3444	7488	3022
18	-21063	-2094	-34551	1249	2622	-1755	-2334	-4461	8259	4098
15.5	-10763	-3970	-20673	2187	3442	-2691	-2315	-5539	8561	5398
14	-5158	-4377	-13287	2554	3737	-2915	-2445	-6134	8344	6032
13	1988	-5174	-3427	3212	4151	-3391	-2572	-6884	8378	6764
10	9346	-5799	5815	3870	4489	-3762	-2440	-7834	8819	7574
8	16310	-6057	14821	4533	5119	-4282	-2694	-8910	9356	8323
6	22879	-6151	23102	5053	5527	-4627	-3021	-9775	10043	9024
3.5	30764	-6608	34303	5738	6334	-5068	-3489	-10869	9821	10020
1.5	37094	-6915	42821	6263	6896	-5483	-3676	-11575	9908	10769
0	39234	-6904	46353	6397	7142	-5690	-3854	-11781	8949	10983

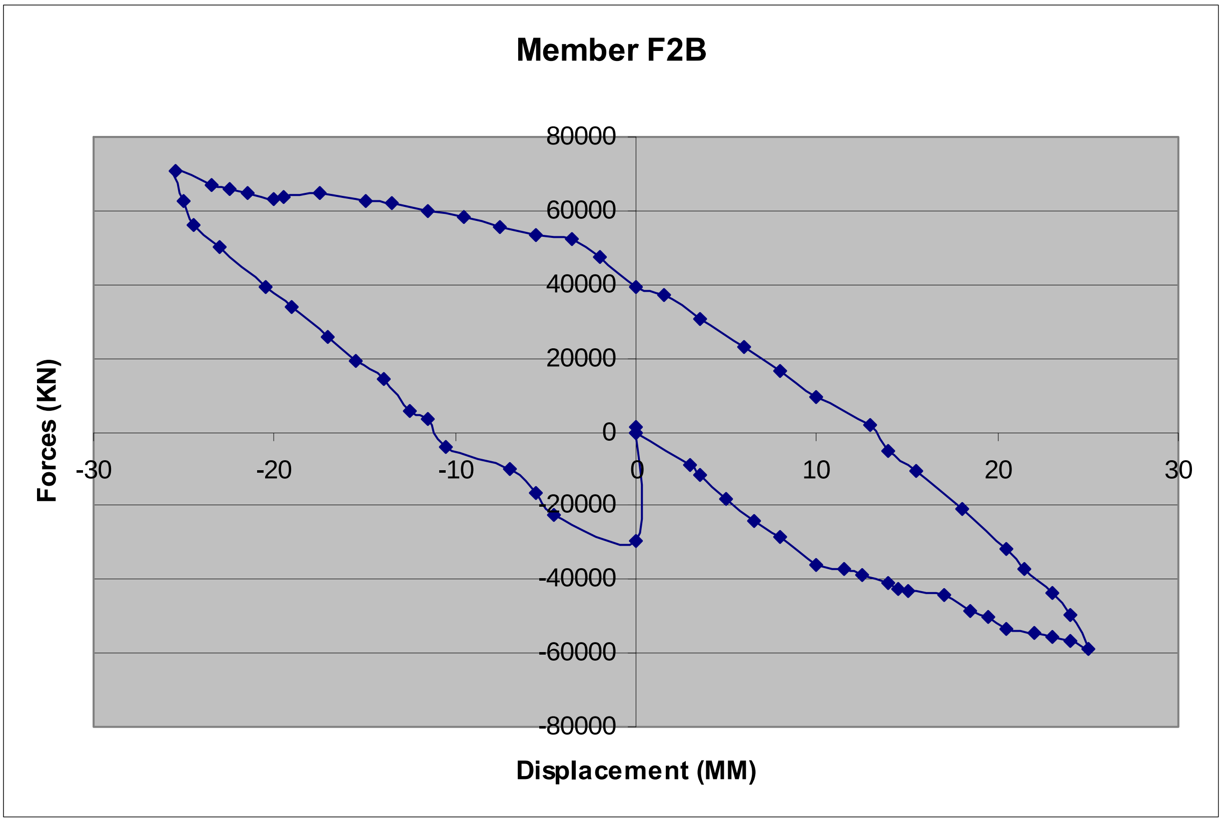
Table A.3.1 - Displacement (mm) *versus* Bar Loads (KN)

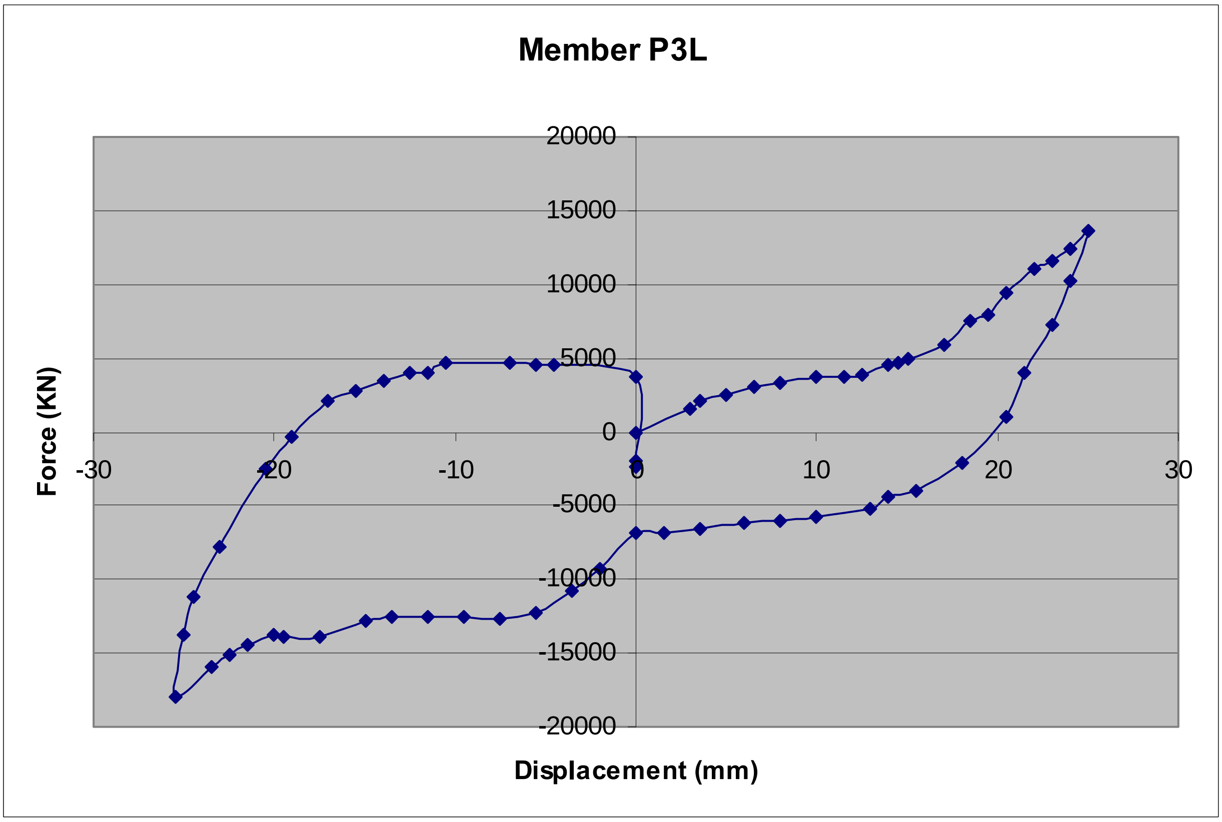
FOUNDATION SETTLEMENT TESTS

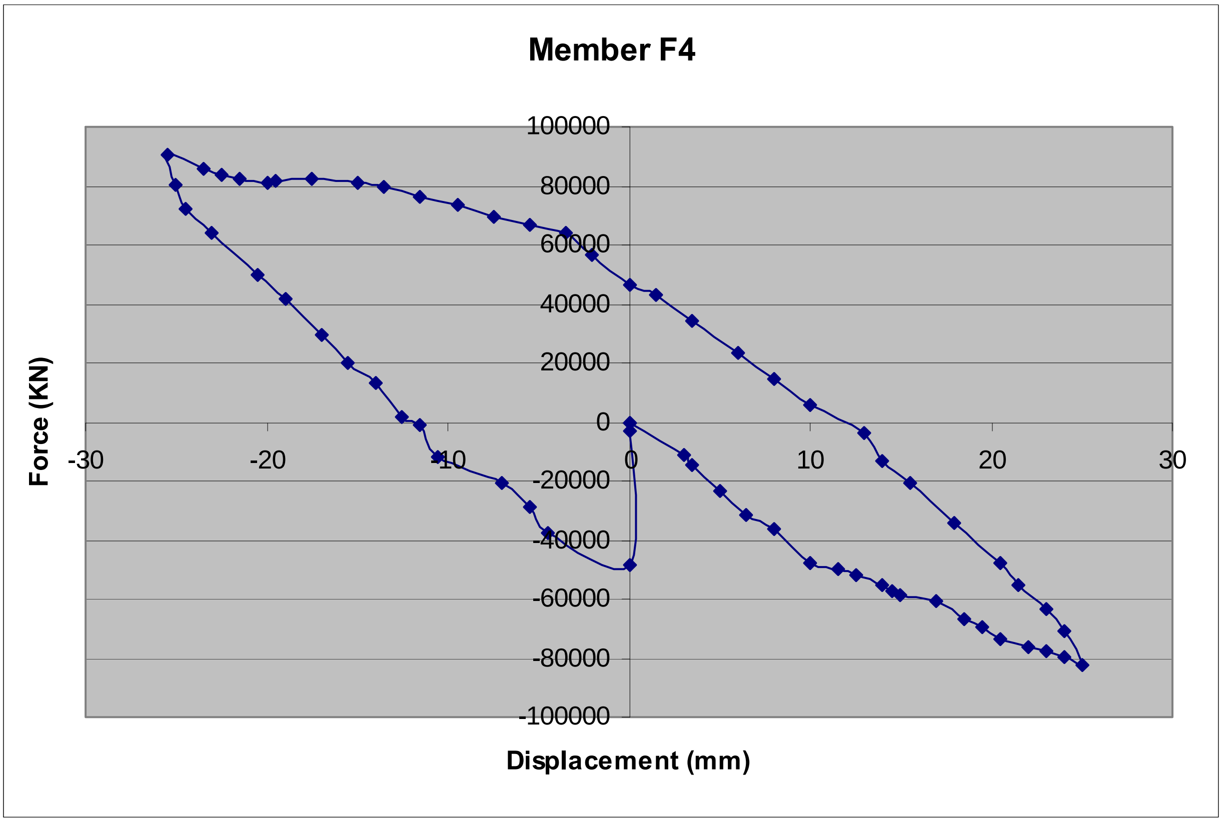
(mm)	F2B	P3L	F4	T12L	T13L	T12T	B11L	T11L	B11T	T11T
0	39234	-6904	46353	6397	7142	-5690	-3854	-11781	8949	10983
-2	47282	-9288	56558	7464	7611	-6226	-3374	-12770	9765	11358
-3.5	52464	-10733	63855	7943	8170	-6623	-3684	-13536	9238	11890
-5.5	53619	-12237	66670	7883	8314	-6710	-3981	-13483	7720	12612
-7.5	55491	-12741	69563	7971	8500	-6918	-4249	-13697	6930	13071
-9.5	58159	-12481	73793	8270	8791	-7169	-4591	-13851	6898	13550
-11.5	59920	-12568	76525	8164	9226	-7256	-4960	-14220	6698	13903
-13.5	61978	-12525	79781	8118	9634	-7420	-5675	-14504	6194	14273
-15	62787	-12803	81002	8164	9723	-7455	-5646	-14573	6101	14427
-17.5	64576	-13962	82679	8487	9730	-7324	-4936	-15043	7416	14580
-19.5	63678	-13835	81819	8413	9745	-7273	-5129	-14949	7836	14507
-20	62933	-13725	80785	8385	9390	-7152	-4847	-14743	8126	14498
-21.5	64624	-14403	82464	8524	9640	-7203	-4588	-15041	8600	14466
-22.5	65850	-15097	83833	8616	9746	-7229	-4576	-15186	8873	14628
-23.5	66991	-15938	86081	8800	9881	-7316	-4769	-15331	9254	14829
-25.5	70554	-17971	90845	9132	10184	-7489	-4522	-15989	10348	15183
-25	62506	-13773	80534	8466	9575	-6857	-4756	-15009	9924	14215
-24.5	56246	-11217	72327	8130	9115	-6303	-4758	-14269	9729	13394
-23	50064	-7818	64157	7611	8531	-5741	-4945	-13365	9183	12741
-20.5	39582	-2441	49895	6538	7815	-4954	-5041	-12109	8185	11398
-19	33991	-275	41725	6111	7409	-4470	-4460	-11420	7915	10661
-17	25873	2039	29647	5472	6761	-3864	-3482	-10595	7356	9502
-15.5	19460	2749	20325	4864	6383	-3336	-2977	-9943	7176	8606
-14	14215	3434	13259	4341	6071	-2947	-2803	-9208	6936	7940
-12.5	5590	3962	1389	3489	5592	-2307	-2441	-8337	6885	7029
-11.5	3718	3962	-1132	3296	5528	-2203	-2410	-8099	6765	6713
-10.5	-4103	4654	-11931	2550	5099	-1581	-2094	-7335	6374	5686
-7	-10263	4706	-20673	1787	4867	-1199	-1988	-6782	5911	4831
-5.5	-16290	4550	-29141	1171	4560	-836	-1912	-6217	6052	4168
-4.5	-22563	4557	-37856	724	4110	-412	-1808	-5557	6783	3358
0	-29739	3728	-48167	218	3465	159	-1682	-4837	8209	2466
0	1298	-2005	-575	1836	2014	-3147	-9998	-7956	1454	6260
0	-201	-2395	-2841	1776	1797	-2749	-9412	-8049	3061	6175

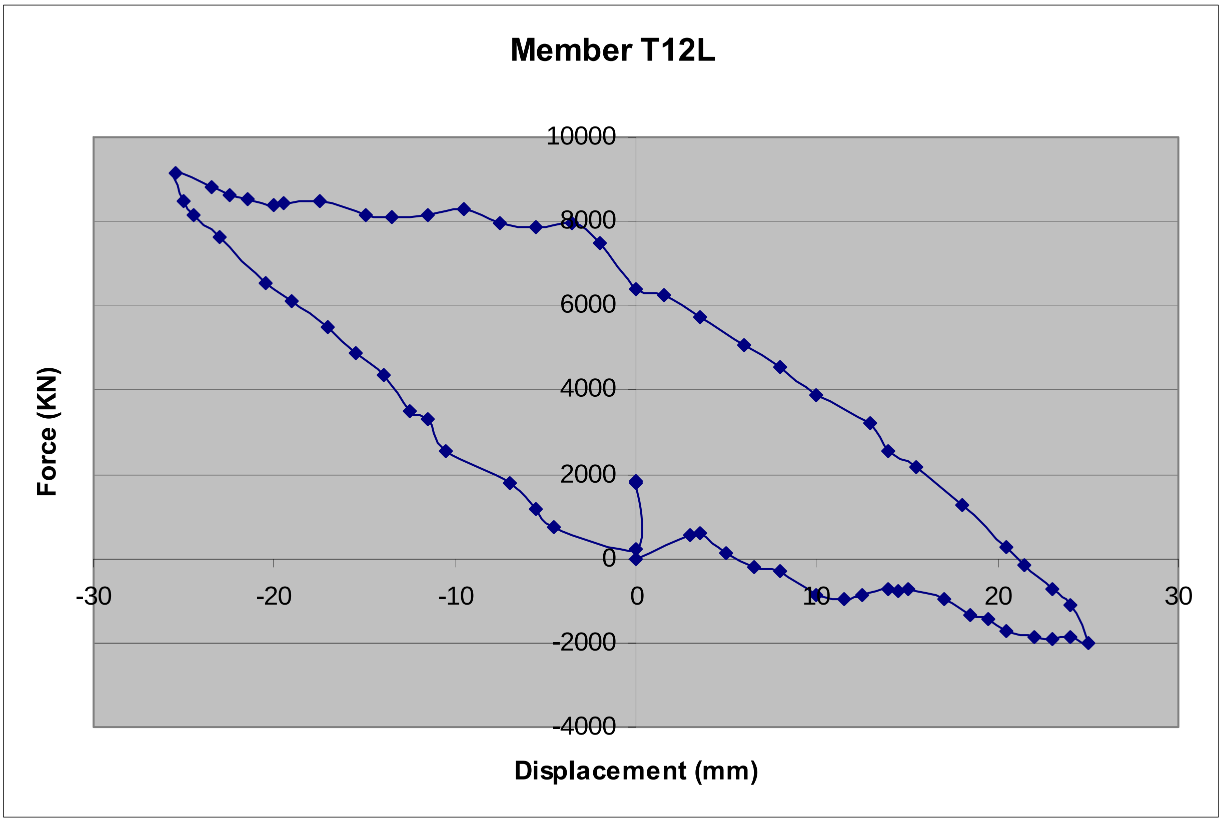
Table A.3.2 – Displacements (mm) *versus* Bars Loads (KN)

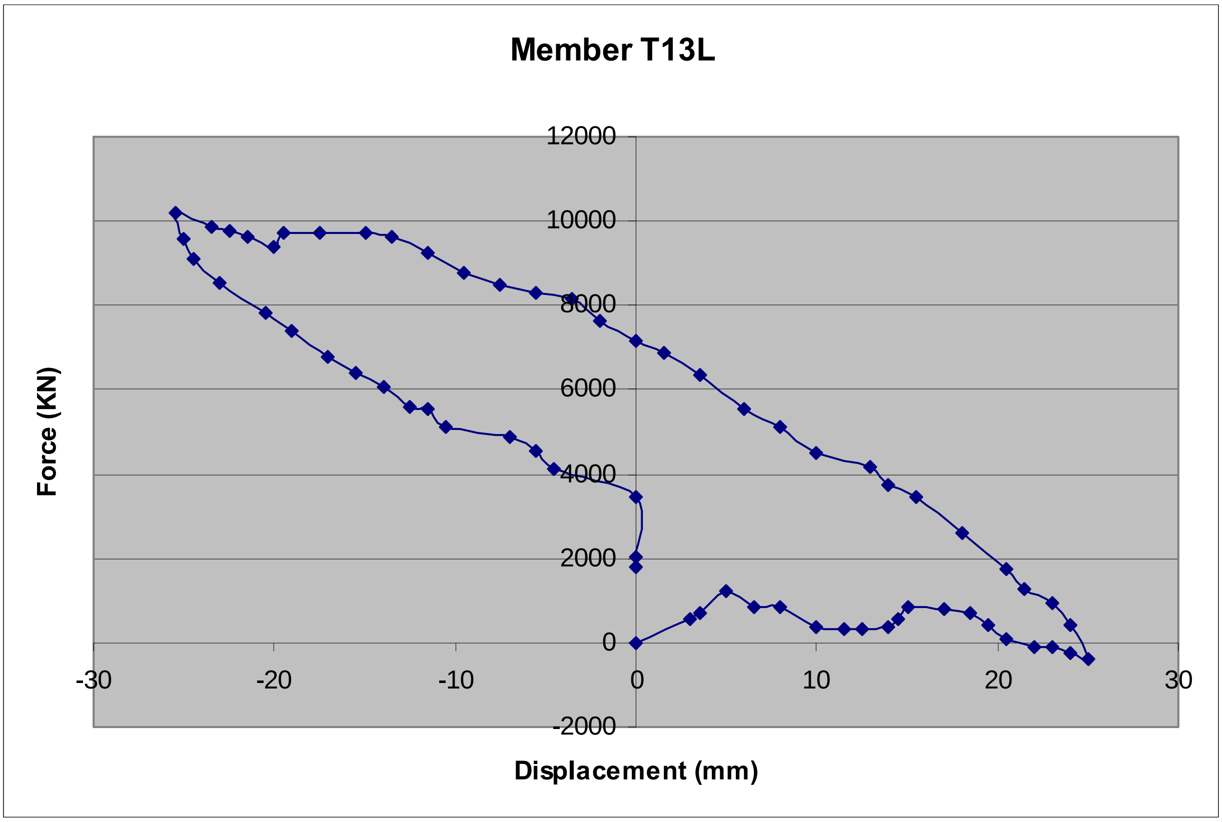
A.4 – BAR LOAD *versus* DISPLACEMENT GRAPHS

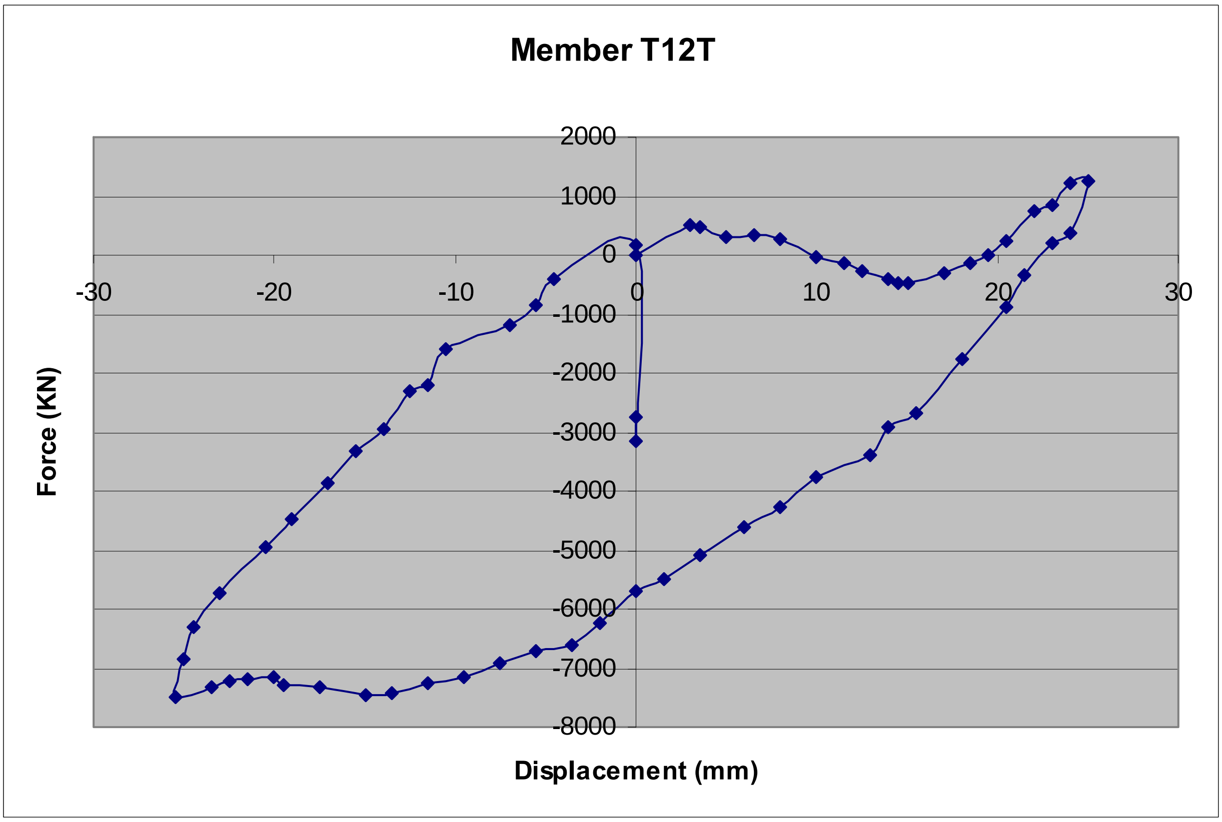


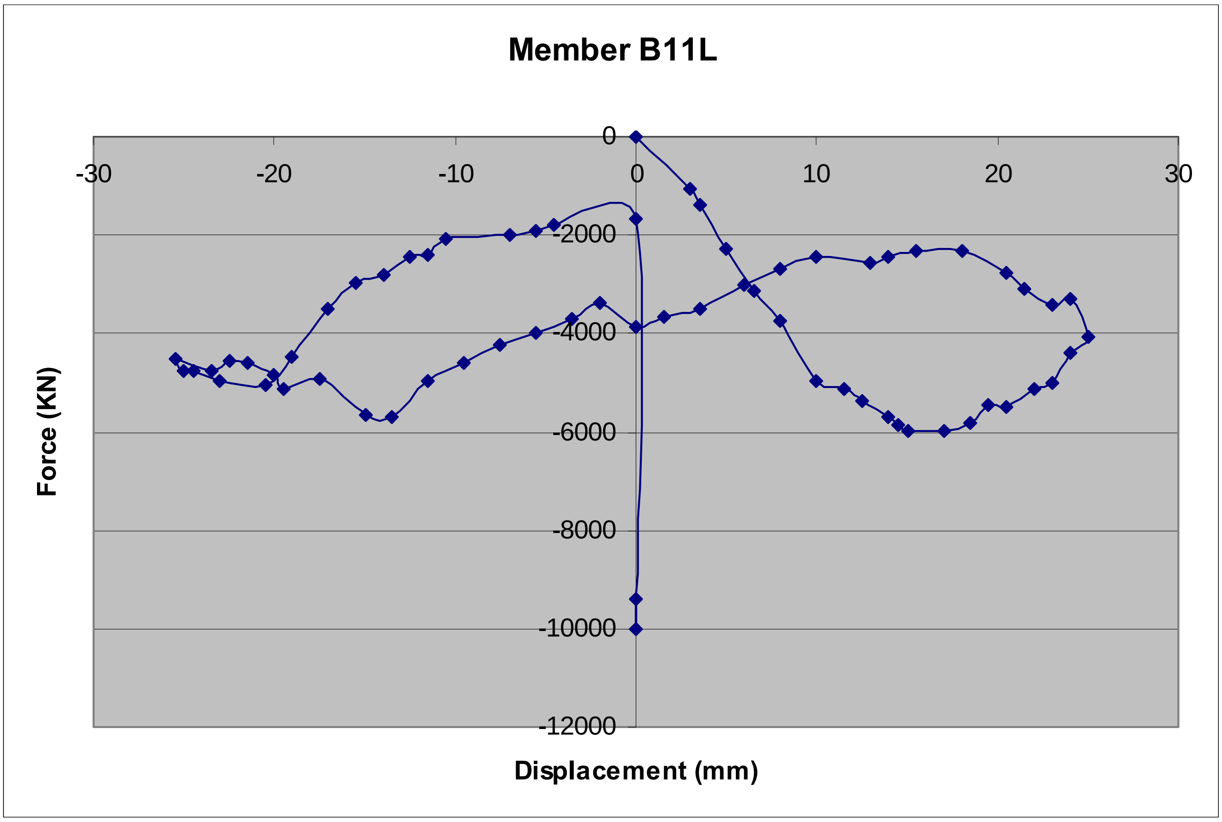


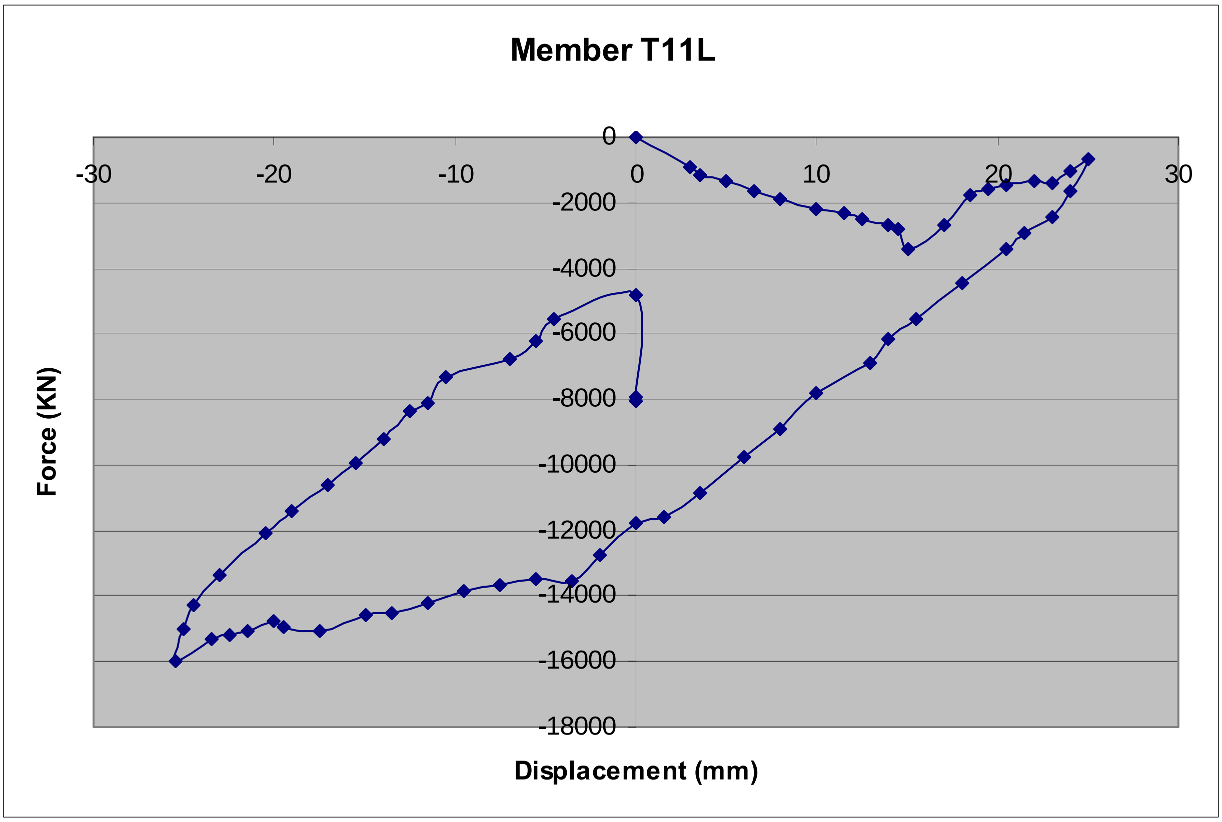

Member F4
100000
80000
60000
40000
20000
0
-20000
-40000
-60000
-80000
-100000
-30
-20
-10
0
10
20
30
Force (KN)
Displacement (mm)

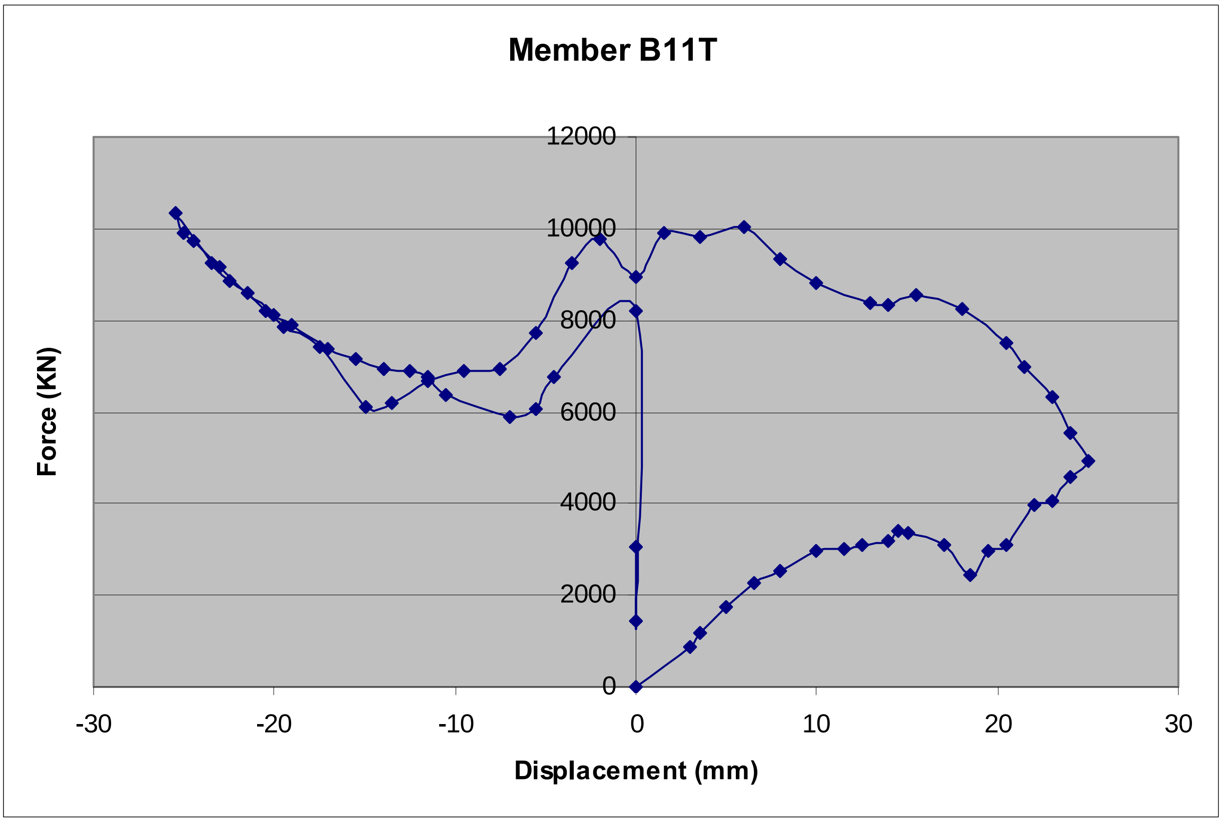

Member T12L
10000
8000
6000
4000
2000
0
-2000
-4000
-30
-20
-10
0
10
20
30
Force (KN)
Displacement (mm)

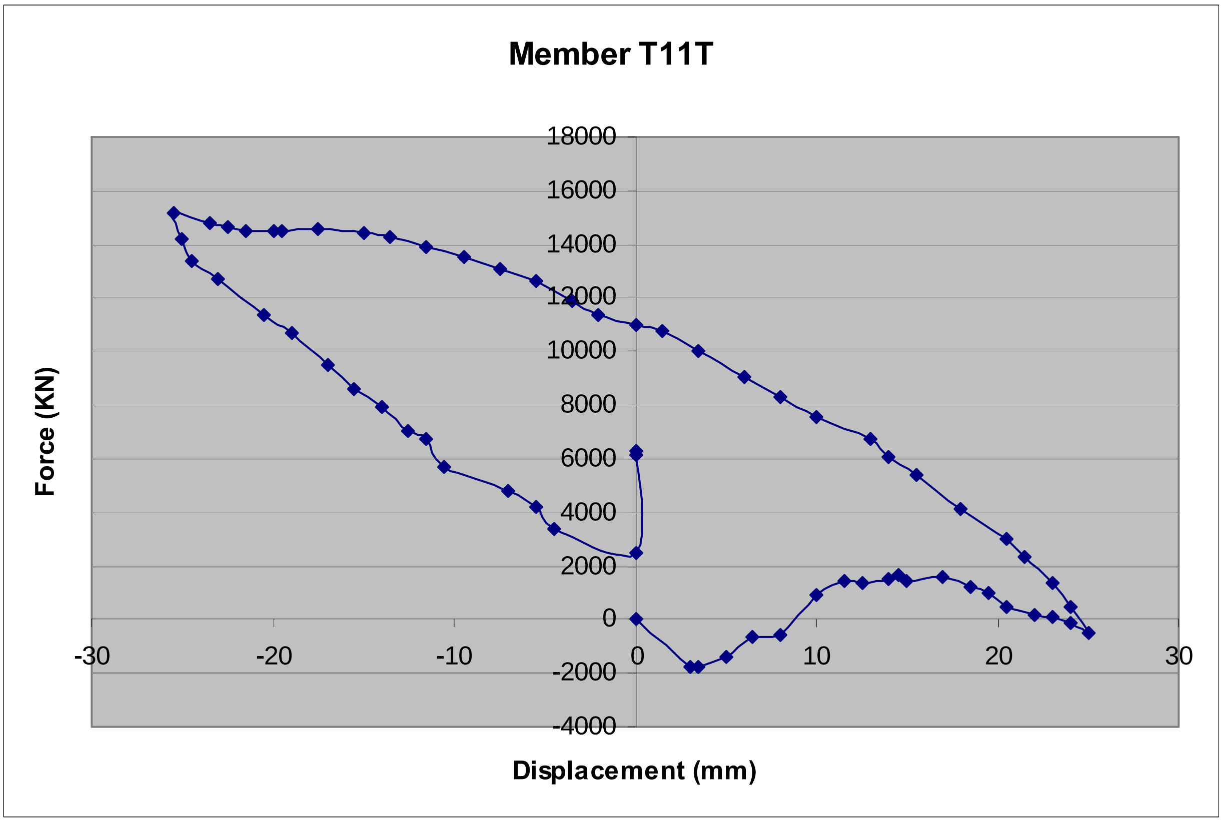

Member T13L
12000
10000
8000
6000
4000
2000
0
-2000
-30
-20
-10
0
10
20
30
Force (KN)
Displacement (mm)


Member T12T
2000
1000
0
-1000
-2000
-3000
-4000
-5000
-6000
-7000
-8000
-30
-20
-10
0
10
20
30
Force (KN)
Displacement (mm)


Member B11L
0
-2000
-4000
-6000
-8000
-10000
-12000
-30
-20
-10
0
10
20
30
Force (KN)
Displacement (mm)


Member T11L
0
-2000
-4000
-6000
-8000
-10000
-12000
-14000
-16000
-18000
-30
-20
-10
0
10
20
30
Force (KN)
Displacement (mm)


Member B11T
Force (KN)
12000
10000
8000
6000
4000
2000
0
-30
-20
-10
0
10
20
30
Displacement (mm)


Member T11T
Force (KN)
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
-2000
-4000
-30
-20
-10
0
10
20
30
Displacement (mm)

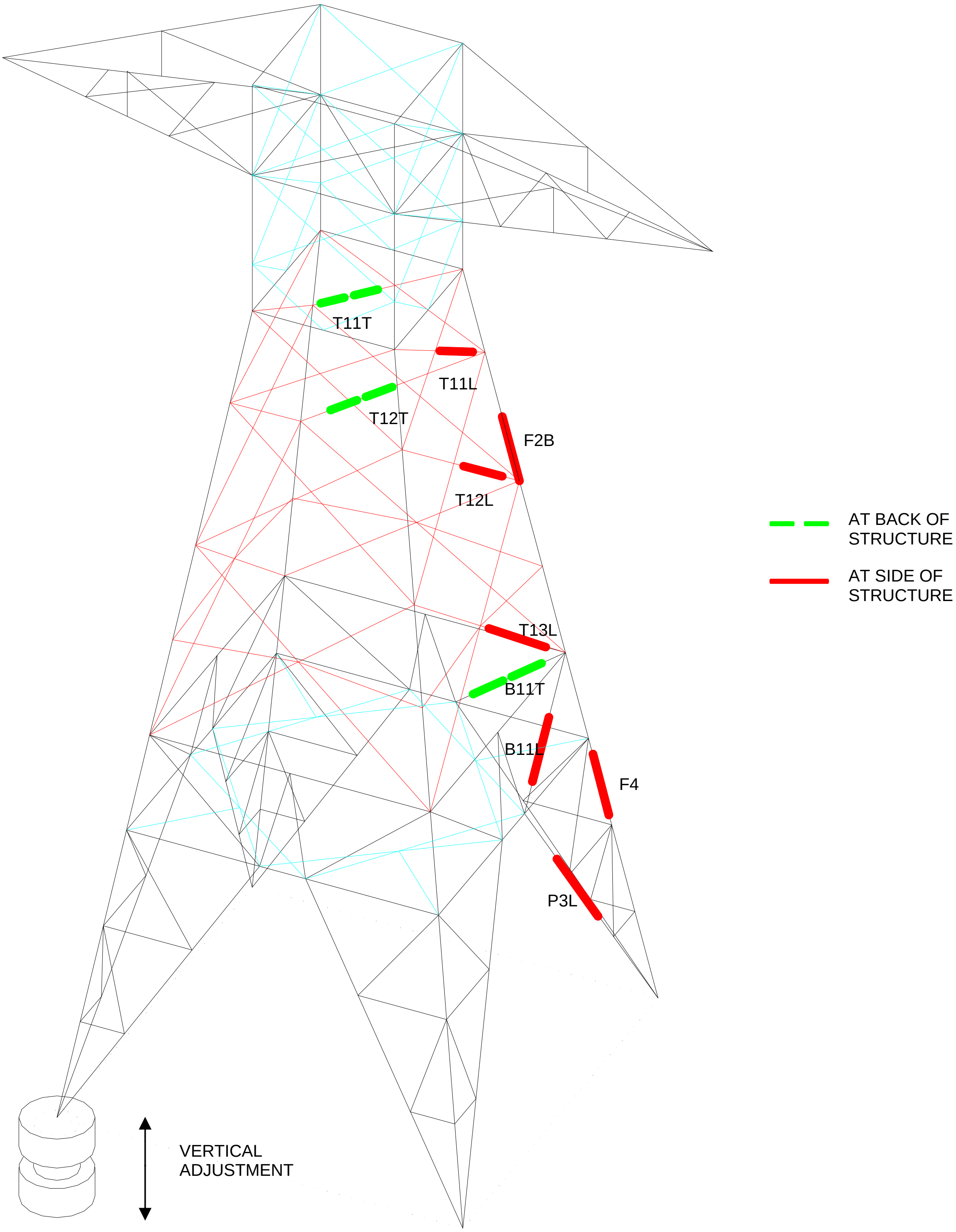
FOUNDATION UPLIFT TESTS

ANNEX B:

ESTIMATED *versus* MEASURED LOADS ON MONITORED BARS

B.1 - MONITORED BARS

The bars that were strain gaged monitored during the foundation displacement tests can be seen in the following drawing:



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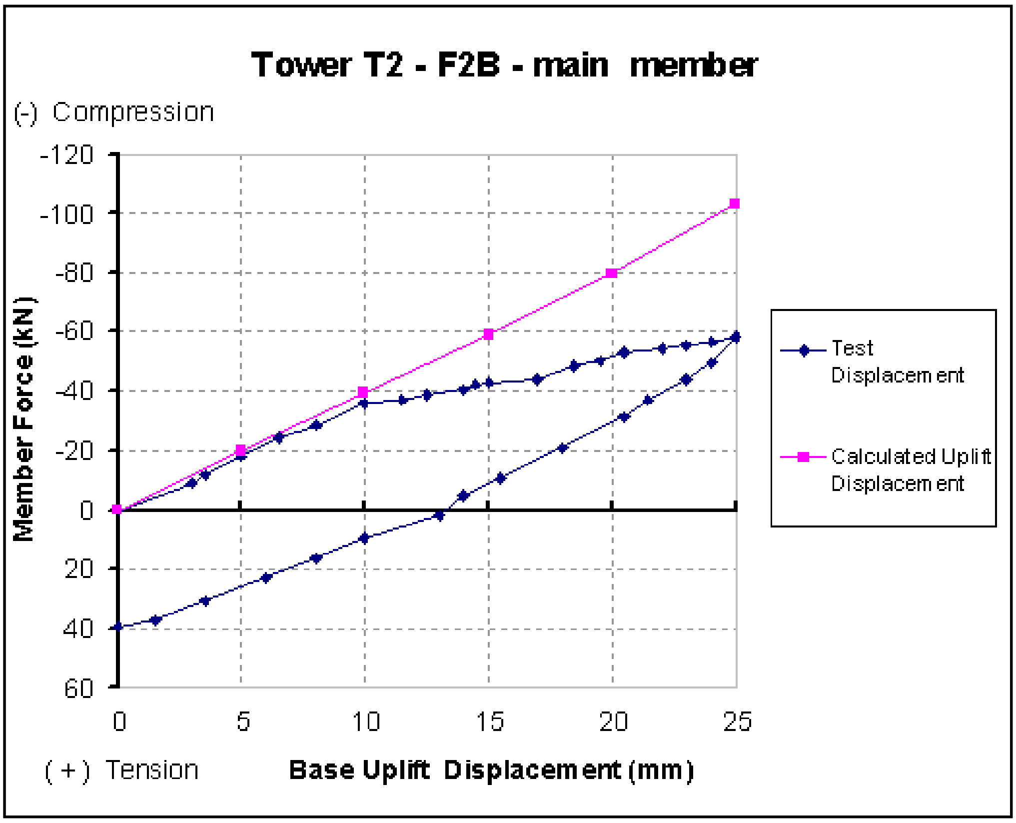
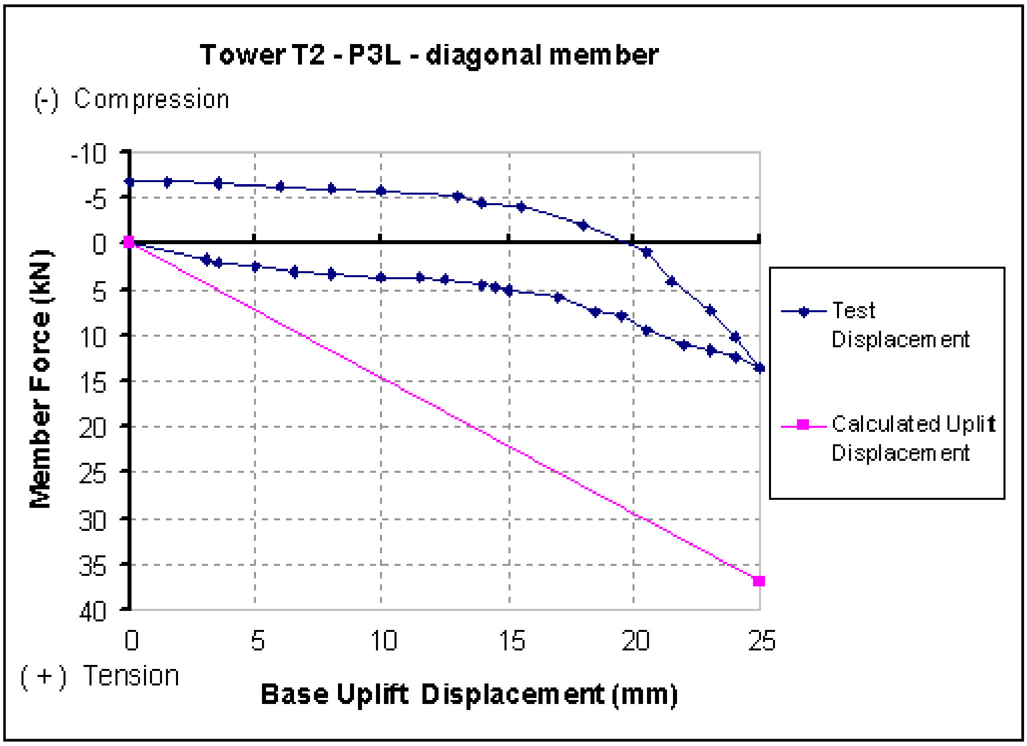
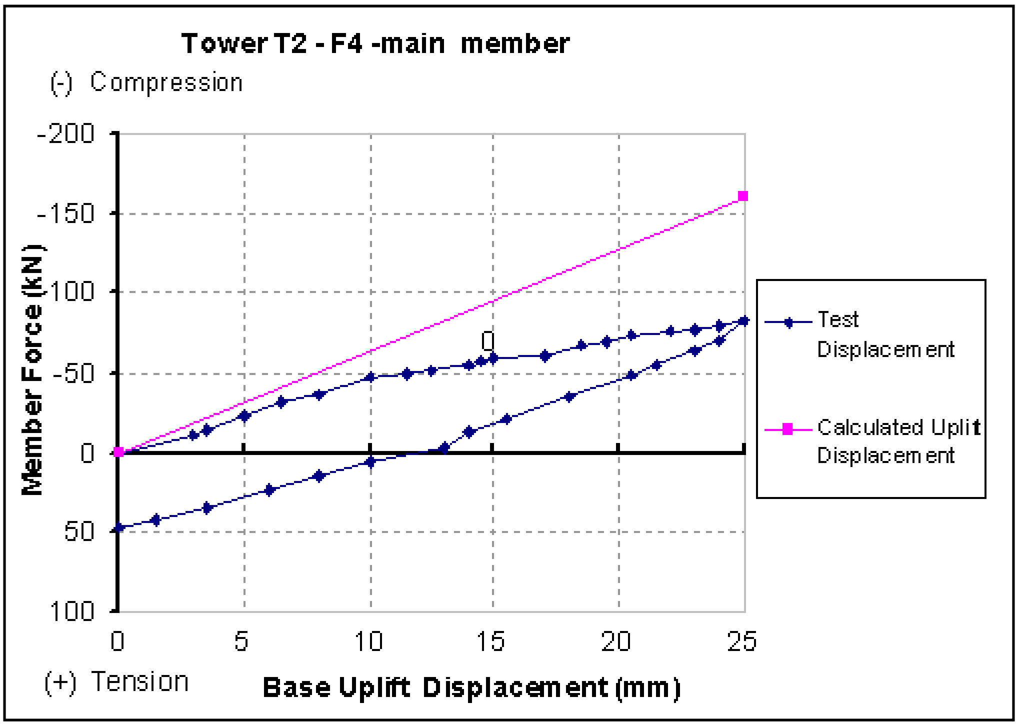
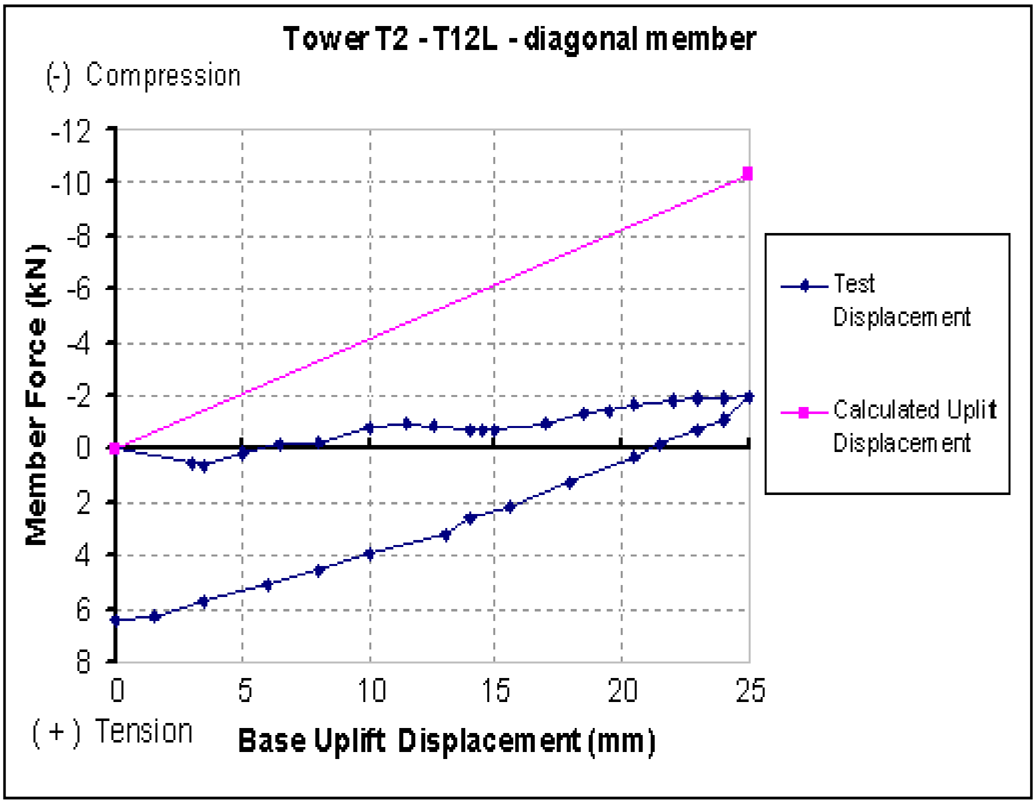
B.2 – ESTIMATED *versus* MEASURED LOADS

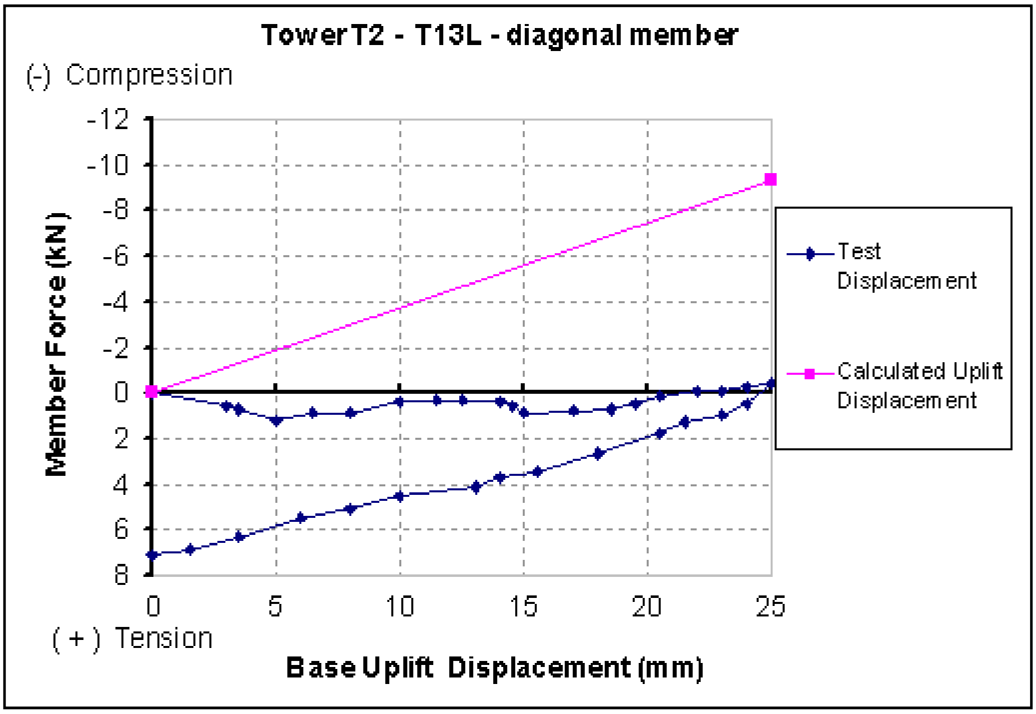
UPLIFT DISPLACEMENT TEST

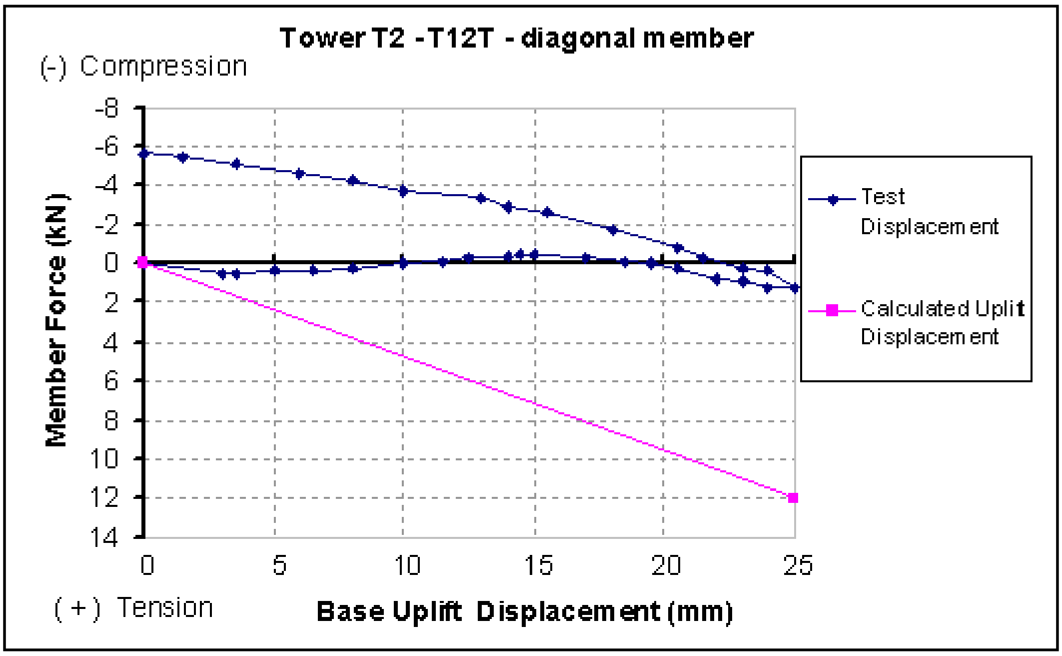


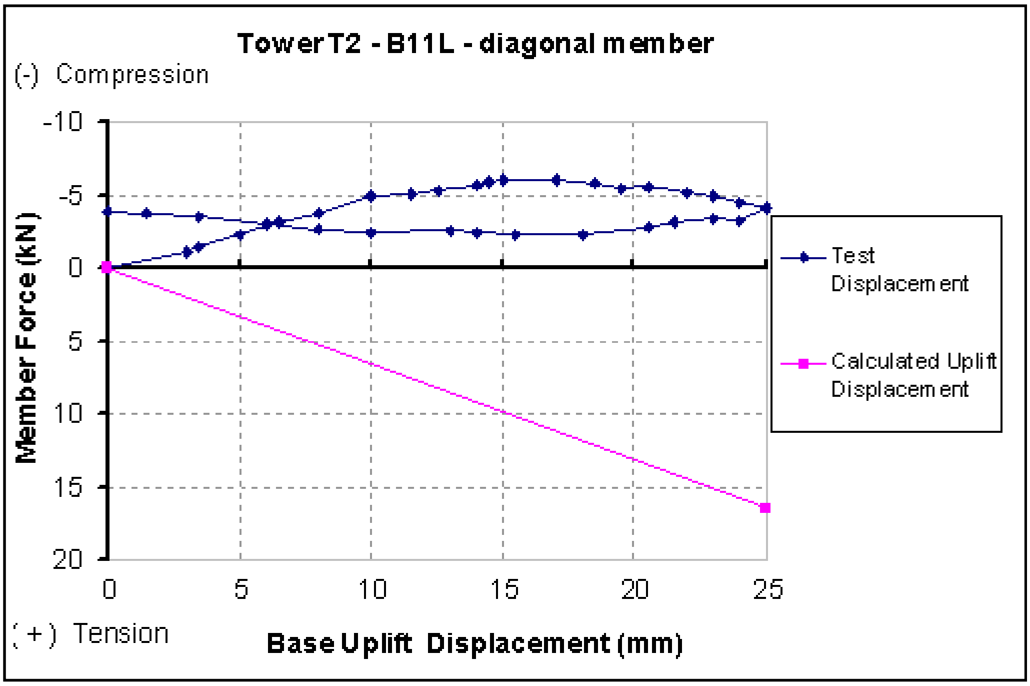


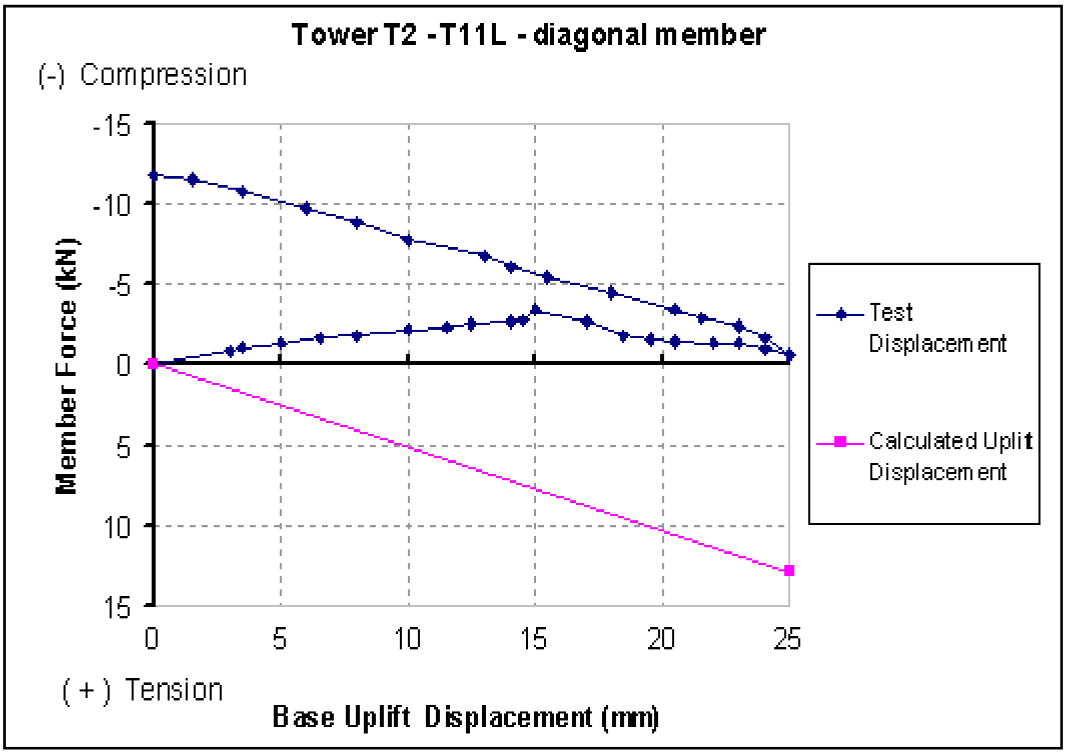

Tower T2 - F4 -main member
(-) Compression
-200
-150
-100
-50
0
50
100
0
5
10
15
20
25
(+) Tension
Base Uplift Displacement (mm)
Member Force (kN)
Test Displacement
Calculated Uplit Displacement

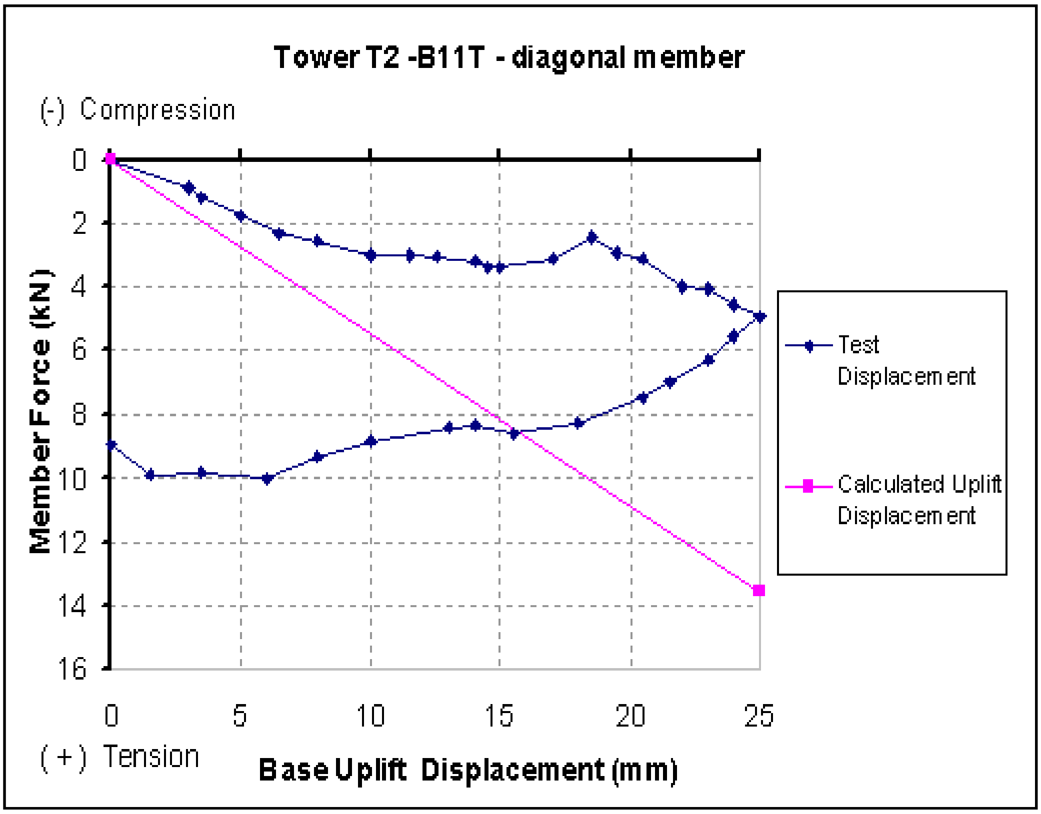

Tower T2 - T12L - diagonal member
(-) Compression
-12
-10
-8
-6
-4
-2
0
2
4
6
8
0
5
10
15
20
25
(+) Tension
Base Uplift Displacement (mm)
Member Force (kN)
Test Displacement
Calculated Uplit Displacement

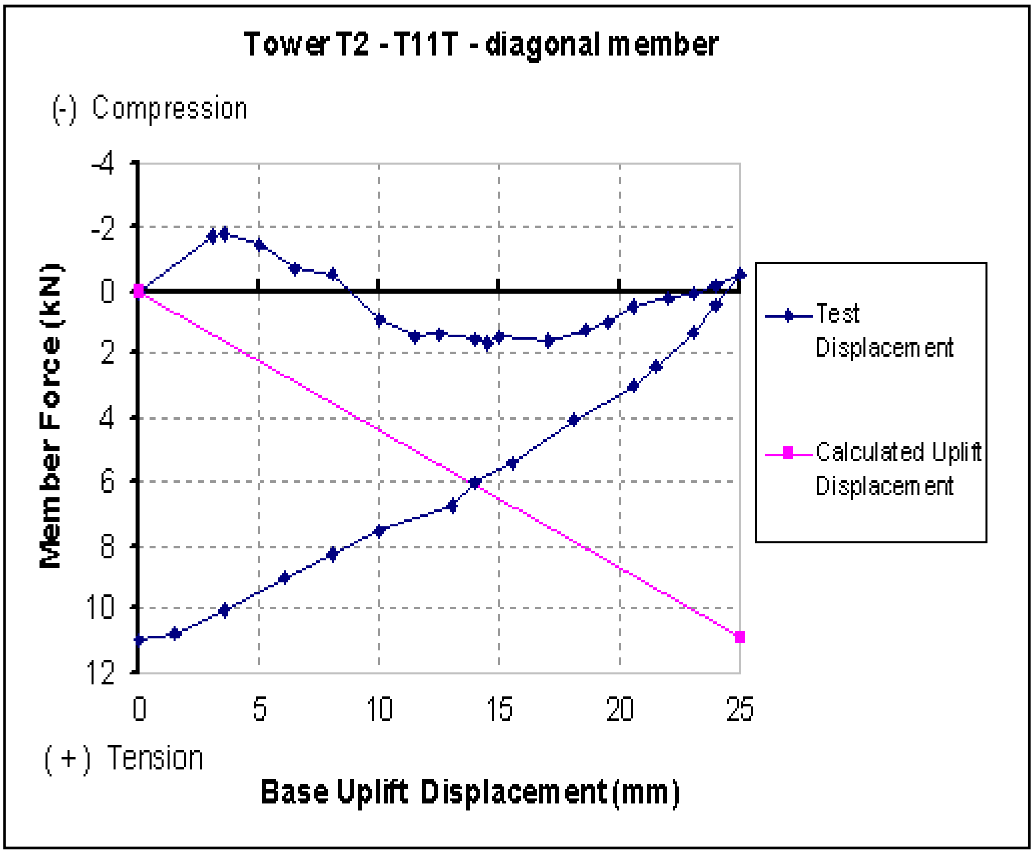

Tower T2 - T13L - diagonal member
(-) Compression
Member Force (kN)
-12
-10
-8
-6
-4
-2
0
2
4
6
8
0
5
10
15
20
25
(+) Tension
Base Uplift Displacement (mm)
Test Displacement
Calculated Uplift Displacement


Tower T2 -T12T - diagonal member
(-) Compression
Member Force (kN)
-8
-6
-4
-2
0
2
4
6
8
10
12
14
0
5
10
15
20
25
(+) Tension
Base Uplift Displacement (mm)
Test Displacement
Calculated Uplift Displacement


Tower T2 - B11L - diagonal member
(-) Compression
-10
-5
0
5
10
15
20
Member Force (kN)
0
5
10
15
20
25
(+) Tension
Base Uplift Displacement (mm)
Test Displacement
Calculated Uplift Displacement


Tower T2 -T11L - diagonal member
(-) Compression
-15
-10
-5
0
5
10
15
Member Force (kN)
0
5
10
15
20
25
(+) Tension
Base Uplift Displacement (mm)
Test Displacement
Calculated Uplift Displacement


Tower T2 -B11T - diagonal member
(-) Compression
Member Force (kN)
0
2
4
6
8
10
12
14
16
0
5
10
15
20
25
(+) Tension
Base Uplift Displacement (mm)
Test Displacement
Calculated Uplift Displacement


Tower T2 - T11T - diagonal member
(-) Compression
Member Force (kN)
-4
-2
0
2
4
6
8
10
12
0
5
10
15
20
25
(+) Tension
Base Uplift Displacement (mm)
Test Displacement
Calculated Uplift Displacement