1 COVERING LETTER

2 TECHNICAL REPORT

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3 COMPUTER ANALYSIS REPORT

4 COMPUTER ANALYSIS DRAWINGS

5 UNIVERSITY TEST SAMPLE RESULTS

6 TENSION & TWIST DATA SPREADSHEETS

7 CALIBRATION CERTIFICATE FOR DILLON

8 STICK DRAWINGS OF 625FT MAST

9 LAYOUT DRAWINGS

10 PHOTOGRAPHS

TABA

16 May 1993

Your ref: DTCG43-93-C-H9ZE43

Our ref: TA 300C

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COMMANDER
USCG Activities Europe
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Dear Sir

Re: <u>Kargaburun Mast investigation</u>

We have pleasure in offering to you our report on the collapsed LORAN C mast radiator at Kargaburun, Turkey.

The report contains the following sections of information:

Section 1.	Covering letter
Section 2.	Technical report on the mast incorporating the computer analysis information
Section 3.	Computer analysis report for both masts, compiled by THE FANTOZZI COMPANY of San Jose
Section 4.	Diagrams from the computer analysis
Section 5.	Test samples report
Section 6.	Tension and twist data spreadsheets
Section 7.	Calibration test certificate for Dillon
Section 8.	Stick drawings for the mast showing the fall sequence
Section 9.	Site layout drawings
Section 10.	Site photographs

You will see from the technical comments that even after exhaustive computer analyses it is not possible to identify the actual element of the mast that failed first. We are as you will see able to demonstrate in which span the point of failure occurred and the reasons why it failed where it did.

The TACO mast that you have in store has also been assessed and it has been found to be superior in design and construction to the original Stainless mast.

However, in defence of the original mast, it was not possible to design to the same standards as a modern design due to the amount of assumptions that had to be made in the 50s and 60s as computers were not available to tackle the mathematical problems encountered. We also now have significantly more

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data available to assist designers than was available when the original mast was built. The actual fabrication quality of the Stainless mast was very good and the internal galvanising of the leg sections excellent.

You will see the four most important things that together most likely conspired to cause the collapse of the mast, given the information available, were:

1. *The original structure was marginal in the form of a top loaded mast radiator when considering ice or snow*
2. *The foundation locations for the guy anchors and TLE anchors were incorrect*
3. *Maintenance of the mast did not take (2) into account and also did not adequately ensure that the mast was plumb, without twist and with equal tensions on the torque stabiliser guys. When these guys are not equal tensions in a pair they will actually induce torque in the structure.*
4. *The heavy snow load on all the cables, which in this instance appears to have been the "coup de grace"*

As we have mentioned in the report, we do strenuously advise the USCG to have all the remaining guyed masts inspected without delay. TABA Ltd will be pleased to quote you for undertaking this work. This would basically take two forms:

1. *Site surveys to establish the current state of the system*
2. *Computer analyses of the structures to identify any design limitations*
3. *Recommendations for each site with details of what is needed to prolong the life of the structures*
4. *Quotations for undertaking the work*

TABA will also be pleased to quote to the USCG for drawing up superior maintenance schedules for your systems, including designing a more rigid and all embracing document for the inspectors. We would place a great deal of emphasis on having a professional Structural Engineer being responsible for checking that:

1. *The inspectors are suitably qualified to undertake the work and have all the relevant data on the structure available to them at the time of the inspection*
2. *The results of the inspection are vetted to ensure that anomalies are picked up and rectified*
3. *A full set of documents for the structure is held on the site and updated as to the work done on the structure*

We will be pleased to discuss any of these aspects with you

Yours faithfully

Heath Hollinsworth
Managing Director

Section 2

2.1 TECHNICAL REPORT ON THE STRUCTURAL COLLAPSE

CLIENT:	*United States Coast Guard (Activities Europe)*
SITE:	*LORSTA Kargaburun (7990 Yankee)* *Turkish Army Depot* *Marmara Sea* *TURKEY*
STRUCTURE:	*625' LORAN C Mast; Navigation Beacon.* *Triangular 5' Face Mast (Tube Construction)* *Messrs. Stainless Inc. PENNSYLVANIA USA*
TOPOGRAPHY:	*Coastal flat terrain. Gently sloped site (NE - SW) 1 in 50. Mast base & TX building on levelled ground in the middle of the site within 1200' of the sea.*

2.2 EVENTS

2.2.1 *25/02/1993 - Total collapse of mast at 18.33 (Local time)*

2.2.2 *11/03 to 15/03/93 - Site survey (TABA Ltd.) to collect & evaluate all necessary data in order to determine cause & nature of collapse.*

2.2.2.1 *Measure fall pattern (essential to determination of collapse sequence.)*

2.2.2.2 *Photograph as necessary to create permanent record for investigation.*

2.2.2.3 *Obtain relevant samples for testing purposes.*

2.2.2.4 *Relocate all the sections to alongside the access road to the Tx room to facilitate further inspection of what was the underside*

2.2.2.5 *Collaborate with USCG inspection team to ensure that both parties are in possession of all material & documented evidence.*

2.3 DOCUMENTATION issued by the USCG

2.3.1 During site survey

2.3.1.1 Local Met. data : Abnormally heavy snowfall following cold spell. 400 kg /m³ wet snow in 24 hours. Snow accretion possible during calm period (14 mph NE wind).

2.3.1.2 Site plan: Original surveyors layout. (Corps of Engineers)

2.3.1.3 Stainless manual: Report # 1100.

2.3.1.4 Equipment: Dillon & Calliper.

2.3.2 Subsequent Communications received

2.3.2.1 Inspection reports: 1982 onwards (not all reports are comprehensive)

2.3.2.2 Maintenance Manual

2.3.2.3 USCG internal investigation documents and photos.

2.3.2.4 Witness statements

2.3.2.5 TACO Manual.

2.3.2.6 Video Cassettes.

2.4 OBSERVATIONS MADE ON SITE

2.4.1 Civil damage minimal, mainly to Tx building roof.

2.4.2 Structure appears to have been materially sound prior to event. Few signs of corrosion, although paint not in prime condition. Internal galvanising of tubes appeared to be very good.

2.4.3 Stays & TLEs display basketing at the mast connections but no visible deterioration, other than event failure. Some "LAPP" insulators failed on impact with the ground. The base insulator was badly damaged due to the collapse.

2.4.4 Slight damage to mast-head as a result of cushioned landing.

2.4.5 Massive damage at two locations.

2.4.6 Signs that tower had folded during collapse, "U" bolt marks on mast legs.

2.4.7 *Paint undisturbed on all turn-buckles.*

2.4.8 *Fall pattern consistent with buckling failure of spine.*

2.5 PROBABLE CAUSES OF THE FAILURE

From detailed analysis of data, collected & observed, a series of drawings have been produced by TABA Ltd.in order to present the facts. These drawings reflect the results of field studies, computer analysis and mechanical tests.

The list of relevant drawings and sketches is given below:

2.5.1	14 stick sketches:	900-039	Sheets 1 - 14
2.5.2	G.A. of mast:	900-035	Sheet 1
2.5.3	Stay and TLE orientation:	900-035	Sheet 2
2.5.4	Anchor block, reduced levels:	900-035	Sheet 3
2.5.5	Stay/TLE break up pattern:	900-035	Sheet 4
2.5.6	Details of mast sections:	900-035	Sheet 5
2.5.7	Layout of collapsed mast:	900-035	Sheet 6
2.5.8	Temp bench mark orientation:	900-035	Sheet 7
2.5.9	Spread sheet:	TENS300.WK3	
2.5.10	University test results:	93/929	
2.5.11	Computer Analysis report		
2.5.12	Equipment test certificates		

The following resume covers aspects relating to the suitability of the structure in respect of function and design concept.

2.6 Manufacturer's Tolerances

2.6.1 *These values were grossly exceeded from construction. (It may be interesting to correlate the values of alignment with resect to seasonal temperatures and wind directions.)* ?

2.6.2 *Values for tensions are set in order to calculate differential radii in relation to ground slope. Alternatively allowances are acceptable in stay tensions values if the stay radii are not reset to counteract the grade variations*

2.7 Design Standards

2.7.1 *The mast would not have conformed to the current practises or standards of design and construction, (see computer analysis which shows that the design was marginal)*

2.7.2 *Poor maintenance of spine compression and stability through mal-adjusted and unbalanced tensions would not have assisted in these short-comings (see spreadsheet of tension and twist data)* ?

2.7.3 *It has not yet been determined whether or not stress relief for welds connecting leg to flange or gusset to leg was carried out at the fabrication stage*

2.8 RECOMMENDATIONS

2.8.1 *Management of Inspection procedures*

2.8.1.1 *The process of correlating inspection data must be systematic and afford the compiler an explicit overall picture of the state of the structure under scrutiny. This data together with all necessary equipment certificates, site datum and fixed points of reference must always be available on site.*

2.8.1.2 *Details of material specifications and drawings of the structure and its components must be held by the owner and should have been noted by any inspector.*

2.8.1.3 *The inspection staff must be aware of any facts that merit special attention, in particular, modifications that infer that a structure may not be capable of supporting normal loading if specifically reduced tolerances are exceeded.*

2.8.1.4 *If possible, meteorological data for the site should be recorded on a regular basis. The life expectancy for a structure may span periods of climatic change.*

2.8.1.5 *Due accord must be given to the man-power necessary to inspect & maintain a structure of this complexity. One man crews may be seen as an economical proposition but the time span involved in proper maintenance would be prohibitive.*

2.8.1.6 *We would recommend that the code of practice used by the USCG be updated and that the format for the inspection reports be redesigned. The USCG inspectors should then be made fully aware of the implications of their tasks and be fully cognisant of the requirements for the mechanical and electrical stability of the structures.*

2.8.1.7 *WE STRONGLY RECOMMEND THAT ALL OF THE USCG GUYED MASTS STILL STANDING THROUGHOUT THE WORLD, BE THOROUGHLY INSPECTED TO ENSURE THAT THERE ARE NO OTHER CANDIDATES FOR THIS (or any other) TYPE OF COLLAPSE.*

2.8.1.8 ***(TABA Ltd will be pleased to quote the USCG for advice on or actually mounting a professional inspection schedule to accommodate the remaining structures).***

2.9 STOCK REPLACEMENT MAST

2.9.1 This structure is far superior in latent strength to the Stainless design (see Computer Analysis report).

2.9.2 Recommendations for its use depend on the quality of control exercised in its storage.

2.9.3 Due care will need to be accorded on site when choosing sites for the replacement of the stay and TLE anchors.

2.9.4 An evaluation of its performance against current codes is given in the Computer Analysis report.

2.10 NOTES

The lack of certain substantiated documentation has lead to the assumptions that are the basis of this report. (Note that any subsequent revelations may justify further investigation and contradictory conclusions may then be drawn)

The above statement is deemed necessary subject to the provision of a full report history of the structure. The information contained in maintenance reports prior to 1982, was not in TABA's possession until very recently and due to other committments we were not able to incorporate the data. This data, particularly that which is relevant to excessive twist and alignment inherited during construction is comparable in importance with a need to include mid-span readings as well as those at stay levels.

It is interesting to note that the USCG Tower Manual actually specifies that mid-span readings be documented along with those at the guy levels. You will see from the "stick sketches" that considerable alignment deviations will be noted if the guy tensions are incorrect, for whatever reason.

We suspect that the Stainless type # 1100 structure, modified to carry TLE,s, was operating near its functional limit under normal conditions. Failure to adhere strictly to the manufacturer's tolerances exposes a major weakness in the policy of adopting a general code, namely the Inspection Manual, in assessing the adequacy of specific structures. All quoted values for twist etc under calm conditions for this structure are deemed excessive.

W.E.Jackson
C.Eng. M.I.Struct.E.

H. Hollinsworth
Managing Director

Section 3

COMPUTER ANALYSIS REPORT

3.1 ANALYSIS OF EXISTING MAST

The nonlinear static analysis utilized for the investigation of the collapse of the 625ft LORSTA mast considers geometry and material nonlinearity. The geometrically non linear formulations utilized are total lagrangian and updated lagrangian method. The material nonlinearity uses nonlinear elastic curve description. The algorithms utilized are standard incremental-iteration. Structural loading is described by a time history in a time like variable called "pseudo-time" related to load magnitude. The variation with pseudo-time of the node concentrated forces, element pressures, nodal temperature gradients and specified acceleration are specified as part of the analysis option and member force data.

Component elements of the structure are modelled as geometrically nonlinear-linear elastic elements or geometrically nonlinear with material nonlinearity depending on the member application. The geometrically nonlinear analysis procedure employed for these elements is standard total lagrangian techniques. This nonlinear formulation considers large displacements, large rotations and large strains. The variation of the element cross-sectional area with load is considered. The coordinate system for the model considers alternate local coordinate systems for simplified input of coordinate data.

The computer model of the existing mast uses 3D truss elements to represent the mast legs, diagonal braces and horizontal girts. Tower members are grouped according to span levels. Although the tower legs are continuous through panel joints, experience has shown that the truss element represents the behaviour of the legs with sufficient accuracy and provides substantial savings in computational effort. The 3D truss element is a general 3D element which is designed to carry only axial loads.

3D beam elements are used for the analysis of the umbrella beams supporting the TLEs. The 3D element provides bending stiffness and is capable of resisting applied loads along its length.

Mast stays and TLEs are represented as geometrically nonlinear, elastic 3D truss elements. Each cable is segmented into a series of truss elements representing the stay of TLE. Each cable forms and element group. A preprocessor is utilized to generate the cable profile under initial tensions using the following catenary equation.

$$L^2 = V^2 + \frac{H^2 \, Sinh^2(a)}{a^2} \quad \text{where:}$$

L = actual cable length
V = projection of the cable in the direction Y_1
H = projection of the cable in the direction X_1

$$a = \frac{W_1 \, H}{2Fh}$$

W_1 *= cable weight per unit*
Fh = horizontal component of the cable tension

An iterative scheme is used to solve for the roots of the catenary equation. The preprocessor provides profile coordinates in the local cable axis along its length. Any number of points can be output to accurately describe the profile of the cable. For the purpose of this investigation, coordinates are specified at insulator locations and at one or two points between insulators, depending on insulator spacings. Coordinates along the ACSR radiators are specified at five points between the tower and the Lapp insulator. These coordinates are transformed to the model coordinate system. Initial strains are included in the stay and TLE formulations to incorporate the pretension into the cable under initial conditions. The pretensions used in the analysis where the tensions specified in the 22nd May 1992 tower inspection report by PENN-TECH. These tensions were recognised to be the best estimate of the initial cable tensions at the time of the collapse. However these tensions may be somewhat low for the actual tensions because of the lower temperatures at the time of the collapse.

ACTEUR Inspection

For the "initial condition state", the tower is acted upon only by its own weight and cable pretensions. The tower legs and horizontal girts were formulated as geometrically linear elements to conserve computer time. The diagonal braces were formulated as elastic nonlinear elements with tension yielding and compression limiting stress-strain curve. Due to the elastic shortening of the tower, the diagonal members go into compression under initial conditions. The compression stress-strain curve for the diagonals limits the compression load to the Eular buckling load (Pe). This allows the investigation into the effects of the initial twist and "out of plumb" *condition reported in the 22nd May 1992 tower inspection report.*

The Eular buckling load was considered to be the maximum sustainable compression load for any member. An effective length of 0.9L, where L is the distance along the diagonal from face of leg to face of leg, is used to represent the partial fixity of the connection. The yield stress of the material is the limiting tensile capacity of the member.

For increasing load states, the tower legs and horizontal girts were formulated as elastically linear members with compression limiting stress-strain curves based on the Euler buckling load using a K factor of 0.9 for the girts. The effective length factors used in the analysis are based on generally accepted K factors for bracings. Although more complex formulations for effective length factors have been presented in the literature, all are predicated on a number of assumptions that must be fully realised in the design of the structure. The uncertainties in assessing the actual fabrication of the existing mast makes a more detailed evaluation of the effective length unwarranted and the effective length factors used in the analyses are sufficiently accurate to predict the collapse mechanism.

For the tower legs, the limiting compression load was determined using the critical buckling formulation for buckling of continuous beams on elastic supports. The horizontal girts act to restrain the tower legs at regular spacings. From the theory of elastic stability, the variation of the critical load is approximately in the same proportion as the rigidity of the support. When the rigidity of the supports is small, the deflection curve of the buckled leg has no inflection points. For greater rigidity of the supports, an inflection point occurs at the middle of the leg. When the stiffness of the supports approaches the magnitude given by the equation mP/L, the critical load approaches the Euler load times the number of spans.

Where:

m = number of spans of length L/m
P = mP<u>e</u>, with P<u>e</u> the Eular load for a column length L

For any span on the Stainless tower, the rigidity of the girts were found to behave absolutely rigid and the critical load for the tower legs equals m^2EI/L^2. The girt attachment points became inflection points. Thus the modelling of the legs as 3D truss elements is satisfactory.

The limiting stress-strain formulations for individual members enable us to look at the local behaviour of the individual elements of the tower and allow for force redistribution in the limit state. Force redistributions are caused by the nonlinear behaviour of the members in an increasing load collapse analysis. The force redistribution may cause members to exhibit nonlinear behaviour and yield in the case of a tension member, or buckle in the case of a compression member. However, because of strain hardening, a yield tension member can typically absorb additional force, whereas a compression member resists decreasing force for increasing shortening after reaching its buckling force. Thus, a compression member cannot resist the additional force but has to shed force and cause additional force redistributions into other members. Under increasing load, the failure mode is demonstrated by large displacements at the formation of a collapse mechanism.

3.2 RESULTS - INITIAL CONDITIONS

Fig 1 shows the 625ft LORSTA mast under initial pretensions. The tower is a Stainless Type G5, modified to accept the top radials by replacing the standard horizontal girts at the top of the tower with the radial umbrella support beam. The tower model geometry is based on the original Stainless installation drawings covered under report #1100. Material properties for the EHS guys and ACSR radiators were obtained from standard handbooks on these materials. Tower material properties were obtained from the manufacturer and reflect the yield strengths from the physical and chemical reports from the producers.

The model geometry reflects the site topography as depicted on the general site plan for TACK II Kargaburun, dated 1st September 1959 by the Corp of Engineers. The base of the tower at the top of the base insulator was taken as datum. Cable anchor points were based on a linear interpolation between contours. The stay anchor points and TLE anchor points were located 18" and 6" respectively above grade.

✱ *Analysis of the tower for initial pretensions under calm conditions shows that the tower was twisted and "out of plumb" prior to the collapse. The results of the analysis are summarised in table 1. The calculated vertical mast load at the base under initial conditions is 81,000 lbs.*

✱ I do not concur.

TABLE 1. Tower alignment @ initial conditions

Level	Twist (deg)	Displacement (in)	Azimuth (deg ETN)
1	0.30 ccw	0.178	274
2	0.16 ccw	0.097	316
3	0.88 ccw	0.452	308
4	0.68 cw	0.409 ≈ .5"	71
5	1.44 ccw	0.874	356
6	2.45 cw	1.480 ≈ .6"	12

>2.12° ccw

Does not agree w/ report.

Fatal flaw in analysis

Table 1 shows the effects of the unbalanced pretensions caused by the grade variations at the site. Table 1 reflects the equilibrium position of the mast under initial conditions with the guy and radial anchor point locations determined from the topographical information. Values in Table 1 refer to the model geometry. There is no direct correlation between the values in Table 1 and the reported twist and alignment data. Table 1 is based on an equilibrium position using a perfectly straight mast as datum. The alignment data is a set of averages for alignment and twist based on the positional placement of the measuring instrument. (Theoretically, the relative direction of twist and displacement should be the same as reported, if measured correctly).

FIG 2 shows a view of the upper section of the tower. The displaced shape is shown dashed and clearly shows the initial displacements and twist.

The initial twist and deflection was caused by the uneven tensioning in the cables and the grade variation at the site. Although the tensions measured in the tower inspection report were within the manufacturers tolerances, the grade variation was not accounted for. Typically grade variations are accounted for during installation by moving the anchor block along the axis of the cable to maintain the same vertical angle. If the ground elevation at the work point of a particular guy is higher than the ground elevation at the tower base, the anchor is moved toward the tower. Conversely, if the guy anchor is than the tower base, the guy anchor is moved away from the tower.

By not making the proper adjustments to the anchor locations to take into account grade variations, the guy sets and radials altered the resultant forces on the mast. The uphill cables at the existing anchor locations have a smaller vertical angle than assumed in the design and thus impart a greater horizontal force on the tower. The opposite downhill cables have a larger vertical angle and impart a smaller horizontal force on the tower. In addition, the larger vertical angle of the downhill stays puts a greater vertical load on the tower. Thus, the *tower must move to remain in equilibrium under the initial tensions.*

From the alignment results, it is seen that the uphill stays and TLE's have displaced the mast in a northerly direction. The variations in tension and slope have combined to torque the mast at levels 4, 5 and 6 where the twist in the tower changes direction.

3.3 COLLAPSE ANALYSIS

The collapse of the mast was initiated by the accretion of wet snow on the cable members. Observers on the site reported a snow build up on the stays of up to 1-1/2" in effective radial thickness prior to the collapse of the mast. Successive analyses were done, increasing the weight density of the stays and the TLE's to account for the effect of increasing snow accretion. It was assumed that a correlation exists between the occurrence of glaze ice and that of wet snow. It is recognised that both types of ice form under different ambient air temperature conditions and wind speeds. However, the other icing paraoeters are comparable. A good fit has been reported between a series of observed snow events and calculated snow accretions if a density of 25 lbs/ft^3 and a collection efficiency of unity are assumed. Since the collection efficiency of ice is typically equivalent to unity, the equivalent radial snow thickness can be determined from the standard equation for glaze ice adjusted for the difference in densities.

$Ws = 0.5454\,[(d)Is + Is^2]$ where:

Ws = weight per foot of wet snow (pounds per foot)
Is = radial thickness of wet snow (inches)
d = bare diameter of cable (inches)

Therefore the effective cable density used in the analyses is the weight of the bare cable plus the weight of the wet snow divided by the actual cable diameter.

The collapse of the LORSTA mast was initiated by torsional buckling in span 5 (section between guy levels 4 and 5) at about the 470ft level of the mast.

Fig 3 shows the failure mechanism at span 5.
Fig 4 shows the displaced shape of the upper sections of the mast.

The results of the analysis of the mast under 1-1/2" snow load are summarised in Table 2. It is seen from Table 2 that the mast was severely torqued at level 4. Under normal wind loads, the LORSTA mast could tolerate the built in twist and "out of plumb" alignment caused by the grade variations and uneven tensions. However, the misalignment combined with the heavy snow loads caused the mast to buckle torsionally.

TABLE 2. Tower alignment under 1-1/2" snow load

Level	Twist (deg)	Displacement (in)	Azimuth (deg ETN)
1	0.30 ccw	0.183	312
2	0.44 cw	0.266	12
3	1.29 ccw	0.779	354
4	1.48 cw	0.896	26
5	1.81 ccw	1.096	315
6	1.66 cw	1.007	320

83.29°

If we now compare Tables 1 and 2, it is seen that the mast has moved in the general direction of the longer cables (downhill side). This is to be expected since the weight of these cables will be greater due to a greater accretion of snow. In addition, the twist and displacement at the top of the mast is less under snow load than at initial conditions but has more than doubled at level 4. However, the relative displacement between level 4 and level 6 remains about the same. This suggests that the failure mechanism was not the classical buckling phenomena but rather occurred as a result of local instability. The calculated vertical load at the base of the mast under 1-1/2" snow load is 214,200 lbs.

Typically, for pin based masts, the fundamental mode of instability is approximately a single half wave. However, the TLE's provide sufficient stiffness to effectively "pin" the top of the mast. Generally, the fundamental mode of a top loaded mast becomes a number of half waves equal to the number of spans of the mast. The strength of the mast subjected to uniform axial compression is reached at the tangent-modulus Euler load for the mast section. However, buckling can occur in a torsional mode under axial compression while the longitudinal axis remains relatively straight.

When torsional buckling occurs, the critical load is smaller than either the Euler load or the purely torsional buckling load. When the initial eccentricity from the unbalanced pretensions is considered, the critical load is reduced further.

Under compression, the legs buckled in span 5, by rotating about the longitudinal axis of the mast. As snow accumulated on the cables, the vertical load on the tower increased reaching its critical value, so that each leg had a slightly deflected form of equilibrium. Because of the deflection, bending stresses occurred and were superimposed on the initial uniformly distributed compressive stresses. At the same time, the initial compressive stresses now act on slightly rotated cross sections. From Tables 1 and 2, it is seen that the relative twist between levels 4 and 5 (span 5) increased about 55% under snow loads. Although the percentage increase in twist may have been greater at other spans, the combined effect of increased twist, axial load and displacement was critical at span 5.

Δ 1.17°

3.4 ANALYSIS OF THE TACO REPLACEMENT MAST

A Type TACO guyed triangular mast as manufactured by Technical Appliance Corp, was analyzed in accordance with the provisions of EIA/TIA Standard ANSI/EIA/TIA- 222-E and BSI CP3 for a design wind speed of 90 mph, as mentioned in the TACO specifications. In order to verify the original design, the analysis assumes a level site and uniform tensions at each guy level.

The mast model geometry is based on the TACO drawings and specifications for a 625 ft Type 6000-S mast. Physical properties of the "Alumoweld" guys and radiators were obtained from Copperweld Steel Company. Material properties for the mast were assumed to be to ASTM A441 Grade 50 steel. This steel grade was however not verified. If the physical properties of the TACO mast in the stores is different than assumed for the purposes of this analysis, the results and any conclusions derived from this analysis are subject to change.

Based on our analysis, the TACO 625 ft 6000-S mast is in conformance with the design provisions of ANSI/EIA/TIA-222-E and BSI CP3 for a basic design windspeed of 90 mph without ice (snow) loading, and 30 mph wind with 1/2" radial ice (1-1/2" snow). Furthermore, the torsional rigidity of the TACO mast was found to be 55 percent greater than the original Stainless mast. This occurs because the lateral torsional rigidity of a mast is a function of the face width, bracing stiffness, leg cross sectional area and panel height. Torsional rigidity is directly proportional to the stiffness of the bracing and *to the square of the mast face width. Furthermore, the torsional rigidity is improved by the ratio of the total cross sectional area of the legs over the effective bracing area. Torsional stiffness is inversely proportional to the panel height,(angle of inclination of the bracing). The TACO mast has a lower angle of inclination of the bracing, thus a shorter panel height, solid leg members and larger bracing members. Based on these findings, the TACO mast will be able to withstand a 1-1/2" snow load under calm conditions and assuming no twist or "out of plumb conditions", without collapse. In addition, the higher initial pretensions improve the initial stiffness of the guy sets, significantly reducing the deflections and increasing the resistance to instability.*

In order to install the TACO tower on the existing site, the stay tensions must be adjusted to reflect the change in grade across the site. Ideally, the grade variations would be accommodated by moving the anchor blocks, as previously described, and as delineated in the TACO mast erection and maintenance manual. However, the cost of moving the anchors in accordance with the manufacturer's recommendations may be prohibitive.

The following formula may be used to adjust the stay tensions shown on the installation drawings, to fit the existing grades and anchor locations. The formula is based on calm conditions and at an ambient temperature of 40 degrees F.

$$Tn = Ts \frac{\cos a}{\cos b} \quad \text{where:}$$

$$\tan b = \frac{V - (Dv)}{H} \qquad \tan a = \frac{V}{H}$$

Tn = new pretension
Ts = specified pretension
V = vertical distance above the mast base of the guy level for which the new pretensions are desired
H = horizontal distance from mast centre line to guy anchor point
Dv = the elevation differential between the ground elevation at the mast base (W.P.O.) and the anchor point (positive if above the W.P.O.)

The anchor point elevations should be verified before applying the adjustments.

Proper pretensioning of the guy cables is very important to the stability of the mast. Very low pretensions greatly reduce the initial stiffness of the guy sets, significantly increasing deflections and reducing resistance to instability.

3.5 *MAINTENANCE AND INSPECTION*

Final tensioning of the mast should result in a vertically aligned mast under calm conditions. Mast guys and radials should be tensioned to the pretensions specified on the manufacturer's drawings, adjusted for grade variation and temperature effects. If the existing anchors are to be used for the new mast, grades should be verified and the proper tension adjustments calculated. Tensioning shall be applied incrementally at each guy level, in order to keep the mast nearly vertical during (construction) tensioning.

It should also be noted that the existing foundations may well be inadequate to sustain the TACO mast, bearing in mind that the overall weight on the base foundation will be much higher than for the Stainless mast and that the guy tensions are also much higher than those for the Stainless mast.

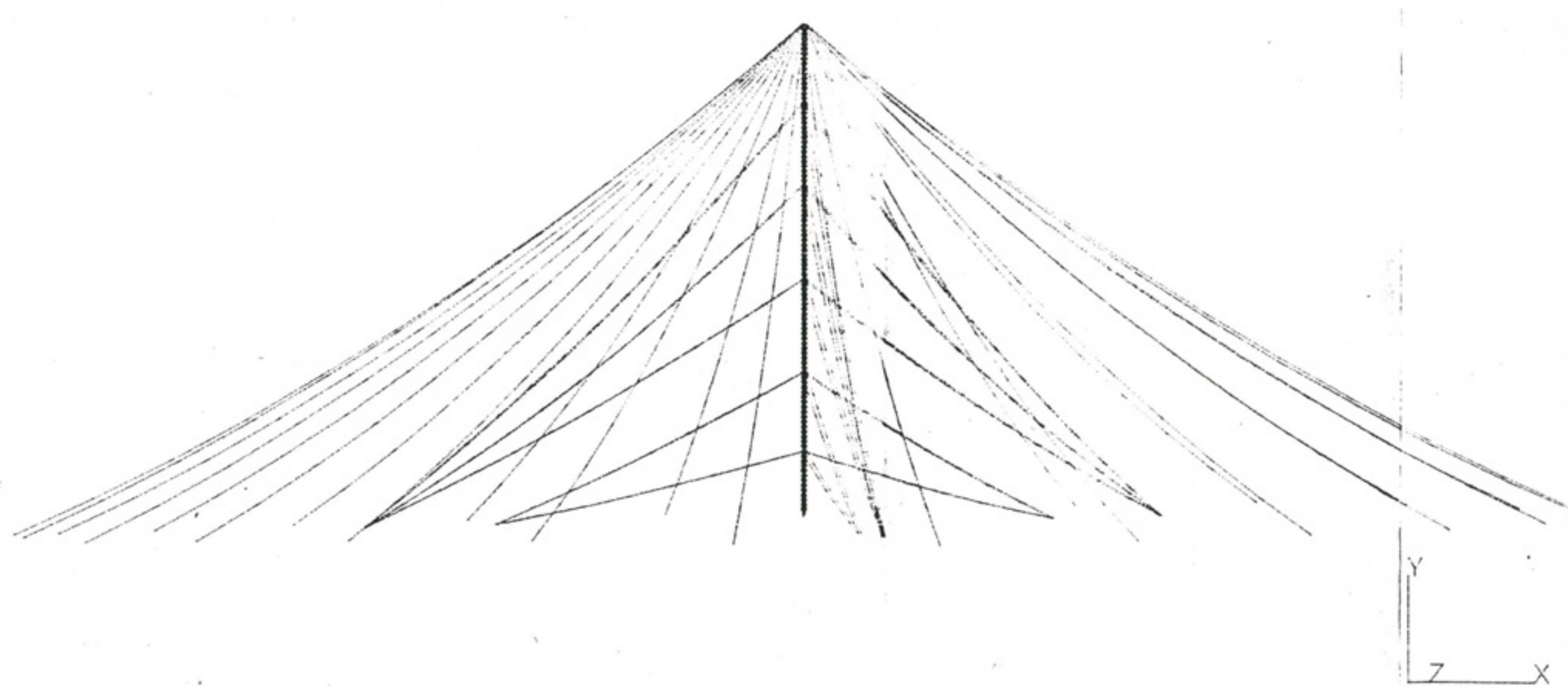

FIG. 1 - 625 FT LORAN "C" MAST   KARGABURAN, TURKEY

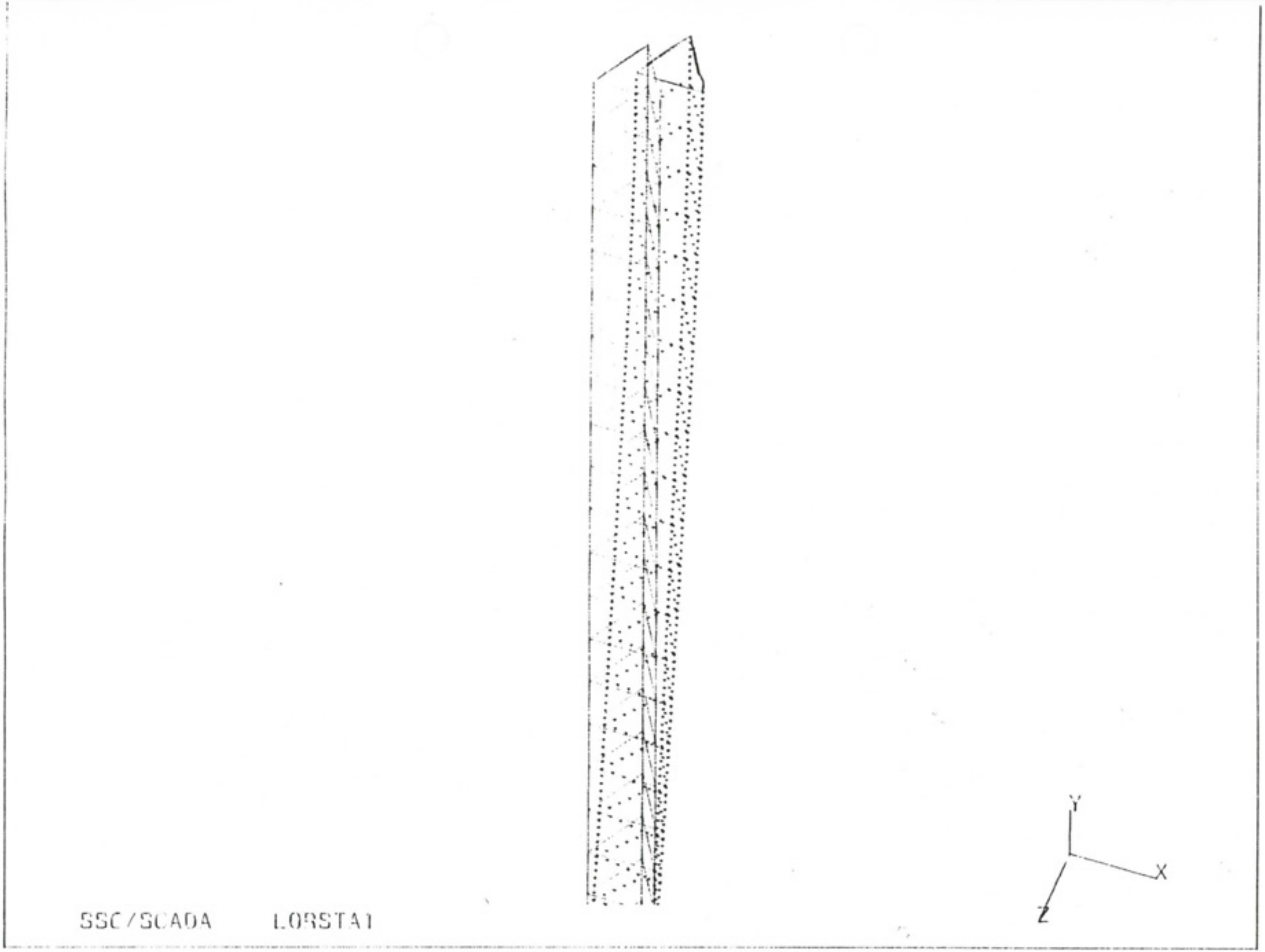

FIG. 2 - TOP DISPLACEMENTS @ INITIAL CONDITIONS

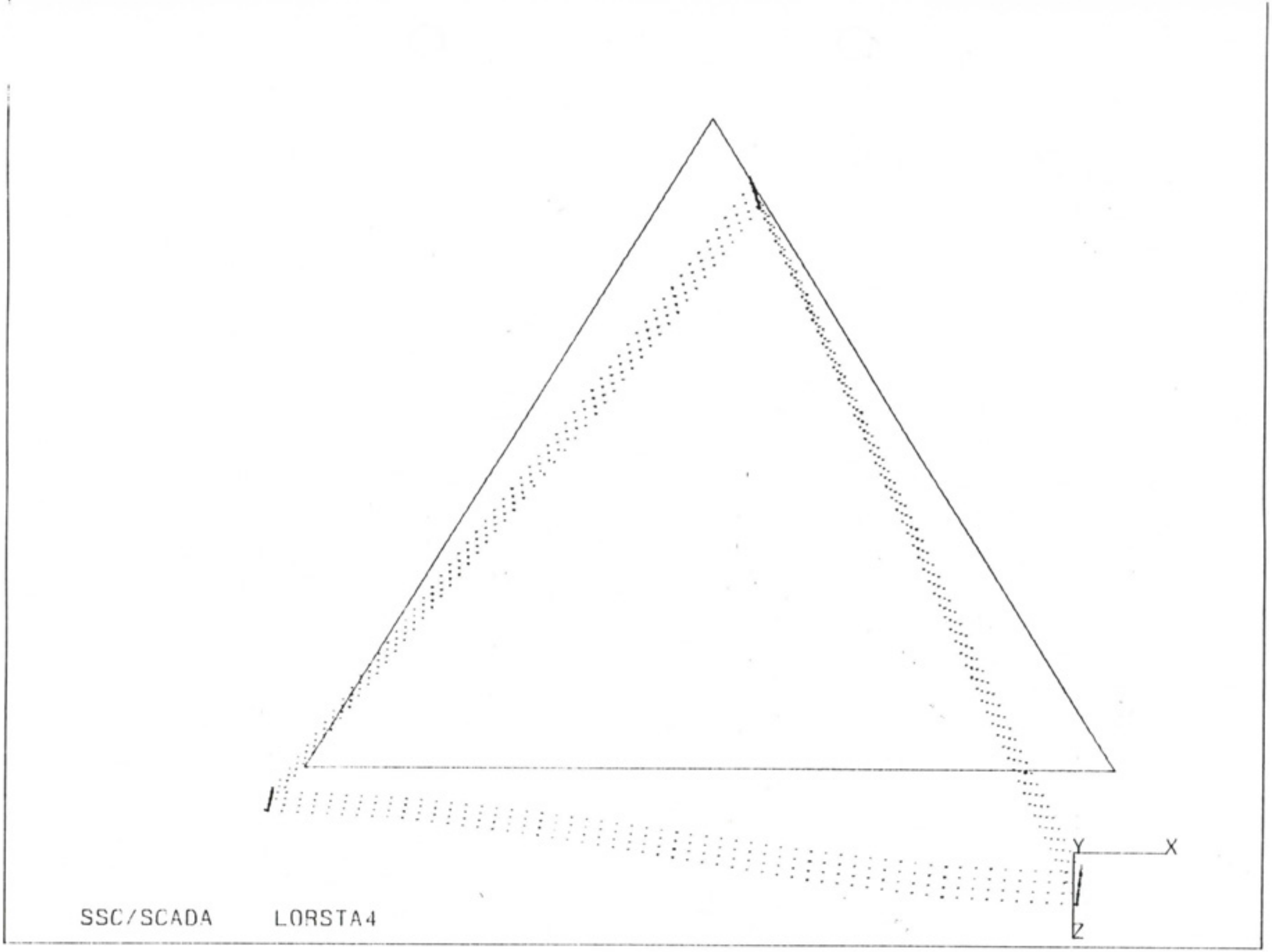

FIG. 3 - FAILURE MECHANISM @ SPAN 4

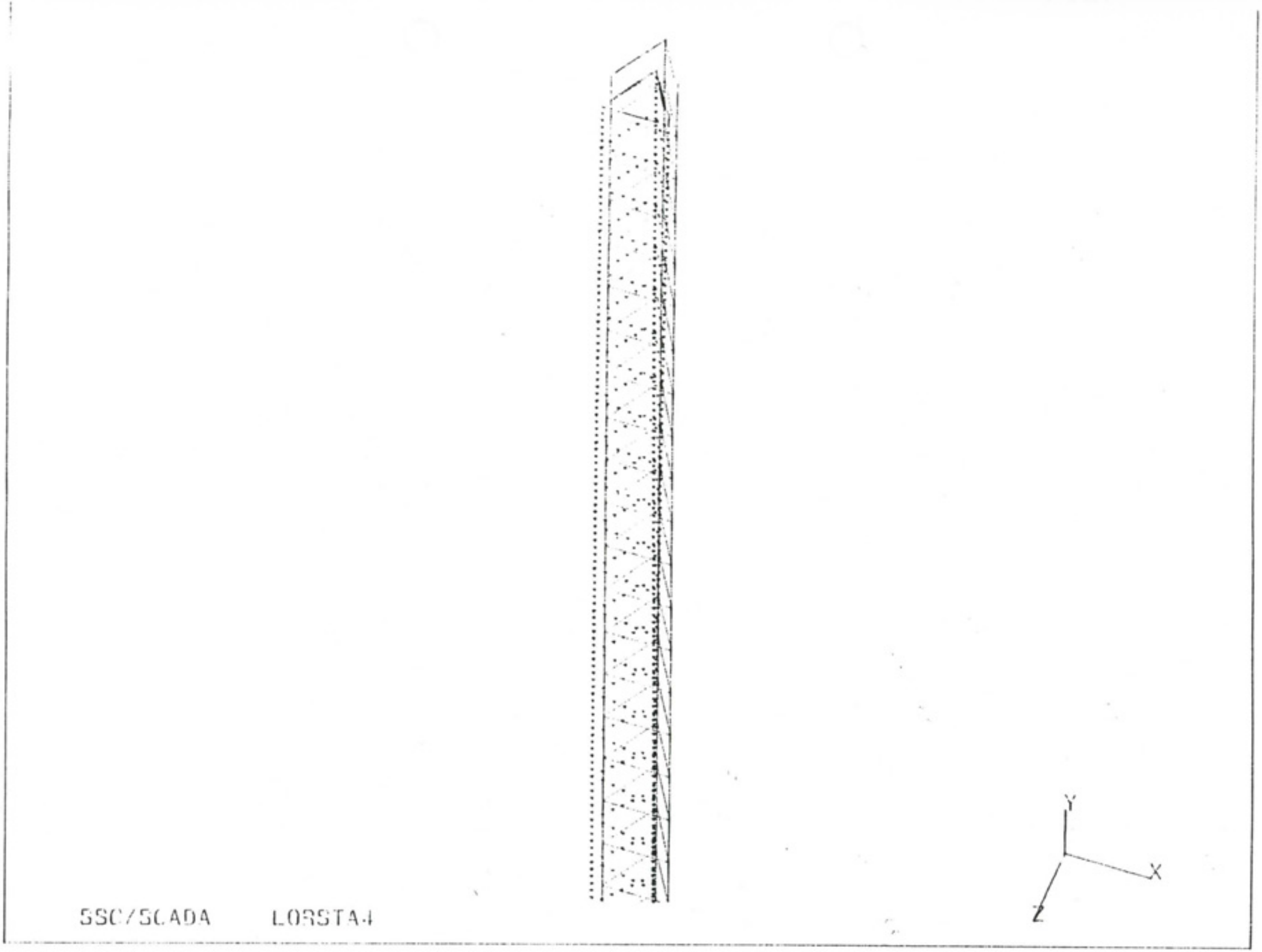

FIG. 4 - DISPLACEMENTS @ 1-1/2 IN. SNOW LOAD

5

93/9

7.4.1993

ENDEM İnsaat San. ve Tic.A.Ş.
Saatcı Bayırı, Yol Sok.,
Çağlayan Apt. Kat 3
Gayrettepe 80280
İSTANBUL

FAILURE ANALYSIS REPORT

Your Ref: Petition dated on 29.3.1993,

Our Ref: Assignment date 6.4.1993 and No: 93/929,

Fracture surfaces of the enclosed specimens which are named as "BROKEN WELD, BEND SECTION and BROKEN BOLT" were inspected according to your request, and the following conclusions were obtained :

1. BROKEN WELD (Sample # : S5)

Broken weld was inspected with stereo microscope and no evidence of fatigue were found on the fracture surface. Fracture probably occured due to excessive bending of the weld seam.

2. BEND SECTION (Sample # : S6)

Fracture surface of bend pipe was inspected with stereo microscope and no evidence of fatigue were observed. Fracture probably occured in ductile manner with bending type overload.

3. BROKEN BOLT

The fractographic examination of the fracture surface of the bolt showed that the type of fracture was pure shear, and the fracture was a result of excessive shear loads created during the failure of the tower.

Mehmet DEMİRKOL	Barlas ERYÜREK	Selahaddin ANIK
Doc.Dr.	Prof.Dr.	Prof.

Yukarıdaki imzaların adı geçenlere ait olduğu tastik olunur.

İSTANBUL TEKNİK ÜNİVERSİTESİ . İNŞAAT FAKÜLTESİ
YAPI LABORATUARI MALZEME GRUBU . AYAZAĞA KAMPÜSÜ _ İSTANBUL

Tensile Test

Endem İnşaat ve Sanayi A.Ş.

RAPOR NO./TARİHİ : 144 / 5.4.1993
BAŞVURU NO./TARİHİ : 1054 / 5.4.1993

Your Reference : M.1 : 93/159, date March 15.1993.

The required tensile testswere carried out on a number of pieces which were taken by your firm from the fallen mast in the American Base near Marmara Ereğlisi and the test results are given below :

1.) Tensile test of bolts in gusset (the sample S8-S9)

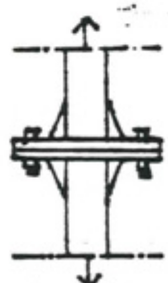

Figure 1.

The test apparatus was prepared by the Endem firm. The sample failed at 15 tons by the slipping of one of the bolts through the nut in figure 1.

2) Test for shear of bolts in gusset (the sample S10-S11)

a)

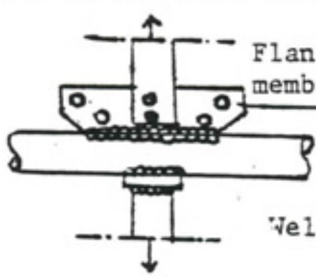

Figure 2.

The sample failed at 10.8 tons by cutting the bolts in Figure 2.

b)

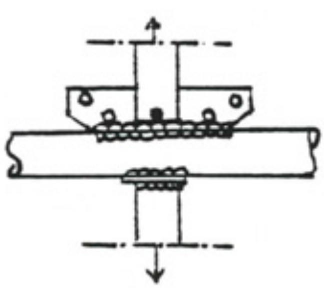

Figure 3.

Tested two samples, failed at the same load at 5.8 tons by cutting the bolts in figure 3.

3) Tensile test of welded gusset plate (the sample S12-S13)

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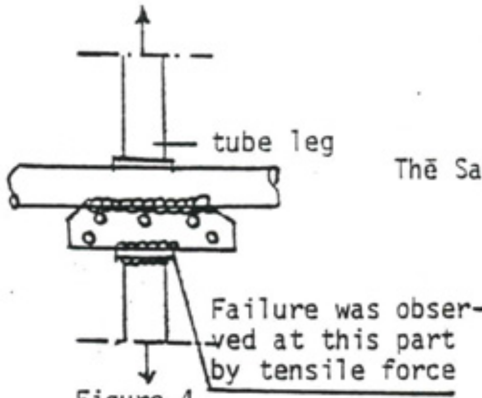


Figure 4.

The Sample failed at 30.4 tons load at the intersection of the holding part and flange in figure 4.

Failure wasn't observed at the weld of the gusset.

4) Tensile test at flange joint (the sample S14-S15)

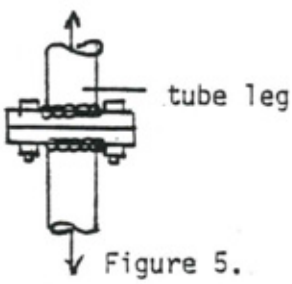


Figure 5.

No failure was observed until 50 tons in figure 5.

5) Tensile test for stays. The test were carried out according to Turkish Standard 3721. Ever each stays have seven wires and the diameter of a wire is 3.6 mm.
The results are given below :

Sample Number	Ultimate Tensile Force (KN)	Yield tensite force (KN)
AA3	96.1	79.8
	94.2	78.4
BA3	101.0	89.8
	99.0	90.3
	98.1	94.0
	99.1	88.4
BB3	101.0	91.2
	95.2	91.2
AA2	98.1	91.2
	98.1	93.3
AB3	98.1	88.4
	101.0	90.5
BA2	101.0	96.2
	100.0	96.2

Research Ass. Kemal YÜCEL

Prof. Dr. Mehmet UYAN

6

ECTION REPORT ANALYSIS TENS300.WK3

USCG
KARGABURUN - TURKEY

LANE	MINIMUM TENSION LBS	MAXIMUM TENSION LBS	DATE NOV 1982 TENSION LBS	VARIANCE FROM MIN LBS	DATE DEC 1983 TENSION LBS	VARIANCE FROM MIN LBS	DATE JULY 1984 TENSION LBS	VARIANCE FROM MIN LBS	DATE 11 OCT 1984 TENSION LBS	VARIANCE FROM MIN LBS	DATE 15 OCT 1984 TENSION LBS	VARIANCE FROM MIN LBS	DATE 17 OCT 1984 TENSION LBS	VARIANCE FROM MIN LBS	DATE SEPT 1985 TENSION LBS	VARIANCE FROM MIN LBS	DATE DEC 1986 TENSION LBS	VARIANCE FROM MIN LBS	DATE APR 1988 TENSION LBS	VARIANCE FROM MIN LBS	DATE APR 1990 TENSION LBS	VARIANCE FROM MIN LBS	DATE MAY 1992 TENSION LBS	VARIANCE FROM MIN LBS
A1	1260	1450	1275	15	1300	40	1200	-60	1250	-10	1325	65	1410	150	1250	-10	1310	50	1290	30	1330	70	1220	-40
B1	1260	1450	1330	70	1475	215	1225	-35	1250	-10	1360	100	1440	180	1300	40	1310	50	1310	50	1360	100	1220	-40
A2	1260	1450	1250	-10	1400	140	1200	-60	1225	-35	1275	15	1300	40	1250	-10	1275	15	1260	0	1270	10	1250	-10
B2	1260	1450	1300	40	1420	160	1200	-60	1275	15	1325	65	1350	90	1275	15	1275	15	1340	80	1300	40	1280	20
A3	1260	1450	1350	90	1500	240	1200	-60	1250	-10	1400	140	1450	190	1300	40	1425	165	1360	100	1350	90	1320	60
B3	1260	1450	1325	65	1420	160	1180	-80	1215	-45	1345	85	1400	140	1250	-10	1325	65	1300	40	1310	50	1250	-10
TWIST (DEG			0.3 CCW		0.3 CCW		0.5 CCW										0.33 CCW		0.50 CCW		0.33 CCW		0.33 CCW	
A1	1120	1290	1160	40	1300	180	1140	20	1175	55	1215	95	1325	205	1250	130	1260	140	1160	40	1260	140	1200	80
B1	1120	1290	1150	30	1300	180	1080	-40	1125	5	1210	90	1290	170	1200	80	1200	80	1110	-10	1190	70	1190	70
A2	1120	1290	1200	80	1320	200	1125	5	1225	105	1175	55	1150	30	1230	110	1200	80	1160	40	1170	50	1240	120
B2	1120	1290	1240	120	1310	190	1140	20	1250	130	1200	80	1200	80	1200	80	1225	105	1150	30	1160	40	1220	100
A3	1120	1290	1175	55	1350	230	1100	-20	1150	30	1275	155	1350	230	1180	60	1210	90	1140	20	1130	10	1200	80
B3	1120	1290	1040	-80	1350	230	1200	80	1250	130	1350	230	1225	105	1150	30	1250	130	1125	5	1140	20	1250	130
TWIST (DEG			0.6 CCW		0.6 CCW		1.33 CCW										1.33 CCW		1.00 CCW		1.25 CCW		1.33 CCW	
A1	1240	1430	1330	90	1420	180	1300	60	1340	100	1350	110	1450	210	1450	210	1390	150	1300	60	1350	110	1270	30
B1	1240	1430	1400	160	1550	310	1360	120	1380	140	1450	210	1500	260	1450	210	1475	235	1375	135	1440	200	1250	10
A2	1240	1430	1275	35	1310	70	1250	10	1300	60	1250	10	1175	-65	1300	60	1325	85	1275	35	1260	20	1260	20
B2	1240	1430	1275	35	1250	10	1250	10	1300	60	1270	30	1225	-15	1300	60	1300	60	1275	35	1240	0	1290	40
A3	1240	1430	1475	235	1520	280	1375	135	1425	185	1600	360	1510	270	1205	-35	1430	190	1340	100	1310	70	1330	90
B3	1240	1430	1450	210	1550	310	1350	110	1400	160	1550	310	1560	320	1325	85	1375	135	1390	150	1290	50	1330	90
TWIST (DEG			1.2 CCW		1.2 CCW		2.5 CCW										3.00 CCW		3.00 CCW		3.00 CCW		3.00 CCW	
A1	1000	1150	1060	60	1130	130	1110	110	1125	125	1100	100	1150	150	1175	175	1150	150	1075	75	1100	100	1030	30
B1	1000	1150	1060	60	1110	110	1060	60	1100	100	1090	90	1140	140	1150	150	1100	100	1050	50	1070	70	1040	40
A2	1000	1150	1175	175	1150	150	1180	180	1250	250	1090	90	1075	75	1200	200	1150	150	1120	120	1100	100	1090	90
B2	1000	1150	1250	250	1200	200	1200	200	1300	300	1110	110	1075	75	1185	185	1200	200	1140	140	1100	100	1150	150
A3	1000	1150	1175	175	1150	150	1000	0	1050	50	1125	125	1160	160	1000	0	1080	80	1050	50	980	-20	1050	50
B3	1000	1150	1040	40	1160	160	1000	0	1025	25	1100	100	1170	170	1050	50	1075	75	1050	50	970	-30	1050	50
TWIST (DEG			1.7 CCW		1.7 CCW		2 CCW										2.50 CCW		2.00 CCW		2.00 CCW		2.00 CCW	

LANE	MINIMUM TENSION LBS	MAXIMUM TENSION LBS	DATE NOV 1982 TENSION LBS	VARIANCE FROM MIN LBS	DATE DEC 1983 TENSION LBS	VARIANCE FROM MIN LBS	DATE JULY 1984 TENSION LBS	VARIANCE FROM MIN LBS	DATE OCT 1984 TENSION LBS	VARIANCE FROM MIN LBS	DATE OCT 1984 TENSION LBS	VARIANCE FROM MIN LBS	DATE OCT 1984 TENSION LBS	VARIANCE FROM MIN LBS	DATE SEPT 1985 TENSION LBS	VARIANCE FROM MIN LBS	DATE DEC 1986 TENSION LBS	VARIANCE FROM MIN LBS	DATE APR 1988 TENSION LBS	VARIANCE FROM MIN LBS	DATE APR 1990 TENSION LBS	VARIANCE FROM MIN LBS	DATE APR 1992 TENSION LBS	VARIANCE FROM MIN LBS
A1	1500	1725	1575	75	1650	150	1600	100	1600	100	1525	25	1640	140	1600	100	1575	75	1500	0	1540	40	1430	-70
B1	1500	1725	1600	100	1650	150	1600	100	1600	100	1500	0	1675	175	1600	100	1575	75	1475	-25	1560	60	1470	-30
A2	1500	1725	1625	125	1520	20	1600	100	1650	150	1500	0	1500	0	1625	125	1625	125	1575	75	1560	60	1550	50
B2	1500	1725	1675	175	1560	60	1650	150	1700	200	1525	25	1550	50	1675	175	1630	130	1575	75	1600	100	1560	60
A3	1500	1725	1620	120	1750	250	1550	50	1560	60	1775	275	1730	230	1600	100	1600	100	1540	40	1490	-10	1540	40
B3	1500	1725	1640	140	1750	250	1600	100	1600	100	1800	300	1730	230	1600	100	1575	75	1550	50	1460	-40	1600	100
TWIST (DEG			2.4 CCW		2.4 CCW		2.25 CCW										2.50 CCW		2.25 CCW		2.25 CCW		2.25 CCW	
C1	1050	1208	1200	150	1250	200	1175	125	1225	175	1225	175	1300	250	1275	225	1200	150	1210	160	1140	90	1220	170
C2	1050	1208	1250	200	1330	280	1200	150	1300	250	1300	250	1350	300	1300	250	1275	225	1260	210	1170	120	1280	230
C3	1050	1208	1200	150	1250	200	1200	150	1275	225	1200	150	1300	250	1225	175	1250	200	1150	100	1140	90	1250	200
C4	1050	1208	1180	130	1300	250	1150	100	1225	175	1225	175	1275	225	1250	200	1230	180	1125	75	1100	50	1150	100
C5	1050	1208	1200	150	1330	280	1215	165	1300	250	1250	200	1300	250	1250	200	1250	200	1150	100	1140	90	1200	150
C6	1050	1208	1210	160	1330	280	1200	150	1300	250	1200	150	1250	200	1275	225	1200	150	1150	100	1120	70	1175	125
C7	1050	1208	1260	210	1350	300	1250	200	1350	300	1275	225	1300	250	1300	250	1215	165	1220	170	1200	150	1275	225
C8	1050	1208	1175	125	1250	200	1225	175	1225	175	1175	125	1160	110	1225	175	1200	150	1110	60	1100	50	1160	110
C9	1050	1208	1225	175	1250	200	1210	160	1250	200	1200	150	1210	160	1300	250	1200	150	1200	150	1150	100	1200	150
C10	1050	1208	1170	120	1200	150	1175	125	1250	200	1175	125	1125	75	1230	180	1200	150	1140	90	1110	60	1140	90
C11	1050	1208	1210	160	1180	130	1200	150	1275	225	1200	150	1140	90	1250	200	1200	150	1140	90	1180	130	1160	110
C12	1050	1208	1180	130	1320	270	1180	130	1250	200	1175	125	1150	100	1250	200	1250	200	1140	90	1180	130	1125	75
C13	1050	1208	1250	200	1320	270	1260	210	1350	300	1275	225	1260	210	1300	250	1325	275	1260	210	1290	240	1240	190
C14	1050	1208	1280	230	1240	190	1250	200	1300	250	1300	250	1250	200	1225	175	1300	250	1250	200	1300	250	1230	180
C15	1050	1208	1225	175	1250	200	1150	100	1250	200	1100	50	1105	55	1100	50	1150	100	1100	50	1090	40	1100	50
C16	1050	1208	1280	230	1250	200	1210	160	1300	250	1225	175	1200	150	1250	200	1275	225	1200	150	1200	150	1190	140
C17	1050	1208	1200	150	1300	250	1140	90	1200	150	1140	90	1200	150	1100	50	1175	125	1140	90	1130	80	1150	100
C18	1050	1208	1225	175	1250	200	1130	80	1250	200	1180	130	1200	150	1150	100	1200	150	1175	125	1150	100	1140	90
C19	1050	1208	1200	150	1250	200	1120	70	1175	125	1150	100	1180	130	1125	75	1150	100	1100	50	1090	40	1100	50
C20	1050	1208	1190	140	1250	200	1100	50	1200	150	1215	165	1250	200	1125	75	1150	100	1140	90	1100	50	1150	100
C21	1050	1208	1200	150	1250	200	1080	30	1175	125	1225	175	1220	170	1175	125	1190	140	1075	25	1090	40	1140	90
C22	1050	1208	1210	160	1250	200	1000	-50	1200	150	1175	125	1250	200	1100	50	1200	150	1120	70	1100	50	1200	150
C23	1050	1208	1225	175	1310	260	1150	100	1225	175	1250	200	1450	400	1250	200	1300	250	1240	190	1220	170	1300	250
C24	1050	1208	1275	225	1360	310	1175	125	1275	225	1325	275	1450	400	1300	250	1325	275	1310	260	1240	190	1340	290
TWIST (DEG			3.5 CCW		3.5 CCW		2 CCW										3.00 CCW		3.00 CCW		3.25 CCW		3.00 CCW	

T ==		DATE NOV 1982	DATE DEC 1983	DATE JULY 1984	DATE OCT 1984	DATE OCT 1984	DATE OCT 1984	DATE SEPT 1985	DATE DEC 1986	DATE APR 1988	DATE APR 1990	DATE MAY 1992
RADIAL	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT	AVERAGE INCHES LEFT/ RIGHT
C11		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.093 LEFT	0.000	0.000	0.094 LEFT
C7		1.000 left	0.000	0.000	0.000	0.000	0.094 LEFT	0.375 LEFT	0.375 LEFT	0.188 LEFT	0.375 LEFT	0.000
C3		0.000	0.375 RIGHT	0.000	0.000	0.000	0.188 LEFT	0.000	0.281 LEFT	0.188 LEFT	0.000	0.000
C11		0.250 LEFT	0.000	0.094 LEFT	0.000	0.000	0.188 RIGHT	0.000	0.093 LEFT	0.000	0.188 LEFT	0.000
C7		0.250 right	0.000	0.000	0.000	0.000	0.094 LEFT	0.375 LEFT	0.563 LEFT	0.000	0.375 LEFT	0.094 RIGHT
C3		0.000	0.375 RIGHT	0.094 RIGHT	0.000	0.000	0.281 LEFT	0.375 LEFT	0.281 LEFT	0.188 LEFT	0.000	0.094 LEFT
C11		1.000 LEFT	0.000	0.468 LEFT	0.375 LEFT	0.188 LEFT	0.656 RIGHT	0.000	0.375 LEFT	0.094 LEFT	0.750 LEFT	0.094 LEFT
C7		0.000	0.000	0.094 RIGHT	0.188 LEFT	0.000	0.094 LEFT	0.563 LEFT	0.750 LEFT	0.000	0.750 LEFT	0.000
C3		0.000	0.000	0.750 RIGHT	0.188 RIGHT	0.563 RIGHT	0.281 LEFT	0.375 LEFT	0.093 LEFT	0.094 LEFT	0.000	0.094 LEFT
C11		2.000 LEFT	1.000 LEFT	1.313 LEFT	1.000 LEFT	0.188 LEFT	0.094 RIGHT	0.000	0.281 LEFT	0.094 LEFT	0.750 LEFT	0.094 LEFT
C7		0.000	0.000	0.000	0.563 LEFT	0.094 RIGHT	0.375 LEFT	0.750 LEFT	0.657 LEFT	0.094 LEFT	1.313 LEFT	0.000
C3		0.250 LEFT	0.000	1.313 RIGHT	0.875 RIGHT	1.000 RIGHT	0.234 LEFT	0.188 LEFT	0.093 LEFT	0.094 LEFT	0.375 LEFT	0.094 RIGHT
C11		4.000 LEFT	3.000 LEFT	3.188 LEFT	2.438 LEFT	0.750 LEFT	0.000	0.188 LEFT	0.375 LEFT	0.281 LEFT	0.938 LEFT	0.375 LEFT
C7		0.000	1.000 LEFT	0.750 LEFT	1.000 LEFT	0.563 LEFT	0.844 LEFT	1.000 LEFT	1.125 LEFT	1.125 LEFT	1.500 LEFT	0.563 LEFT
C3		0.250 RIGHT	1.500 RIGHT	2.250 RIGHT	1.250 RIGHT	1.000 RIGHT	0.375 LEFT	0.281 LEFT	0.657 LEFT	0.281 LEFT	0.750 LEFT	0.188 RIGHT
C11		5.000 LEFT	4.000 LEFT	5.625 LEFT	3.563 LEFT	1.500 LEFT	0.563 RIGHT	0.032 RIGHT	0.563 RIGHT	0.188 LEFT	1.313 LEFT	0.750 LEFT
C7		0.250 right	0.750 LEFT	0.750 LEFT	0.563 LEFT	1.000 LEFT	0.563 LEFT	0.750 LEFT	0.188 LEFT	1.406 LEFT	1.125 LEFT	0.563 LEFT
C3		1.250 RIGHT	3.375 RIGHT	3.375 RIGHT	2.438 RIGHT	1.000 RIGHT	0.094 LEFT	0.500 LEFT	1.500 LEFT	0.468 LEFT	1.688 RIGHT	0.281 LEFT

s where viewed via C19 which is why they are now reversed

Holbrook
LOAD TESTING ENGINEERS LTD.

HOLBROOK GREEN . HALFWAY . SHEFFIELD S19 5FE

Telephone: (0742) 483488 . Fax: (0742) 485101

RECEIVED 1993

Our Ref No: 4327
Date: 23rd March 1993

Taba Limited
51 Osborne Villas
Hove Sussex
BN3 2RA

REPORT OF TEST

EQUIPMENT:- Dillon Strain Gauge
Serial No: AN39562
Range: 0-3500lbs
Increments: 50lbs

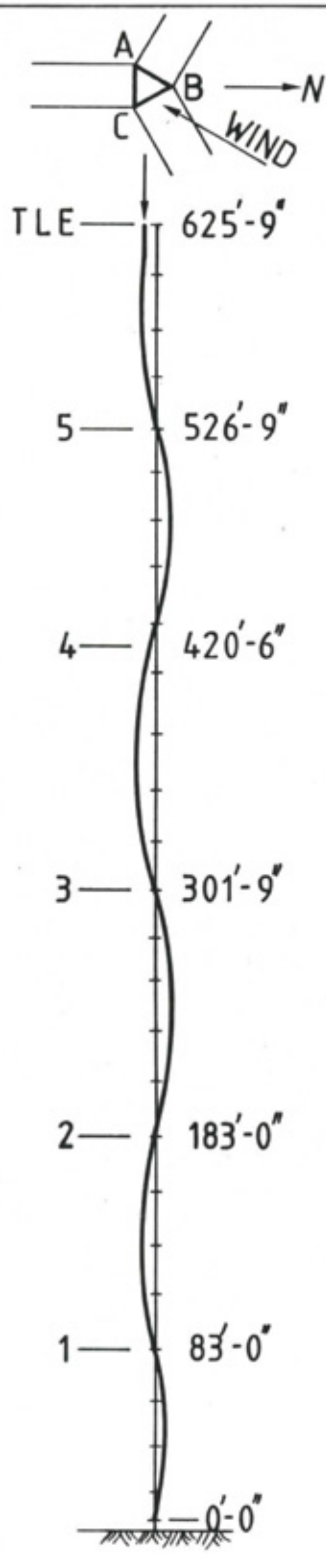
SCOPE OF TEST:- To Determine Accuracy

The Gauge was tested against Electronic Load Cell Serial No: SN1480, and reading taken at 500lbs intervals throughout the range.

READINGS:-

DILLON GAUGE	TRUE READING	DEVIATION	%
500 lbs	506 lbs	+5 lbs	+1.2%
1000 lbs	1012 lbs	+12 lbs	+1.2%
1500 lbs	1562 lbs	+62 lbs	+4.4%
2000 lbs	2068 lbs	+68 lbs	+3.3%
2500 lbs	2574 lbs	+74 lbs	+3.0%
3000 lbs	3058 lbs	+58 lbs	+2.0%
3500 lbs	3542 lbs	+42 lbs	+1.4%

D MORRIS
SENIOR ENGINEER



1st. Diagram

Normal condition, subject to "as erected" geometry, under calm conditions.

DRG. No. 900-039

THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!

1	DRAWN VJS, DATE 4-5-93, CHECKED, APPROVED	NEW ISSUE

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm, 0.0 ± 0.5m[m], 0.00 ± 0.2m[m]

SCALE:

DIMS. IN m[m]

MATERIAL:

FINISH:

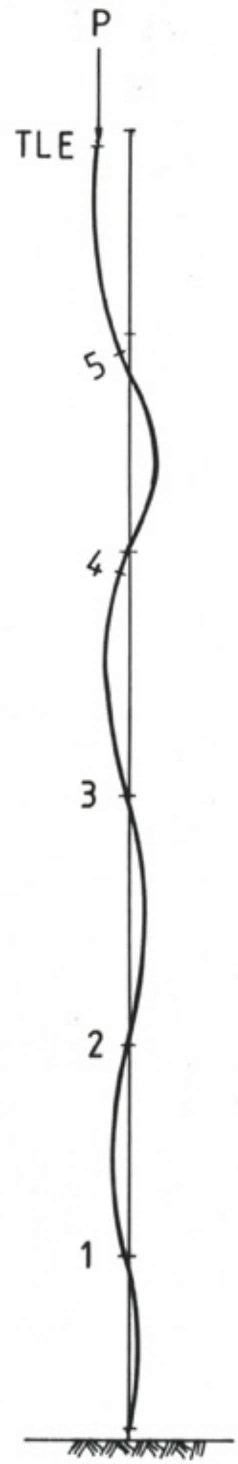
TITLE: LINE DIAGRAM OF FALL[...] MAST-STAGE 1 U.S.C.G. - TURKEY

TA300 C

TABA

TABA LIMITED, 51 OSBORNE VILL[...], HOVE, SUSSEX, BN3 2RA. TEL: 0273 726041 FAX: 0273 204293

DRG. No. 900-039 SHEET 1 OF 14



2nd. diagram

P. Critical reached (load approaching 215000 lbs.) including accretion of wet snow/ice tending to overload status, under light wind, with full stay & TLE compressions & self weight.

DRG. No. 900-039

THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!

1	DRAWN VJS	DATE 4-5-93	CHECKED	APPROVED	NEW ISSUE

TOLERANCES: UNLESS OTHERWISE STATED
0 ± 1mm
0.0 ± 0.5m
0.00 ± 0.2m

SCALE:
DIMS. IN m

MATERIAL:

FINISH:

TITLE: LINE DIAGRAM OF FALL
MAST-STAGE 2
U.S.C.G.

TA300C

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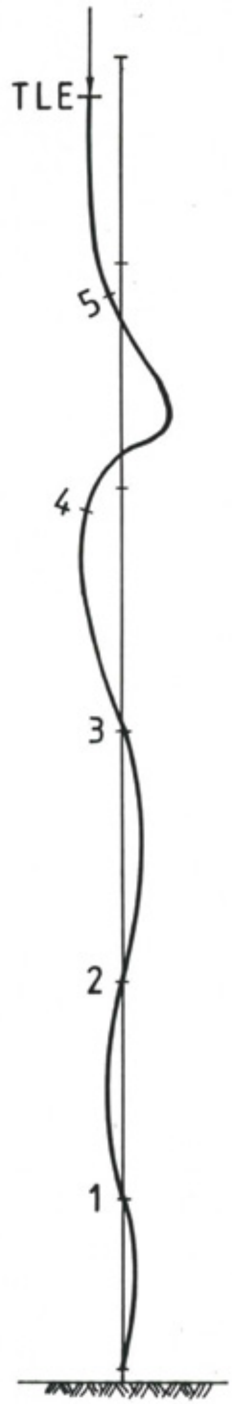
DRG. No. 900-039 SHEET 2 OF 14



3rd. Diagram

Buckling instability occurs, possibly in two zones concurrently forming mechanisms between mast sections 19&20 + between 14&15. This motion is believed to have been torsional clockwise causing tension failure in Leg B (19-20) & Leg C (14-15).

DRG. No. 900-039

THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!

1	DRAWN VJS	DATE 4-5-93	NEW ISSUE
	CHECKED	APPROVED	

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm, 0.0 ± 0.5m, 0.00 ± 0.2m

SCALE:

DIMS. IN m

MATERIAL:

FINISH:

TITLE: LINE DIAGRAM OF FALLI
MAST-STAGE 3
U.S.C.G.-TURKEY

TA300C

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BN3 2RA.
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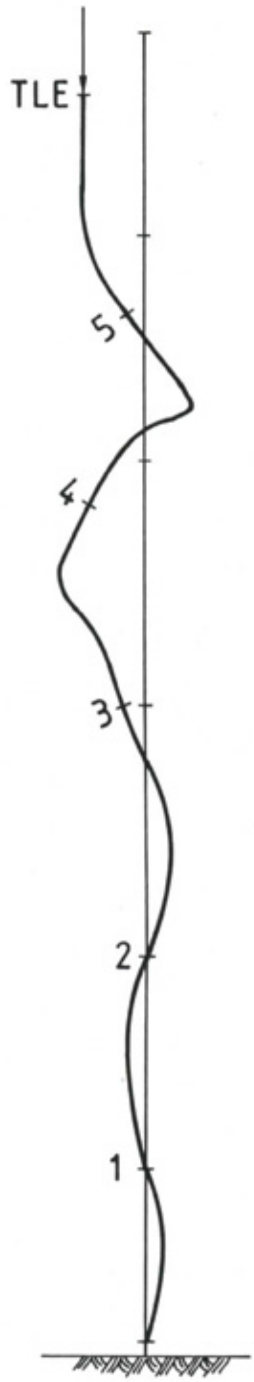
DRG. No. 900-039 SHEET 3 OF 14



4th. Diagram

The down-thrust of 24 TLE's, 12 stays & excess "super-load" further accelerate the impending destruction.

DRG. No. 900-039

THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!

1	DRAWN VJS DATE 5-5-93 CHECKED APPROVED	NEW ISSUE

TOLERANCES:
UNLESS OTHERWISE STATED
0 ± 1mr
0.0 ± 0.5r
0.00 ± 0.2r

SCALE:
DIMS. IN r

MATERIAL:

FINISH:

TITLE:
LINE DIAGRAM OF FAL
MAST-STAGE 4
U.S.C.G. - TURKEY

TA300C

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51 OSBORNE VI
HOVE, SUSSEX,
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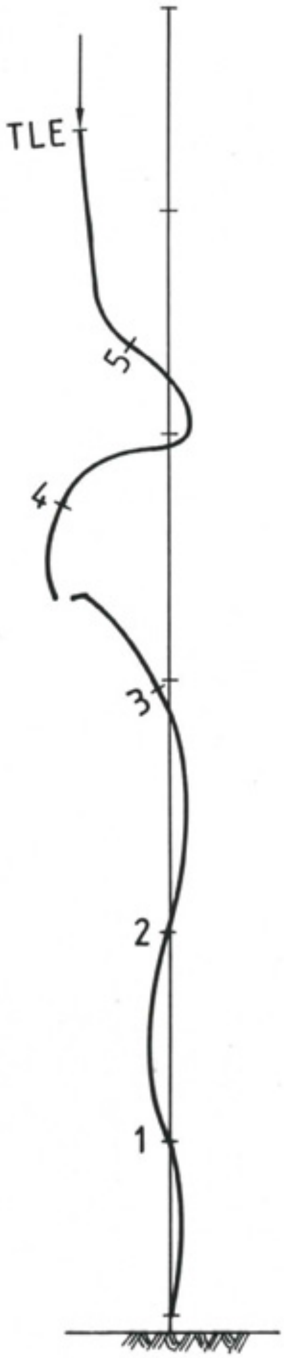
FAX: 0273 20429

DRG. No. 900-039 SHEET 4 OF 14

0 100mm

5th. Diagram

Legs A & B fail & part due to bending at the node formed between mast sections 14 & 15.



DRG. No. 900-039

THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!

DRAWN	V J S
DATE	7-5-93
CHECKED	
APPROVED	
1	NEW ISSUE

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm, 0.0 ± 0.5mm, 0.00 ± 0.2mm

SCALE:

DIMS. IN m

MATERIAL:

FINISH:

TITLE: LINE DIAGRAM OF FALLI
MAST - STAGE 5
U.S.C.G. - TURKEY

TA300C

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51 OSBORNE VILL
HOVE, SUSSEX,
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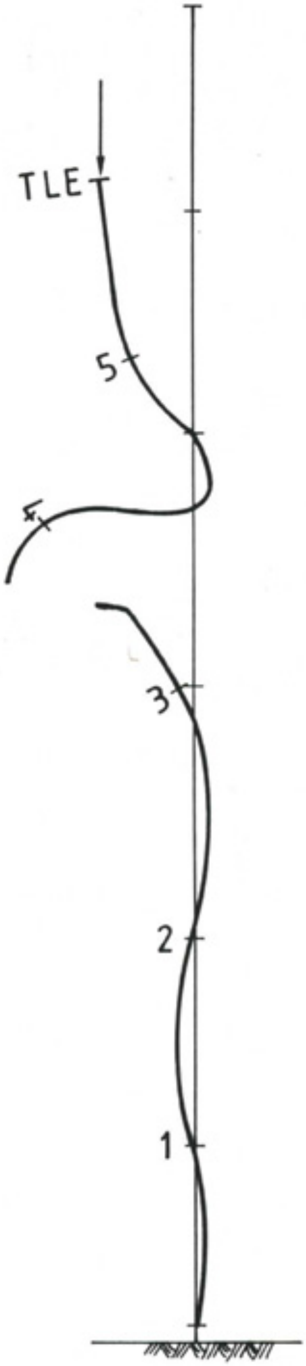
DRG. No. 900-039 SHEET 5 OF 14



6th. Diagram

The release of energy in the failure tends to close the zone between stay levels 4 & 5 whilst down-ward movement continues.

DRG. No. 900-039 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT-ASK!

1	NEW ISSUE
DRAWN	VJS
DATE	7-5-93
CHECKED	
APPROVED	

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm, 0.0 ± 0.5mm, 0.00 ± 0.2mm

SCALE: DIMS. IN mm

MATERIAL:

FINISH:

TITLE: LINE DIAGRAM OF FALLI... MAST-STAGE 6 U.S.C.G.-TURKEY

TA300C

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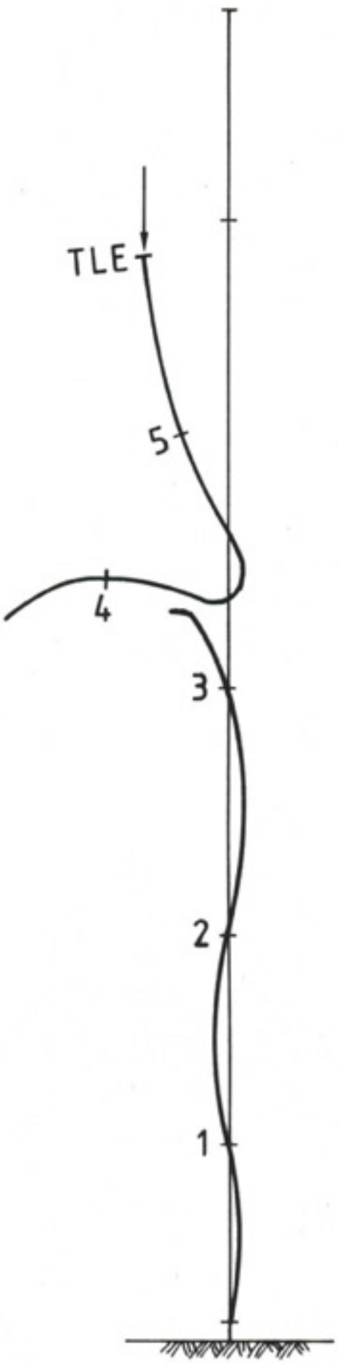
DRG. No. 900-039 SHEET 6 OF 14

DRG. No. 900-039

THIRD ANGLE PROJECTION

DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!



7th. Diagram

Possible collision between lower section & falling unit further closes the zone & imparts a rotation to the descending upper section.

TITLE: LINE DIAGRAM OF FALL MAST-STAGE 7 U.S.C.G.-TURKEY

TA300C

FINISH:

MATERIAL:

SCALE: DIMS. IN m

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm; 0.0 ± 0.5mm; 0.00 ± 0.2mm

1 | DRAWN VJS | DATE 10-5-93 | CHECKED | APPROVED | NEW ISSUE

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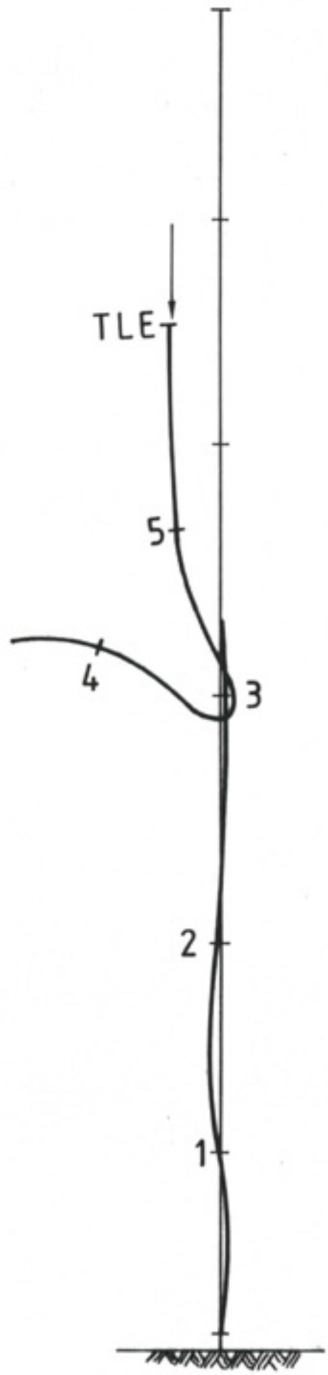
DRG. No. 900-039 SHEET 7 OF 14

DRG. No. 900-039

THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!

8th. Diagram

Broken section collides with & severs Level 3 Lane 3 stays.



1	DRAWN VJS	DATE 10-5-93	NEW ISSUE
	CHECKED		
	APPROVED		

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm, 0.0 ± 0.5mm, 0.00 ± 0.2mm

SCALE: DIMS. IN mm

MATERIAL:

FINISH:

TITLE: LINE DIAGRAM OF FALL
MAST - STAGE 8
U.S.C.G. - TURKEY

TA300C

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HOVE, SUSSEX,
BN3 2RA.
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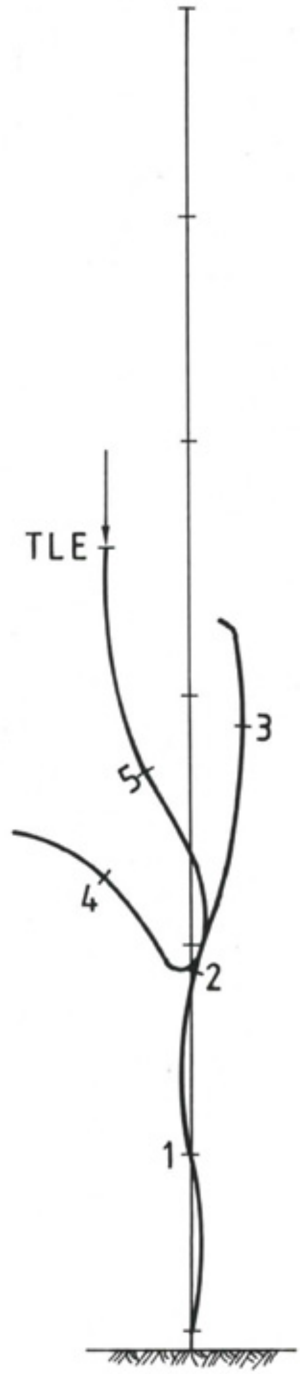
FAX: 0273 204293

DRG. No. 900-039 SHEET 8 OF 14

9th. Diagram

The lower section of the mast is now unstable at level 3 & starts to rotate & move in a S.E. direction. The shearing of Level 2 Lane 3 stays further amplifies this mode.



DRG. No. 900-039

THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!

1	DRAWN VJS	DATE 10-5-93	NEW ISSUE
	CHECKED		
	APPROVED		

TOLERANCES: UNLESS OTHERWISE STATED
0 ± 1mm
0.0 ± 0.5m
0.00 ± 0.2m

SCALE:

DIMS. IN m

MATERIAL:

FINISH:

TITLE: LINE DIAGRAM OF FALLI
MAST-STAGE 9
U.S.C.G. - TURKEY

TA 300C

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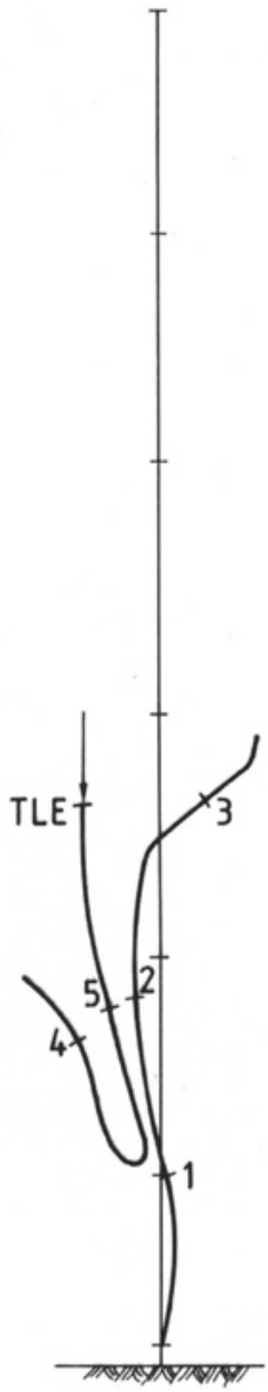
DRG. No. 900-039 SHEET 9 OF 14

DRG. No. 900-039 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT-ASK!



10th. Diagram

The falling section is severly twisted at the node zone & is further disrupted by severing the south stay at Level 1 Lane 3, causing fatal instability to the lower section of mast body.

TITLE: LINE DIAGRAM OF FALLIN
MAST-STAGE 10
U.S.C.G.-TURKEY

TA300C

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DRG. No. 900-039 SHEET 10 OF 14

FINISH:

MATERIAL:

SCALE: DIMS. IN m

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm; 0.0 ± 0.5m; 0.00 ± 0.2m

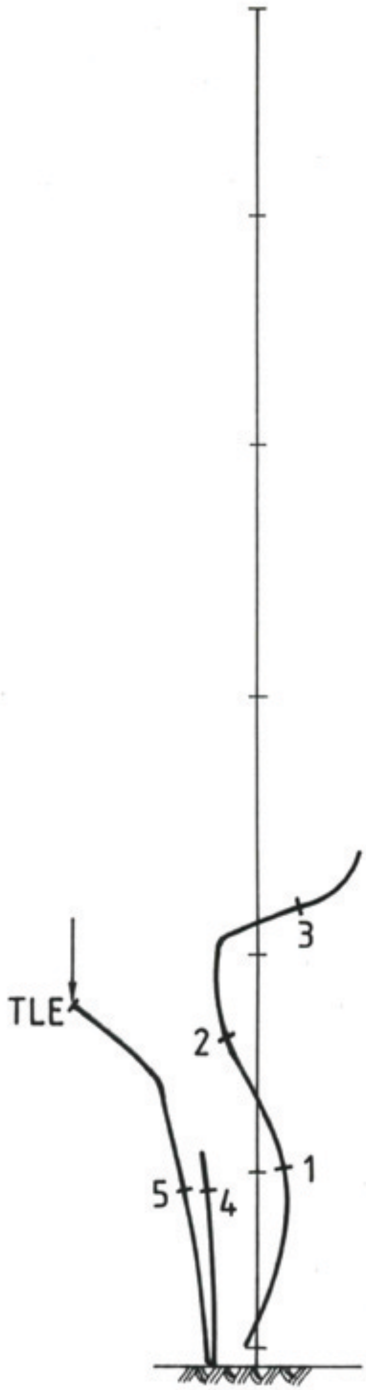
1 DRAWN VJS DATE 10-5-93 CHECKED APPROVED — NEW ISSUE

DRAWN DATE CHECKED APPROVED

DRAWN DATE CHECKED APPROVED

DRG. No. 900-039

THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!



11th. Diagram

Before entering the ground South of the mast base, the descending section breaks stays Level 1 Lane 2 causing a change in direction of movement below Level 1 such that the mechanisms formed between mast sections 3&4 & between 11&12 are accentuated.

1	DRAWN VJS	DATE 11-5-93	NEW ISSUE

TOLERANCES: UNLESS OTHERWISE STATED
0 ± 1mm
0.0 ± 0.5mr
0.00 ± 0.2mr

SCALE: DIMS. IN mm

MATERIAL:

FINISH:

TITLE: LINE DIAGRAM OF FALLI
MAST-STAGE 11
U.S.C.G. - TURKEY

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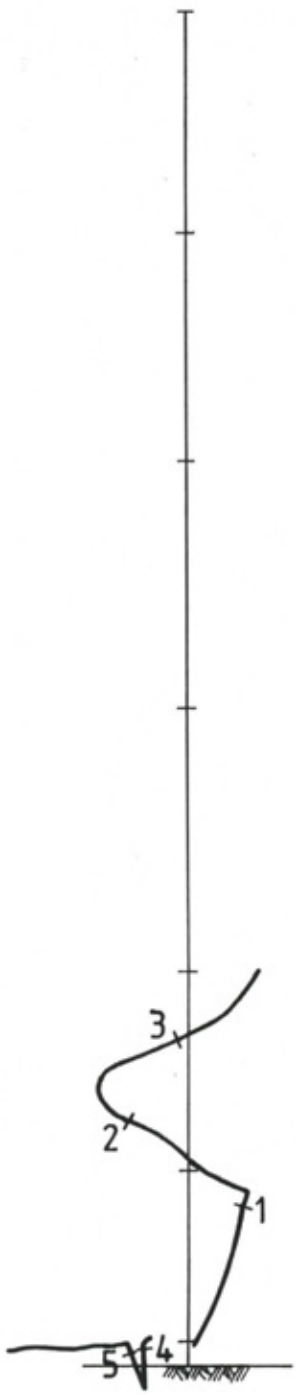
DRG. No. 900-039 SHEET 11 OF 14



12th. Diagram

The upper section has impacted with great force & now the two limbs keel over in their respective directions. The mast section 23 is folded by virtue of the fact that most of the TLE's are still unbroken. Some "LAPP" insulators fail on impact. Mechanisms are now pronounced & base insulator is near to or has failed in shear.

DRG. No. 900-039 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!

TITLE: LINE DIAGRAM OF FALLIN
MAST-STAGE 12
U.S.C.G.-TURKEY

TA300C

TOLERANCES: UNLESS OTHERWISE STATED	0 ± 1mm 0.0 ± 0.5m 0.00 ± 0.2m
SCALE:	DIMS. IN m
MATERIAL:	
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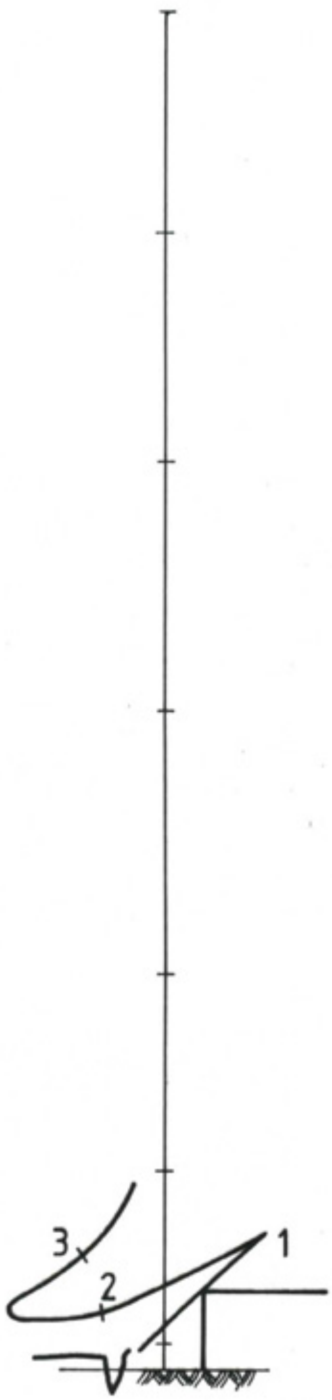
DRG. No. 900-039 SHEET 12 OF 14

13th. Diagram

All stability is removed & mast sections 1-3 contact Auxiliary building.



DRG. No. 900-039

THIRD ANGLE PROJECTION

DRAWN TO BS308

DO NOT SCALE DRG.

IF IN DOUBT - ASK!

1	DRAWN V J S	DATE 11-5-93	CHECKED	APPROVED	NEW ISSUE

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm, 0.0 ± 0.5mm, 0.00 ± 0.2mm

SCALE:

DIMS. IN mm

MATERIAL:

FINISH:

TITLE: LINE DIAGRAM OF FALLING MAST-STAGE 13 U.S.C.G.-TURKEY

TA300C

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DRG. No. 900-039

DRG. No. 900-039 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!

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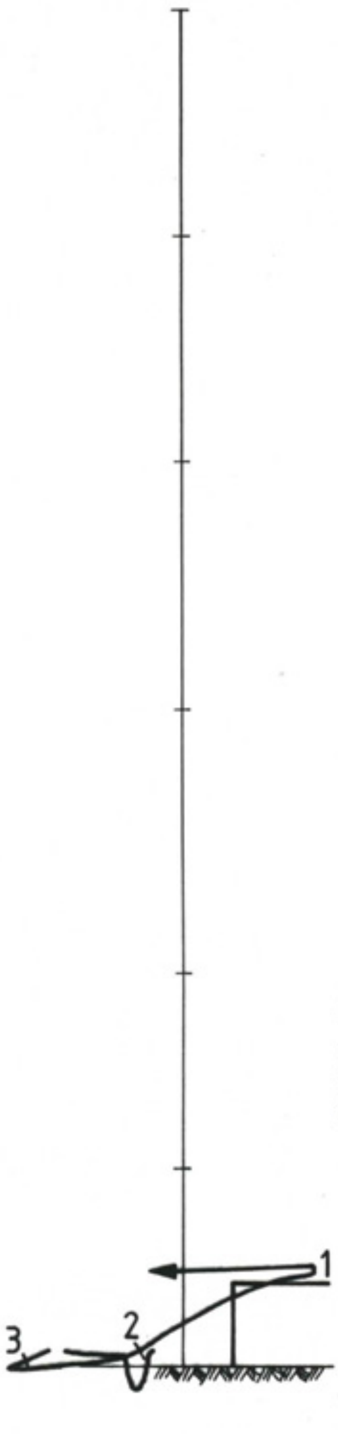
SEC714

14th. Diagram

The remains of the lower part of the mast come to rest against the Tx building. During this final phase the body again fails between mast sections 7 & 8 in a torsional mode.

NOTE -

For plan layout of fallen mast see drawing no. 900-035, sheet 6 of 7.



DRAWN	VJS
DATE	12-5-93
CHECKED	
APPROVED	
1	NEW ISSUE

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm, 0.0 ± 0.5m[m], 0.00 ± 0.2m[m]

SCALE:

DIMS. IN m[m]

MATERIAL:

FINISH:

TITLE: LINE DIAGRAM OF FALLIN[G] MAST-STAGE 14 U.S.C.G. - TURKEY

TA300C

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DRG. No. 900-039 SHEET 14 OF 14

9

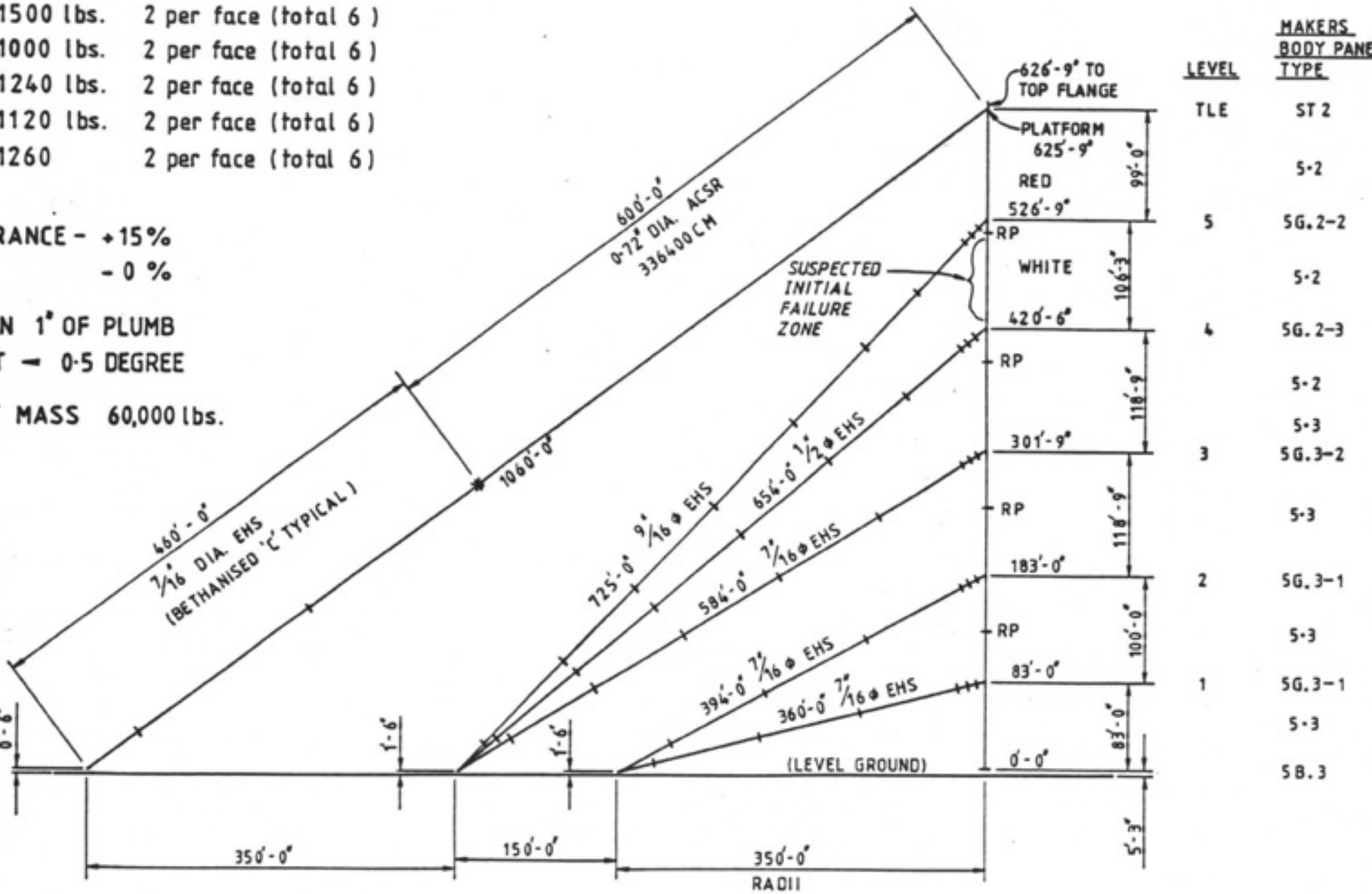
DRG. No. 900-035 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DR | IF IN DOUBT - ASK!

PART	INITIAL TENSION	No.
TLE	1050 lbs.	24@15°
5	1500 lbs.	2 per face (total 6)
4	1000 lbs.	2 per face (total 6)
3	1240 lbs.	2 per face (total 6)
2	1120 lbs.	2 per face (total 6)
1	1260	2 per face (total 6)

TOLERANCE - +15%
- 0 %

WITHIN 1° OF PLUMB
TWIST – 0·5 DEGREE

BODY MASS 60,000 lbs.



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0 — 100mm

1	DRAWN VJS / DATE 7-4-93 / CHECKED / APPROVED	NEW ISSUE

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm; 0.0 ± 0.5mm; 0.00 ± 0.2mm

SCALE: | DIMS. IN mm

MATERIAL:

FINISH:

TITLE: GENERAL ARRANGEMENT OF MAST.
UNITED STATES COAST GUARD - LORSTA KARGABURUN TA300C

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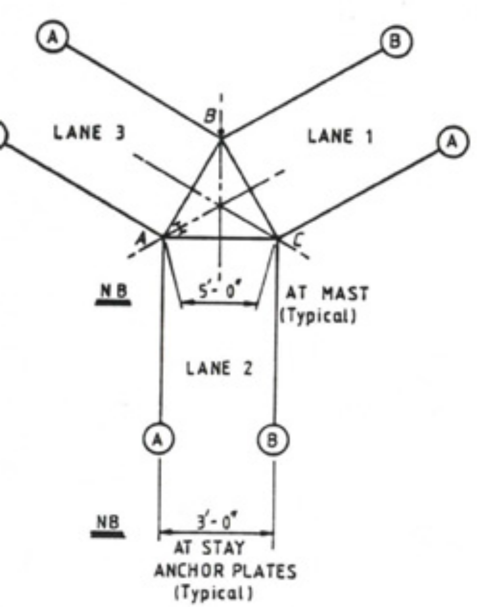
DRG. No. 900-035 | SHEET 1 OF 7 | SIZE A2

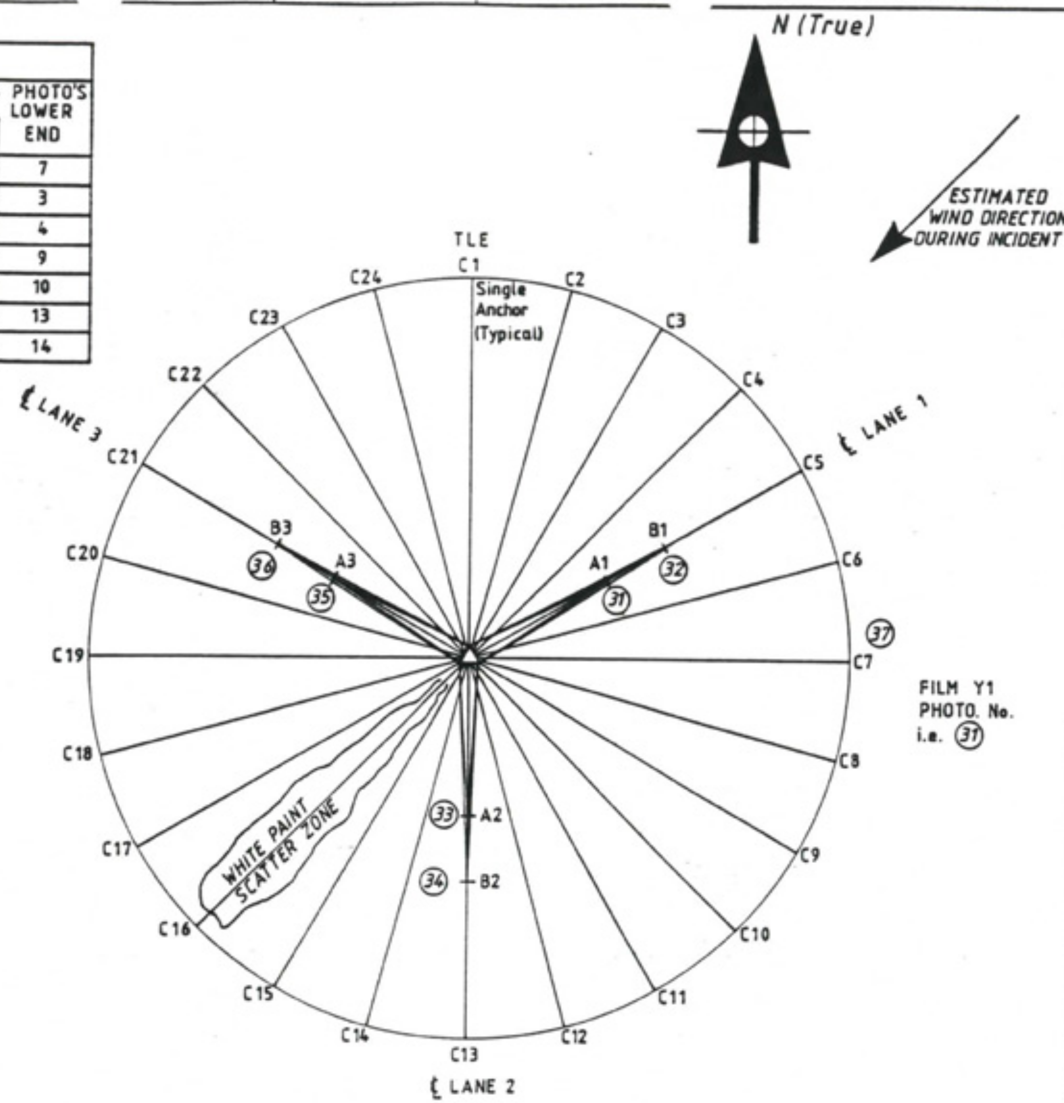
DRG. No. 900-035 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG | IF IN DOUBT - ASK!

BROKEN STAYS

LEVEL	LEG	STAY	INNER BLOCK	OUTER BLOCK	DIST. FROM MAST	FILM C4 PHOTO'S UPPER END	FILM C4 PHOTO'S LOWER END
1	A	Ⓑ	A3	—	111'-6"	8	7
1	A	Ⓐ	A2	—	11'-6"	5	3
1	C	Ⓑ	A2	—	92'-0"	6	4
2	B	Ⓐ	A3	—	45'-3"	11	9
2	A	Ⓑ	A3	—	43'-3"	12	10
3	B	Ⓐ	—	B3	55'-0"	15	13
3	A	Ⓑ	—	B3	49'-9"	16	14



LEG A = LADDER
LEG B = LIGHTING CABLE



FILM Y1
PHOTO. No.
i.e. (31)

1	DRAWN VJS, DATE 7-4-93 — NEW ISSUE
TOLERANCES: UNLESS OTHERWISE STATED	0 ± 1mm; 0.0 ± 0.5mm; 0.00 ± 0.2mm
SCALE:	DIMS. IN mm
MATERIAL:	
FINISH:	
TITLE:	STAY & TLE ORIENTATIONS. UNITED STATES COAST GUARD LORSTA KARGABURUN TA 300 C

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HOVE, SUSSEX,
BN3 2RA.
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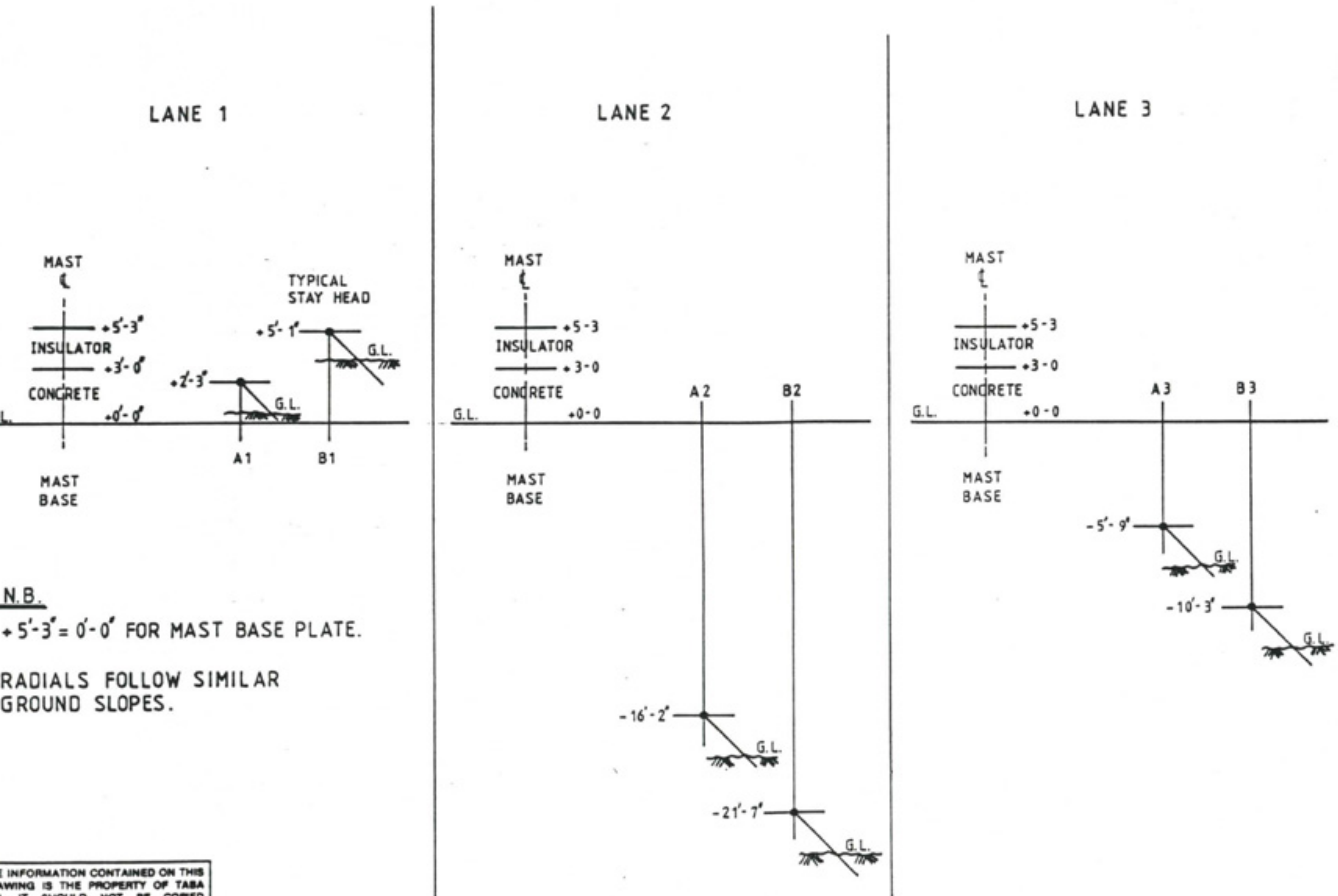
FAX: 0273 204293

DRG. No. 900-035 | SHEET 2 OF 7 | SIZE A2

0 — 100mm

DRG. No. 900 – 035	THIRD ANGLE PROJECTION		DRAWN TO BS308	DO NOT SCALE DRG	IF IN DOUBT - ASK!

LEVEL DIFFERENTIALS
(STAYS TO BASE)



N.B.

+5'-3" = 0'-0" FOR MAST BASE PLATE.

RADIALS FOLLOW SIMILAR GROUND SLOPES.

1	DRAWN VJS DATE 7-4-93 CHECKED APPROVED	NEW ISSUE	

TOLERANCES: UNLESS OTHERWISE STATED: 0 ± 1mm; 0.0 ± 0.5mm; 0.00 ± 0.2mm

SCALE: | DIMS. IN mm

MATERIAL:

FINISH:

TITLE: ANCHOR BLOCK REDUCED LEVELS. UNITED STATES COAST GUARD LORSTA KARGABURUN TA300C

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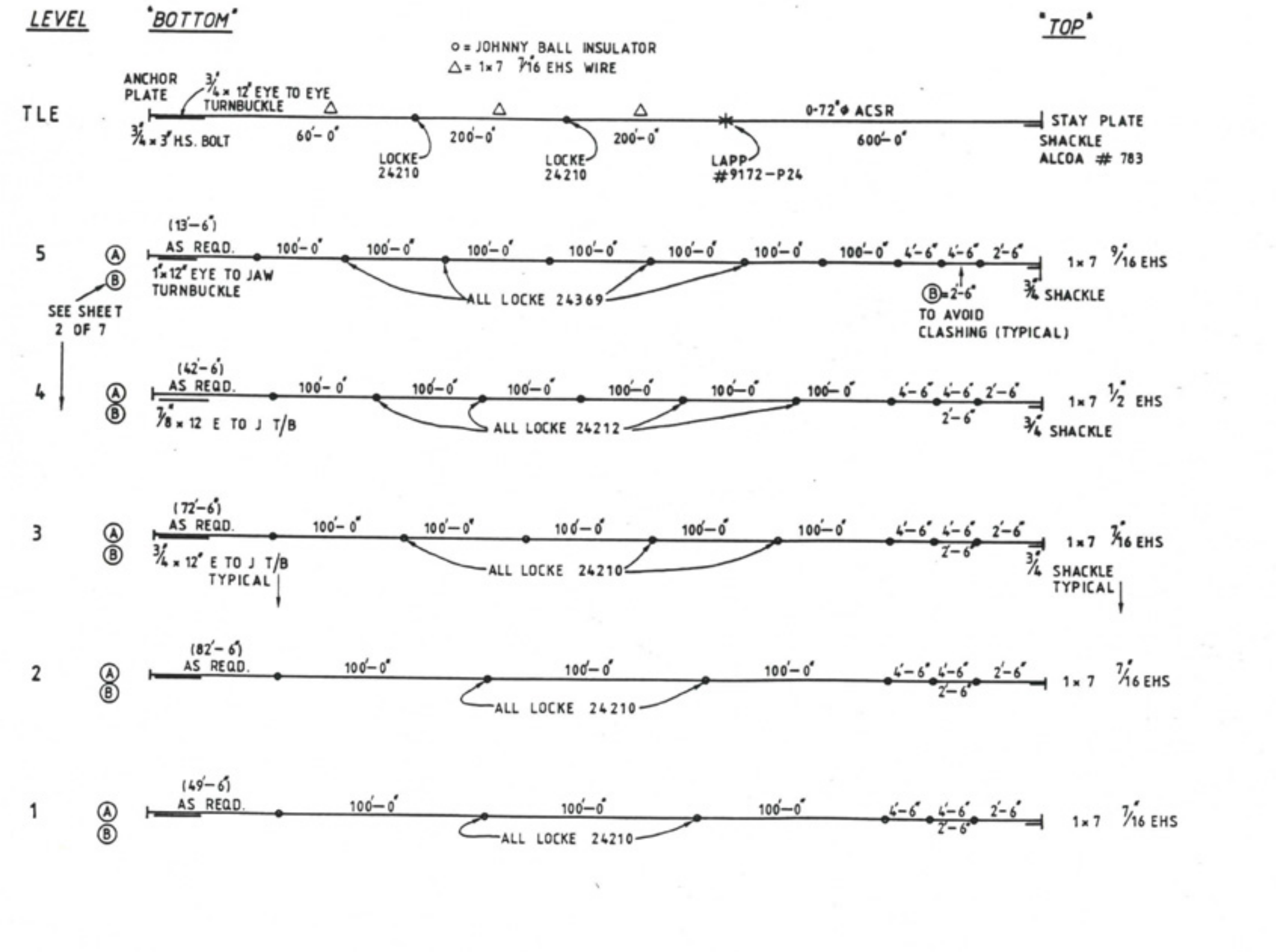
FAX: 0273 204293

DRG. No. 900 – 035 | SHEET 3 OF 7 | SIZE A2

0 — 100mm

No. 900—035 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG. | IF IN DOUBT - ASK!



Issue	Drawn	Date	Checked	Approved	Description
1	VJS	8-4-93			NEW ISSUE

TOLERANCES: UNLESS OTHERWISE STATED: 0 ± 1mm; 0.0 ± 0.5mm; 0.00 ± 0.2mm

SCALE: DIMS. IN mm

MATERIAL:

FINISH:

TITLE: STAY & TLE BREAK UP.

UNITED STATES COAST GUARD
LARSTA KARGABURUN
TA 300C

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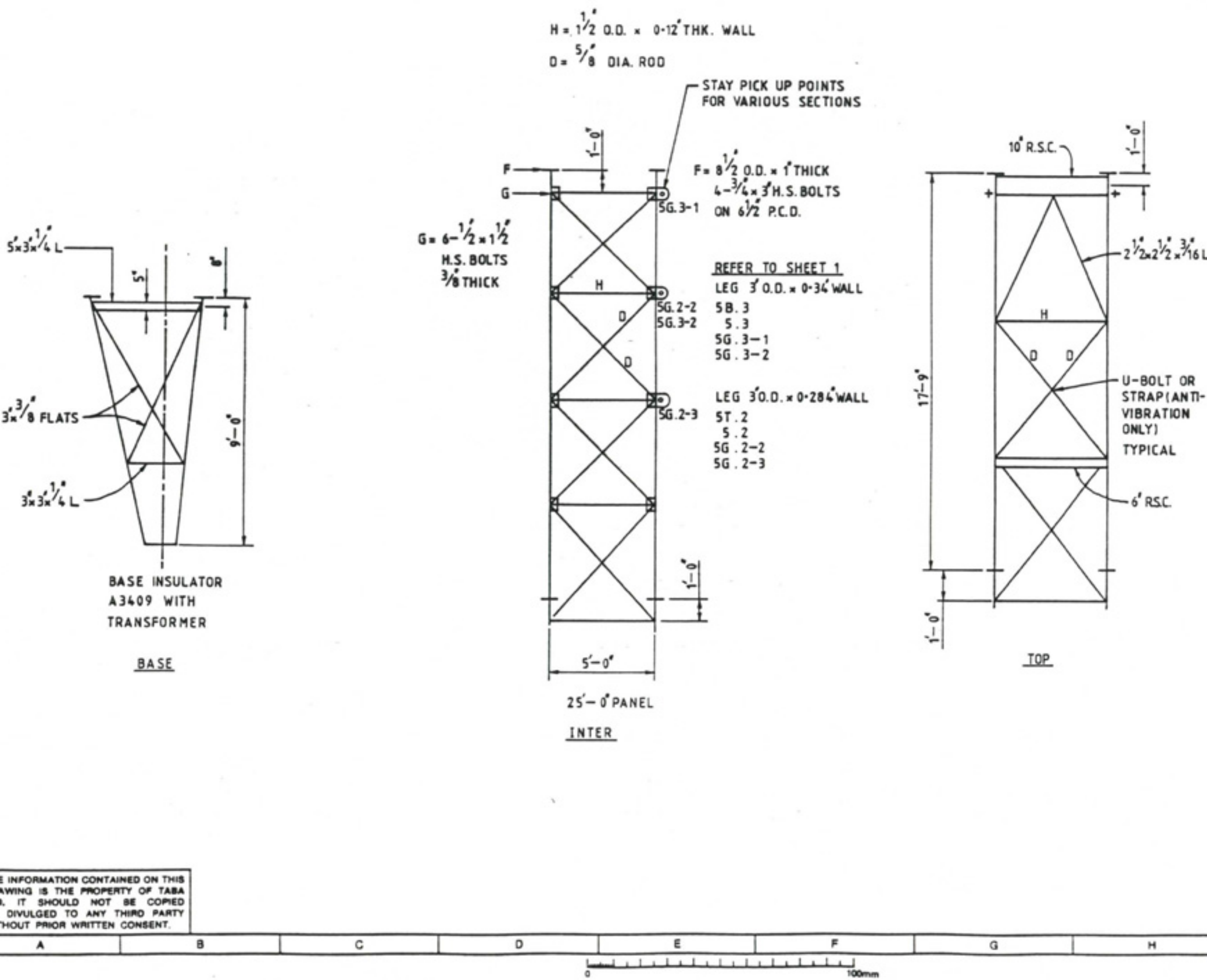
DRG. No. 900 - 035 | SHEET 4 OF 7 | SIZE A2

G. No. 900—035 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DR | IF IN DOUBT - ASK!



1	DRAWN VJS / DATE 15-4-93	NEW ISSUE	

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm; 0.0 ± 0.5mm; 0.00 ± 0.2mm

SCALE: | DIMS. IN mm

MATERIAL:

FINISH:

TITLE:
DETAILS OF MAST SECTIONS.
UNITED STATES COAST GUARD.
LORSTA KARGABURUN
TA300C

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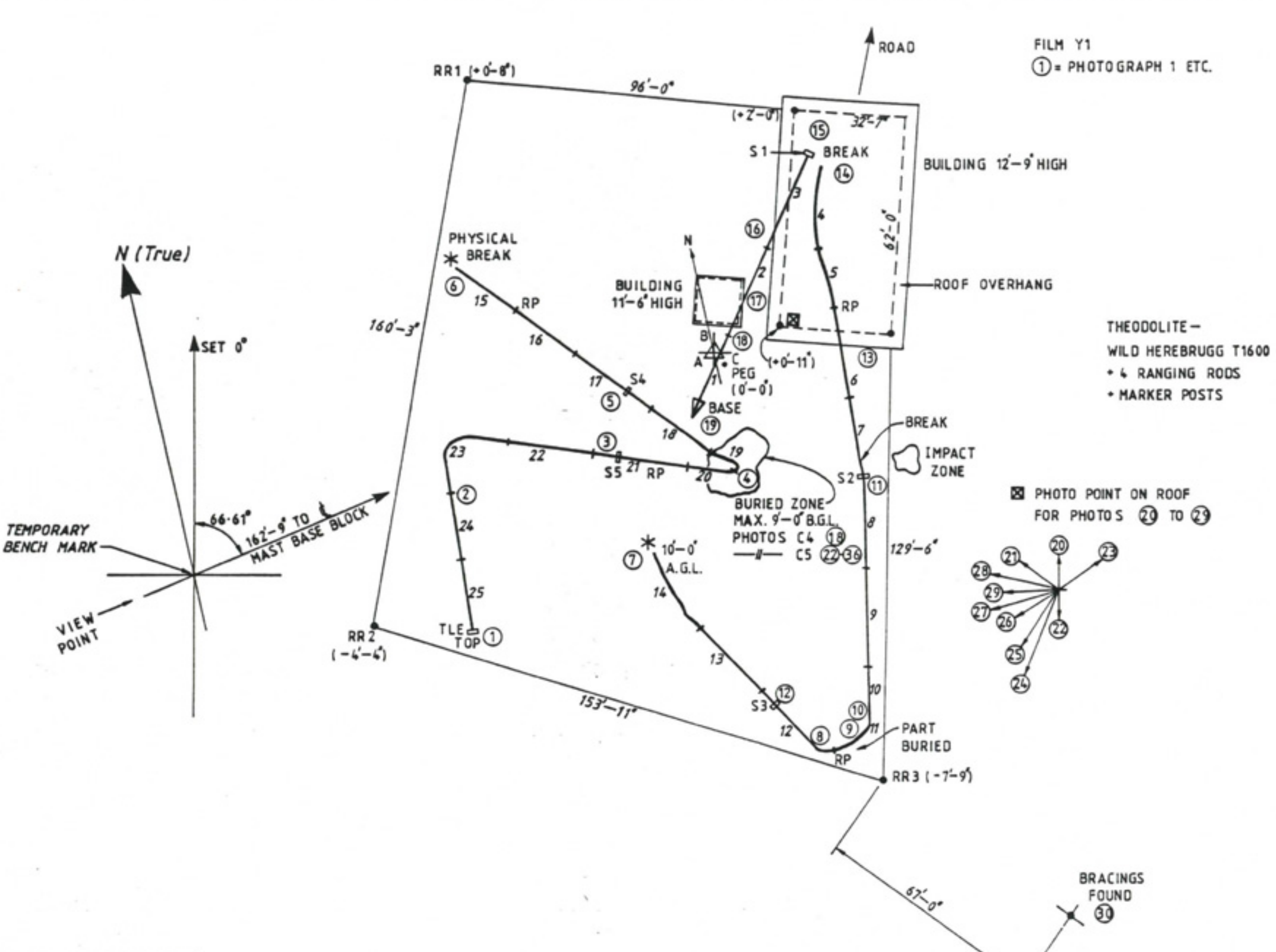
HOVE, SUSSEX,
BN3 2RA.
TEL: 0273 726041
FAX: 0273 204293

DRG. No. 900—035 SHEET 5 OF 7 SIZE A2

G. No. 900—035 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DRG | IF IN DOUBT - ASK!



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Rev.			
1	DRAWN VJS	DATE 14-4-93	NEW ISSUE

TOLERANCES: UNLESS OTHERWISE STATED 0 ± 1mm; 0.0 ± 0.5mm; 0.00 ± 0.2mm

SCALE: 1:250 | DIMS. IN mm

MATERIAL:

FINISH:

TITLE: LAYOUT OF COLLAPSED STRUCTURE. UNITED STATES COAST GUARD LORSTA KARGABURUN TA300C

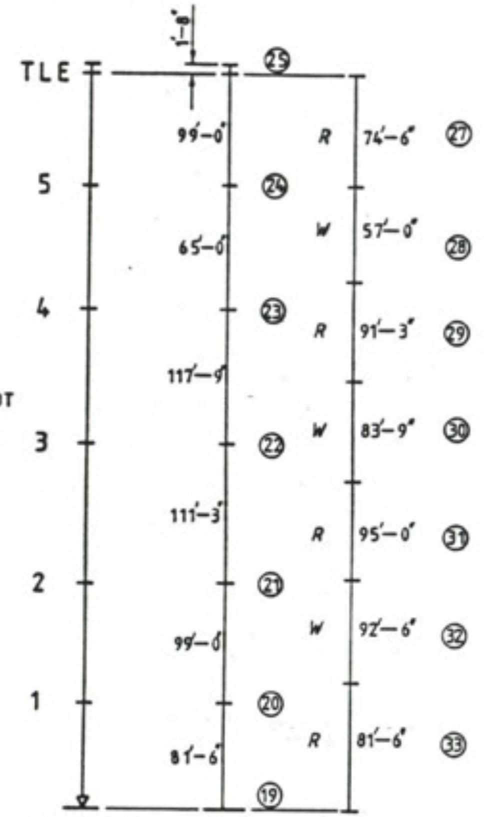
TABA LIMITED, 51 OSBORNE VILLAS, HOVE, SUSSEX, BN3 2RA. TEL: 0273 726041 FAX: 0273 204293

DRG. No. 900—035 | SHEET 6 OF 7 | SIZE A2

0 — 100mm

DRG. No. 900-035 | THIRD ANGLE PROJECTION | DRAWN TO BS308 | DO NOT SCALE DP | IF IN DOUBT - ASK!

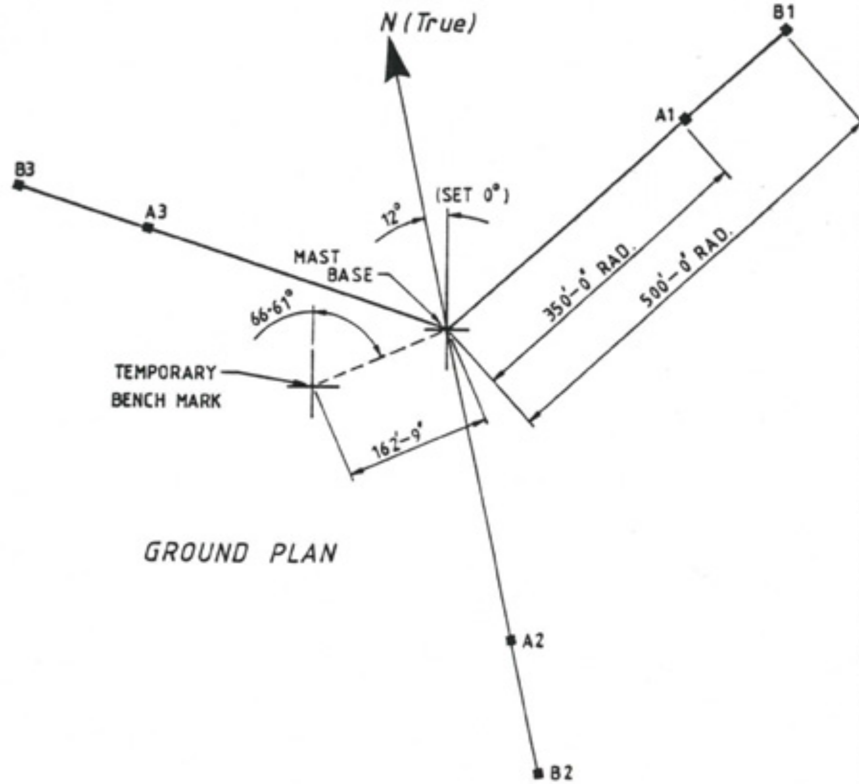
NOTE — STEELWORK NOT COMPLETE



ROADWAY LAYOUT

R = RED PAINT BAND
W = WHITE PAINT BAND

(19) = PHOTO 19 — FILM C4 ETC.



GROUND PLAN

DRAWN VJS / DATE 15-4-93 / CHECKED / APPROVED	1 NEW ISSUE
TOLERANCES: UNLESS OTHERWISE STATED	0 ± 1mm 0.0 ± 0.5mm 0.00 ± 0.2mm
SCALE:	DIMS. IN mm
MATERIAL:	
FINISH:	
TITLE:	TEMP. BENCH MARK ORIENTATION. UNITED STATES COAST GUARD. LORSTA KARGABURUN TA 300C

TABA LIMITED, 51 OSBORNE VILLAS, HOVE, SUSSEX, BN3 2RA. TEL: 0273 726041 FAX: 0273 204293

DRG. No. 900-035 | SHEET 7 OF 7 | SIZE A2

0 — 100mm

Date: May 11, 1993 10:53 AM Message ID: CeuHon019426
From: CeuHonCO/CeuHon
To: W.Thompson
Copies: R.Iwao, J.Gushiken, V.Heu, sr/MLCPs, CO/CEUOakland, ActEurE/ActEur
Attach: KARG.COMP.ANAL
Subject: KARGABURUN TOWER ANALYSIS

Wendy,

Pretty shocking stuff...assume you worked with Vern on this...I think it needs a little work in presentation before we can fully understand what is being said here (I need visuals!)...I can understand a third leg going into tensile failure in a scenario where we have bending perpendicular to the opposite face and a weakened leg section (the one under tension)...gut check tells me that we would have to have uniquely balanced ice loading and windloading to drive the tower to failure in that mode before one of the opposing face legs buckled in compression (l/r exceeded) or the whole tower section collapsed under torsional buckling (more likely)...at any rate, I don't dispute Vern's conclusion at this juncture, I merely need amplification and illustration...if you can find the time before Vern returns, pls create some 3-d images (sketches) illustrating wind, ice, stress numbers and presumed guy orientation being explained here...I read it fast, and need to study it further before we endeavor to go public with this...once you do this and we understand what Vern is talking about, I still don't want to go public until some of our tower experts in the field can take a whack at agreeing or disagreeing with this...want to go very slowly on this until we are absolutely sure what we are talking about.

Vern,

I know you are off to Cape Race and then on to London...thanks for the info thus far...I presume that this will catch up with you in London...pls discuss this with LCDR Veselka when you get together.

Walt, Pat, Bruce,

Pls give me your initial reaction to this...this caught me by surprise and we will obviously be taking this to a greater degree of development both before and after Vern returns to the office...so pls hold close as it is clearly premature for release/debate in the CE community.

Pat,

I don't have Rob Turner's e-mail address, and would appreciate it if you would float a copy of this to him too. Thanks.

v/r J.Peck

****** FORWARDED MESSAGE ******

Date: May 8, 1993 1:22 PM Message ID: CeuHon019221
From: Vernon Heu
To: J.Gushiken
Copies: CEUHonCO, W.Thompson
Blind CC:
Attach: KARG.COMP.ANAL
Subject: KARGABURUN TOWER ANALYSIS

The attachment is my report on the computer analysis of the old Kargaburun tower which I'll be giving to ACTEUR for the investigation report. I'm forwarding it here because it looks like the Kargaburun tower was seriously structurally obsolete, and there are 6 other towers with similar deficiencies. These include: Angissoq, Bo, Estartit, Lampedusa, Sellia Marina, and Wildwood. All involve Stainless tower models #1100, 1150, & 1250.

In brief, the tower legs have too thin wall thicknesses. Kargaburun would only have meet a 55 MPH design wind with no ice, and that is calculated using a parallel wind and not the more critical apex wind.

We might be looking at a very serious situation. Besides verifying my work, we will need to fully analyze the Stainless models 1100, 1150, and 1250. Warnings may need to be issued to the stations, and plans may even be necessary to replace some towers. Serious stuff involving lots of work!

KARGABURUN 625-FT TOWER COMPUTER ANALYSIS

I. INTRODUCTION

The subject of this structural analysis is a 625-foot Loran-C tower, Model 1100, manufactured by Stainless, Inc. that collapsed at Loran Station Kargaburun, Turkey on 24 February 1993.

The philosophy of this analysis is that an understanding of the circumstances of the collapse will aid in the prevention of future tower collapses. The focus of this analysis was to apply all known loading conditions to the tower, and to duplicate the suspected failure scenario.

The "TOWER12" computer program was used for this analysis. The program was developed in 1968, and includes built-in wind calculations. Although wind design standards have changed over the years, the program's wind load formula is still similar to that in the NAVFAC MIL-HDBK-1002/2, and is assumed still adequate for these purposes. This analysis has assumed only the wind direction parallel to the tower face, as this closely matches the prevailing wind direction and it also has been verified by site evidence and witness reports as the wind direction at time of the collapse. Estimated ice build-up on guys have been assumed from witness observations of other antenna guys. The original design guy tensions are also used for this analysis.

II. FINDINGS

Design specifications of the tower have not been located and are assumed to be unavailable. Due to the age of the tower and also because tower design standards have been upgraded in the past two decades, it was suspected that this tower would be underdesigned.

Findings supported this suspicion. Analysis of the tower at 80 MPH (minimum design wind velocity per NAVFAC MIL-HDBK-1002/2) and no ice indicates that compressive forces in the tower legs exceed the allowable stresses as specified by the AISC standards. With the same criteria, the tower would only meet a design wind of 55 MPH with no ice loading. All other structural members, including diagonals, horizontals, and guys, were adequate at these wind loadings.

Under uniform ice loading, the tower fared worse. Tower legs were underdesigned with only 1 inch of radial ice and no wind. With 1/2 inch of radial ice, the design wind was 30 MPH.

It is theorized that the windward guys have surfaces exposed perpendicular to the wind, hense building ice, while the leeward guys are more parallel to the wind and are less likely to contact and accululate falling snow. In a effort to model these icing conditions, an equivalent variable icing condition was manually inputted onto the windward guys - lane #1 structural guys and 7 radial guys. Under these modeled conditions, the design wind for 1/2 and 3/4 inch of radial ice was 35 and 25 MPH respectively, which was not a significant finding. What was significant was that under this type of loading, some tower legs developed tensile forces, the maximum occuring at the midpoint of leg "A" between the 4th and 5th guy levels. It is at tower section 19, located between the 4th and 5th guy levels, that a massive failure occured at the initiation of the collapse. A tension failure in leg "A" of this section is thought to have been a critical factor to this failure.

III. DISCUSSION

As in all computer programs, the output is only as good as the inputted information and more importantly only as good as the program itself. It is not within the expertise of CEU Honolulu to understand the structural assumptions or the accuracy of the program, but some drawbacks and limitations must be kept in mind to keep the program results within context. Some of the drawbacks include a limit of only three wind directions, no direct variable ice loadings, no analysis of buckling of the gross tower section, and most importantly no documentation of the internal mechanisims of the program itself.

Variable ice loading did not conclusively determine the exact circumstances of the collapse because program data input limits prevented a full trial of possible scenarios. The method of inputting a variable ice load onto the tower consisted of inputting a larger unit weight and diameter of guy cable corresponding to the assumed size of radial ice. It is not known if the program fully accepts this manner of input. It should be noted that attempts to input radial ice greater than 3/4 inch with this method resulted in error codes and a sudden change in output stresses. Results of the variable ice loadings show that significant tensile stress, concurrently with locally high compressive stresses, do develop at the midspan between the 4th and 5th guy levels.

Comparitively, direct inclusion of ice loading into the program by the normal input method resulted with maximal leg stresses at locations inconsistant with field observations of the fallen tower.

Analysis of forces in individual tower members under normal loading situations indicate that the hollow tower legs are the critical members of the tower. While leg forces exceeded the allowable compressive stresses, it must be emphasized that the loads did not reach the yield stress of the legs. It is believed that the tower was not likely to fail just because wind and ice loadings exceeded these design limits, and the 33 years of service can attest to that fact.

IV. CONCLUSION

The computer analysis was not able to find that external loading alone had stressed the tower beyond the ultimate capability. It is possible that a combination of an ununiform ice load with moderate winds had overstressed a weak or fatigued member to failure. This specific combination of a heavy icing and high winds is rare since high winds normally have a cleaning affect of ice on the tower and guys.

Overall, the tower structure is obsolete in respect to current design standards. The critical tower element are the hollow tower legs with inadequate wall thicknesses. Some later towers manufactured by Stainless, Inc. have the same 3 inch O.D. pipe legs, but with considerable thicker walls, and similar towers by other manufacturers often have solid tower legs of the same diameter.

V. RECOMMENDATIONS

Due to the nature of the inadequacy of the tower legs, other than full tower replacement, correction is impossible.

Other Stainless, Inc. towers with the same leg construction include the 625-foot Models 1150 and 1250 which were constructed during the late 1950's and early 1960's. Continued analysis of all these tower models will be necessary, and verification should be done with other tower analysis programs. Interium notice to the stations with these towers and also to all respective CEU's and support units is recommended. Precautions should be taken to avoid any work under the antenna durilng times of adverse weather.

Date: May 13, 1993 8:07 AM Message ID: MLCPs1009797
From: si/MLCPs1
To: CeuHonCO/CeuHon
Copies: sr/MLCPs, CO/CEUOakland, ActEurE/ActEur
Attach: Kargaburun
Subject: Tower Failure Analysis

Jeff:

You asked, so here's my two cents worth. At Illinios I played a lot with 625's and 700's on their tower analysis program. I also spent a year sitting on Kargaburun. Everything I learned doing those two things makes it very difficult for me to believe the weather got bad enough to exceed the design limits of the tower. The attachment gives my thoughts on where I think we should look for the answer.

Rob

Thoughts on Kargaburun Failure Analysis

1. The computer analysis probably used the 1968 AISC design equations. That's fine for recreating the tower design, but not so good in analyzing failure modes. These design equations take allowable tensile stress in the legs as 60% of the yield stress (factor of safety = 1.67).

The code takes the ultimate, or breaking stress, of steel to be equal to the yield stress. In fact the ultimate is a little higher than yield (i.e. the stress/strain curve above yield is not really a horizontal line). This gets more complicated in that statistically some samples will break under stresses higher than the published ultimate stress, some lower.

The argument presented above applies also to compression. The slenderness ratio of a compression member usually dictates its failure mode. Very short, stubby members yield before buckling. Somewhat slender members fail by inelastic buckling. Slender members buckle elastically. AISC varies the factor of safety from 1.67 to 1.92 depending on slenderness ratio; i.e. expected mode of failure. The tower legs probably are in the elastic (Euler) bucking range, so the F.S. = 1.92.

If possible I would run the program using allowable tensile stress equal to yield stress and an allowable compressive stress equal to the buckling stress. I think we'll get a better picture of how the tower should fail and at what loads.

2. What was observed in the field? I doubt you could tell if a compression failure occurred. Since the legs probably look like pretzels, who could say if one buckled before falling. You might, however, be able to find evidence of a tensile failure. Assuming the legs were still elastic up to yield stress (more on this later), they should have yielded before breaking. Yielding would increase the length of the leg and maybe cause necking of the cross section.

3. I can't think of many better places for a fatigue failure than the pipe leg of a tower. Knowing only what is in Vernon's computer analysis summary, I'll bet my money on a brittle fracture of one of the legs adjacent to a weld joint, brought on by fatigue. Let's talk about fatigue by using an example (based on 1978 AISC, the latest I had):

Assume a 3" schedule 40 pipe tower leg, braced at 7 foot intervals (sorry I can't remember exactly how 625's are built). Assume A53 pipe (Yield Stess = 35 ksi). Assume the allowable stress on the weld joint is higher than that of the pipe (it always is).

The leg alternates between compression and tension (or less compression) depending on wind direction and how much weight it supports. Max allowable tensile stress is .6(35) = 21 ksi. Max allowable compressive stress (AISC eq. # 1.5-1) for the assumed member is 14.3 ksi. In other words, when the tower was built AISC allows a range of stress of 21 - (-14.3) = 35.3 ksi.

AISC reduces the allowable range of stress as the number of loading cycles increases. For this type of welded joint member, AISC allows:

> 20,000 cycles (1.66 per day for 33 yrs) range = 21.0 ksi
> 100,000 cycles (8.30 per day for 33 yrs) range = 12.5 ksi
> 500,000 cycles (41.51 per day for 33 yrs) range = 8.0 ksi

The example shows the AISC allowable ranges of stress, not the ultimate. It does, however, give you an idea of how dramatically fatigue can reduce the capacity of such a member.

Now let's talk about brittle fracture. Increasing and decreasing strain (dynamic loading) in a member can cause a brittle fracture of the member. The chance of brittle fracture increases in cases where:

a. The temperature is low.
b. Tensile stress exists.
c. A pipe has thick walls.
d. There is 3 dimensional continuity (i.e. a pipe versus a plate) to restrain yielding.
e. Notches are present. Welds are great notches.
f. Multiaxial stresses (i.e. tension plus shear plus torsion) are present.
g. A high rate (rapidly increasing/decreasing) of loading exists.

Any of these sound possible in our case? I think so. If I'm right, look for a broken leg very near a flange in the section where the witnesses saw the failure start.

4. If I'm right, this is very bad news for the other similar towers. There is no fix against fatigue failure. When you get o a certain number of loading cycles, something breaks. That's why we replaced the two valley span antennas at OMSTA Norway. I wrote about the fatigue problem there in an Engineer's Digest article, circa 1988/9.

Date: May 14, 1993 3:25 PM Message ID: CeuHon019800
From: CeuHonCO/CeuHon
To: si/MLCPs1
Copies: sr/MLCPs, CO/CEUOakland, ActEurE/ActEur, W.Thompson, V.Heu, J.Gushiken
Attach: si-1>Kargaburun
Subject: Tower Failure Analysis

Rob,

Thanks...as they say in England, "spot on mate"...am sure that you are angling in the right direction...think we ought to supplement our "in-house" computer analysis with a series (for all of our old towers) of "external" analyses using more modern computer programs and today's design code; perhaps by contracting with one or more manufacturers like "Stainless"... would probably be good to have them also address the "fatigue" questions with some historical data; ie. what have they experienced in the way of failure rates with similar towers in similar age range...think we should do this as a subset of the Kargabarun investigation to see if there are any simulation scenarios that fit our circumstance...we will be moving in this direction shortly.

****** FORWARDED MESSAGE ******

Date: May 11, 1993 6:59 PM Message ID: MLCPs1009783
From: si/MLCPs1
To: CeuHonCO/CeuHon
Copies: sr/MLCPs, CO/CEUOakland, ActEurE/ActEur
Blind CC:
Attach: si-1>Kargaburun
Subject: Tower Failure Analysis

Jeff:

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Rob

Heath Hollingsworth - TABA

Mark Fancucci -

Leave Pat - Number of Hotel
Ask her to pass on

Computer Analysis

* EC / Turkish / US

---

* What are the deltas in load carrying ability for ?

1) Sloping anchor plane
2) Construction twist
3) Unbalanced guy loads

Total - 30/35%

Tension data and alignment are not taken at the same time.

Temperature correction is minor & relative to ?

Snow load

Initial displacement - vs - observed displacement ?

- 3rd Diagram is end of computer run

4 on is based on condition of tower on the ground.