MEASUREMENTS AND INTERPRETATION OF DYNAMIC LOADS ON BRIDGES, PHASE 3. Fatigue strength of orthotropic steel decks

ECSC NR. 7210-KD/609-F4.4/86
INTERIM REPORT: July 1 - December 31, 1988
M.H.Kolstein, J. de Back

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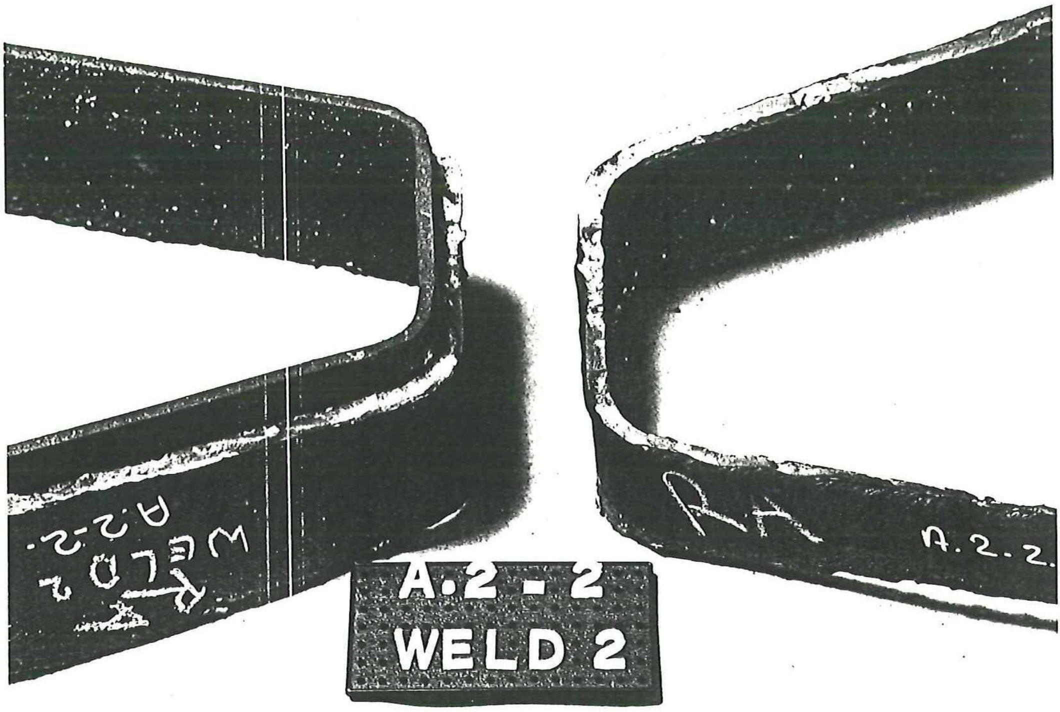

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Steelstructures

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Fatigue strength of Orthotropic Steel Decks

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Measurements and interpretation of dynamic loads on bridges.

Phase 3

Fatigue Strength of Orthotropic Steel Bridge Decks

ECSC - CONVENTION : 7210-KD/609 - F4.4/86

Contract period : 1-7-1986 until 1-7-1988; prolongation until 30-12-88

Applicant : Stichting Staalcentrum Nederland
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by

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May 1989.

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MAY 1989

CONTENTS

0. SUMMARY REPORT FOR THE PERIOD JULY 1 - DECEMBER 31, 1988

During the period under review, the experimental work carried out was concerned with:

- the execution of the last variable amplitude test on the field welded splices in trapezoidal ribs of orthotropic steel bridge decks,
- the execution of the last constant amplitude test on the connection between the cross-girder and the trapezoidal rib of a orthotropic steel bridge deck (series 1).

Further analysis of the straingauge measurements of the variable amplitude tests on the field welded splices in the trapezoidal ribs were carried out to investigate the time of starting the cracks of the testspecimens.
Besides the results of the crack growth measurements of the variable amplitude tests are given in this progress report.

A comparison of the fatigue results of the constant amplitude tests on the field welded rib splices is made with the S-N curves of the Eurocode 3.

1. RESEARCH PROGRAMME

1.1 Introduction

Last years a joint research project was defined and funded by ECCS to investigate traffic loads and the stresses they produce in steel bridges. The work was started in 1976 in a first period (phase 1) of contract [1] and was completed during a second period (phase 2) of the contract in 1983.

Six laboratories in ECCS countries participated in the research:

- Universite de Liege, Service Ponts et Charpentes, Belgium
- Universita di Pisa, Instituto di Scienza della Costruzioni, Italy
- Laboratoire Central des Ponts et Chaussees, France
- Transport and Road Research Laboratory, Bridge Design Division, England
- Fraunhofer Institut fur Betriebsfestigkeit, Fed. Rep. of Germany
- Technische Universiteit Delft, Stevinlaboratorium Staalconstructies, The Netherlands

To specify the programme and coordinate the work among the participants, the ECCS-Executive Committee F4 set up a working group. The work performed individually by the laboratories has been documented in separate reports [2, 3, 4, 5, 6 and 7] and in a common report which presents a synthesis of results and conclusions derived from phase 1 and 2 of the joint research activities [8].

The following summarizing conclusions were drawn :

- The joint research efforts reported were aimed at collecting and elaborating the required information on traffic loads of road and railway bridges and on stresses they induce in the bridge structure with particular reference to fatigue design.

- Its objectives were to present methods and data for developing and harmonising existing codes for bridge design in Europe.

- As a result of this joint research it may be concluded that quite a lot of new information is now available about axle load spectra, corresponding stress spectra and related traffic data. An example presented shows the significance of the stress spectra derived in terms of fatigue life estimation.

- It has been recognised that some components of a steel bridge structure have to sustain a large number of damaging cycles, in particular components of an orthotropic deck structure. In most cases, however, by following existing regulations in dimensioning, the stress ranges will be small enough to ensure a sufficient fatigue life.

- Nevertheless, it appears necessary to include the requirement of fatigue assessment in future bridge design codes. This would not only avoid unexpected and unwanted fatigue cracking of loadbearing components but also avoid unnecessary pessimism if anyone is aware of possible fatigue damage.

- However, fatigue design curves for welded details given by existing or draft codes show quite marked differences at high number of cycles. Therefore no definitive proposal for a fatigue life calculation can be made. Hence, as long as there is a lack of data to make appropriate decisions about the lower end of the fatigue design curves, one of the most important problems to be solved in the near future is to produce suitable test results to allow a uniform definition of those fatigue design curves for stress ranges beyond 2×10^6 cycles.

To specify the third phase program and coordinate the work, the working group meeted several times under the chairmanship of Mr. Bruls of the University of Liège.

The Dutch part of the common programme is explained in the following chapters.

1.2 Dutch part of the third phase research

At the end of the first and second phase of the joint research appears an urgent need to study the fatigue resistance of components parts of steel orthotropic decks at low stress range levels.
The orthotropic steel deck is the element most liable to have fatigue problems, for it is submitted to the direct action of the traffic wheels and thus to the highest stress variations and a great number of cycles.
The design and maintenance of the joint details represent a considerable amount of the costs of the bridge.

The third phase of the research is concentrated upon the analysis of the fatigue strength of orthotropic steel decks under the complex action of road traffic by doing fatigue tests on dynamical welded connections from orthotropic decks.

The Dutch part of the research will be organized as follows.

1.2.1 Fatigue strength of field splices in longitudinal stiffeners of the orthotropic deck plate

Constant amplitude tests on six full scale testspecimens will be carried out at low stress range levels, to define the lower part of the fatigue design curve for stress ranges beyond 2.10^6 or 10^7 cycles.

Also variable amplitude loading on two test specimens using a load or stress spectrum obtained from frist and second phase ECCS-research. Special attention will be paid to long life (low stress) fatigue results (10^8 cycles).

Testing the miner summation by comparing above mentioned test results.

1.2.2 Fatigue strength of stiffener to cross-beam connection of the orthotropic deck plate

Constant amplitude tests on full scale test specimens with different types of geometry to get an agreement of the fatigue behaviour of these connections.

1.2.3 Recommendations for the fatigue calculation of bridge details

Using results of the above mentioned fatigue testing and ECSC-work from phase 1 and 2, recommendations will be developed for the fatigue calculation of bridge details, including recommen-dations how to design details in such a way that no fatigue calculation is needed.

1.3 Choice of details

1.3.1 Field splices in longitudinal stiffeners of the orthotropic steel deck plate

A few years ago, a research program was set up to investigate the design of field splices in orthotropic steel bridge decks with respect to economy and fatigue strength. Laboratory tests were conducted at IBBC-TNO and at the Stevin Laboratory [9]. Three types of field splices were selected, as shown in figure 1.

- Type A : Butt splice with back-up strips
- Type B : Lap splice with fillet welds
- Type C : Butt splice without back-up strips, but with a thick joint plate.

Testing was carried out on one-rib specimens with a complete rib splice, in pure bending under various constant amplitude loading and also with strip specimens cut out of a spliced rib (see fig.2).

In all the tests, the same trapezodial rib was used but it is assumed that the qualitative results apply to other ribs as well.

The results on the bending tests with constant amplitude loading have been plotted in a S-N-diagram on a log-log scale in figure 3. The conclusion can be drawn that the fatigue behaviour of type C is the best and of type B is the worst, with type A in between. Nevertheless, with respect to type C, it must be noted that apparently this type is very sensitive to the quality of the workmanship, for the first series of specimens made with less care gave very bad results. That is one of the reasons why nowadays type A is often used.

The results on the testing of the strip specimens with constant amplitude loading have been plotted in a S-N-diagram on a log-log scale in figure 4.
From the tests on strip specimens the conclusion emerges that strip specimens are not suited for assessing the fatigue strengths of rib splices. If this would not be true, the much cheaper strip tests could be used in future test programs.

As all tests were carried out at high stress range levels where the shape of the S-N-curve amounts about 3, no information is available at the lower stress regions and on the fatigue limit. This part of the S-N-curve is very important because some of the components of the bridges will endure a large number of stress cycles (10^8-10^9). Therefore, our intention is to realize six fatigue tests at low stress range levels to define the lower part of the S-N-curve of previous mentioned detail-geometry corresponding type A (figure 6). Besides, two tests will be done under variable amplitude loading.

The advantage of doing tests on this detail is, that with a limited number of fatigue tests the fatigue behaviour of this detail is known very accurate and reproducible to comparable details of which less information is available.

1.3.2 Stiffener to cross-beam connection of the orthotropic steel deck plate

With respect to the rib to cross-girder connection we intend to investigate the fatigue behaviour of a rolled trapezprofile which is connected to the cross-girder in three different ways, according to figure 5.

- Type CR-I : connection without cutting out
- Type CR-II : connection with small cutting out
- Type CR-III : connection with particular cutting out

In a lot of bridges type CR-I and CR-II are mostly used. However, recent research [10] on the fatigue strength of an orthotropic steel plate deck with trapezoidal closed longitudinal ribs intended for use in railway bridges suggests an improved form of the outcuts in the web of the cross-beam according to type CR-III. First railway bridge of this new orthotropic deck design was installed in 1983 [11].

The main purpose of our investigation is to get an agreement of stress distributions and fatigue behaviour of these connections for road bridges and <u>not</u> to define a fatigue design curve.

For each type two test specimens will be manufactured.

2. WORK IN PROGRESS

2.1 Field splices in longitudinal stiffeners of the orthropic steel deck plate.

In the period under view most of the time was spend by collecting the data of the variable amplitude tests such as strain gauge measurements and crack growth measurements of the fatigue specimens representing the field splice in the longitudinal stiffner of the orthotropic steel deckplate.

2.1.1 Testing programme

A review of the testing programme including the constant amplitude tests as well as the variable amplitude tests are given in the scheme of Table I. The normal stress in the bottom of the rib in the middle of the splice, due to pure bending, was chosen as the main stress parameter in the programme. This stress was defined by the strain gauge numbered 1000.

2.1.2 Constant amplitude tests

In the previous technical reports results of the constant amplitude tests have been reported already. The test results are tabulated in Table II and presented by plotting in a S-N curve, on a double- log scale in Figure 7.

A short review is given in chapter 2.1.2.1.

2.1.2.1. Fatigue results and discussion

Comparing the results of the fatigue tests it appears that a welded construction with a root gap of 0 mm gives a fatigue life far below the welded construction with a root gap of 4 mm.

A welded detail with a root gap of 2 mm is very dangerous because at a level of 163 N/mm^2 it gives a fatigue life comparable with the fatigue life of a detail with a 4 mm root gap. At a lower stress range level, however, the fatigue life is far less.

Changing the weld geometry by using a V-groove did not give the expected improvement. However, it is clear that it is easier for the welder to make a better weld by using a V-groove. A disadvantage is, of course, the fact that you have to prepare both sides of the detail and it is necessary using more welding material.

From figure 8 (specimens with a rootgap of 4 mm) it appears that the fatigue failures of the specimens tested at the stress range level of 150 and 233 N/mm^2 fall between the spread band of the previous tests [9]. So, it can be concluded that the fatigue results of the previous tests (15 tests from [9]) can be used for the fatigue analysis of the ECSC third phase specimens with a root gap of 4 mm.
At a stress range level of 105 N/mm^2 the fatigue life is far more than one would expect. The fatigue failure occurred over 9 million cycles instead of about 3 million cycles.

The specimen tested at a stress range level of 90 N/mm did not fail after almost 30 million cycles.

From the above mentioned results it can be concluded at the moment, that for this type of construction we are almost in the neighbourhood of the constant amplitude fatigue limit. More constant amplitude tests at a stress range level < 90 N/mm^2 is not usefull because it is to be expected that no failures will occur. Therefore, we have decided to use the remaining two test specimens for the variable amplitude tests. That means a change of the original testing programme.

2.1.2.2. Comparison with Eurocode 3.

In figure 9 the fatigue results are compared with the design S-N curves of the Eurocode 3. Ignoring the fact that cracks did not initiates at the bottom side of the rib, it can be concluded, that for the rib splice with a root gap of 4 mm a weld class 80 can be recommended. If the crack initiation point must be taken into account the classification decreases to a weld class 63.

2.1.2.3. Comparison with other research programs.

Results of the Italian partners in this ECSC-research [13] is given in figure 10. Some of the Italian results seemed to be better than the best fatigue strength of the Dutch details.

In the final report a comparison will be made between the ECSC research and recent research carried out in Japan [14] and United Kingdom [15]. The fatigue results of these research programmes are gathered and presenting by plotting S-N curves in figure 11 and 12.

2.1.3. Variable amplitude tests

Phase 1 and 2 measurements on this type of connections showed that the maximum stress amounts 80 N/mm^2. Knowing the results of the constant amplitude tests it is clear that all stress ranges are below the constant amplitude fatique limit. So it is to be expected that a variable amplitude test with a maximum stress range of 80 N/mm^2 results in a unbroken test specimen.

However, during the lifetime of a bridge higher stress ranges can occur. So it is interesting to do variable amplitude tests with a maximum stress of about 120 N/mm^2 and a measured stress range distribution. Probably it will take a lot of time executing such a test. Therefore an orientating variable amplitude tests was done.

2.1.3.1 Orientating variable amplitude test

The constant amplitude test of specimen A.1.3 at 90 N/mm^2 did not show any crack after $29*10^6$ cycles. This specimen was used to do an orientating variable amplitude test.

Knowing the results from Tromp [9] the following spectrum was chosen:

VER 2 - spectrum

$\Delta\sigma_i$ (N/mm^2)	N_i (1 block)
84	82000
108	48000
132	13800
156	5400

180	1000
204	100

$\Sigma n_i = 150.300$

This spectrum was used as a pseudo random signal. After 2.082.875 cycles no cracks were found and it was decided to continue the test leaving out the two classes (VER 2'-spectrum)

Now a crack was discovered after another 1.415.718 cycles.

To compare the variable amplitude stress spectrum with the constant amplitude tests the stress range spectrum was analysed in two different ways:

A. An equivalent stress range $\Delta\sigma_e$ was calculated in a way that N-cycles of that stress range has the same fatigue damaging potential as N-cycles of the stress spectrum, using a third power relationship.

$$\Delta\sigma_e = \left(\frac{1}{\Sigma n_i} \Sigma n_i \Delta\sigma_i^3\right)^{1/3} \ (\mathrm{N/mm^2})$$

B. The final reports of the Université de Liège [6] proposed the following equivalent stress range $\Delta\sigma m$ and belong number of cycles n_m.

$$\Delta\sigma m = \frac{\Sigma n_i \Delta\sigma_i^4}{\Sigma n_i \Delta\sigma_i^3} \ (\mathrm{N/mm^2})$$

$$n_m = \frac{\Sigma n_i \Delta\sigma_i^3}{\Delta\sigma_m^3}$$

in which: $\Delta\sigma_i$ = individual stress range

n_i = individual number of cycles belong to $\Delta\sigma_i$

$\Delta\sigma_m$ = equivalent stress range

n_m = number of cycles belonging to $\Delta\sigma_m$.

The results of above mentioned test are given in figure 13. It appears that using Miners' Rule the variable amplitude test result falls just outside the scatterband of the constant amplitude tests.
Furthermore it is clear that for the <u>first</u> definite variable amplitude test it is necessary to define almost all stress range levels above 80 N/mm^2.

2.1.3.2. <u>Definite variable amplitude tests.</u>

2.1.3.2.1 <u>Test spectra.</u>

In the Working Group meetings it was decided to use loadspectra simulated by Mr. Bruls. Results of the simulation for the Dutch detail are given in the second progress report of the Université de Liège [16].

Analysis of the spectrum (see figure 14) shows that the stress ranges < 18 N/mm^2 cause only 7% of the total damage of the spectrum. These classes amounts however 83.6% of the total number of cycles. So leaving out these classes saves a lot of testing time. The remaining stress range (18 N/mm^2 - 82 N/mm^2) had to be raised 4 times to realise a level almost above the constant amplitude fatigue limit. For testing techniques it was necessary to leave out stress range classes from 264 - 328 N/mm^2. These classes cause 5% of the total damage. The definite spectra which were used, are given in figure 15.

2.1.3.2.2. <u>Results.</u>

The fatigue results of the variable amplitude tests are tabulated in Table III and presented by plotting as a S-N relationship, on a double-log scale, in figure 16 and 17.

These figures also show the constant amplitude test results as well as some S-N Curves according to Eurocode 3. The crack growth measurements are gathered in Appendix A.

Comparing the results it appears that there is a good aggreement between the constant amplitude and variable amplitude tests. One of the tests is still running.

Further analysis will be given in the final report.

2.2 Connection between trapezoidal longitudinal stiffner and the cross beam

2.2.1 Testing programme

As mentioned in chapter 1.3.2 it is our intention to investigate the fatigue behaviour of a rolled trapezprofile which is connected to the cross girder in three different ways (see figure 5).
The main purpose is to get en agreement of stress distributions and fatigue behaviour of the connections for road bridges and not to define a fatigue design curve.
For each type two test specimens will be manufactured.

2.2.2 Field measurements

Before starting the fatige tests some measurements were carried out on an existing bridge to get information about stress distributions in the cross girder and the longitudinal stiffner. Location of the measuring points is given in figure 18.

2.2.3 Fatigue tests

The three specimens were tested until 10 million cycles without crack initiation.The third and last one is running at the moment. Comparison between the measured stresses is given in figure 19. The fatigue load in all the cases was the same. So it possible the measured stresses of the different specimens can be compared.

Becaurse no cracks were found it was decided to built a new testrig which allow us to execute variable amplitude tests or constant amplitude tests at a much higher level. Testrig as well as the testspectrum are the same as they are using at the moment at the Transport and Road Research Laboratory. More information on this part of the research program will be given in the final report or in an additional report.

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4. LIST OF FIGURES

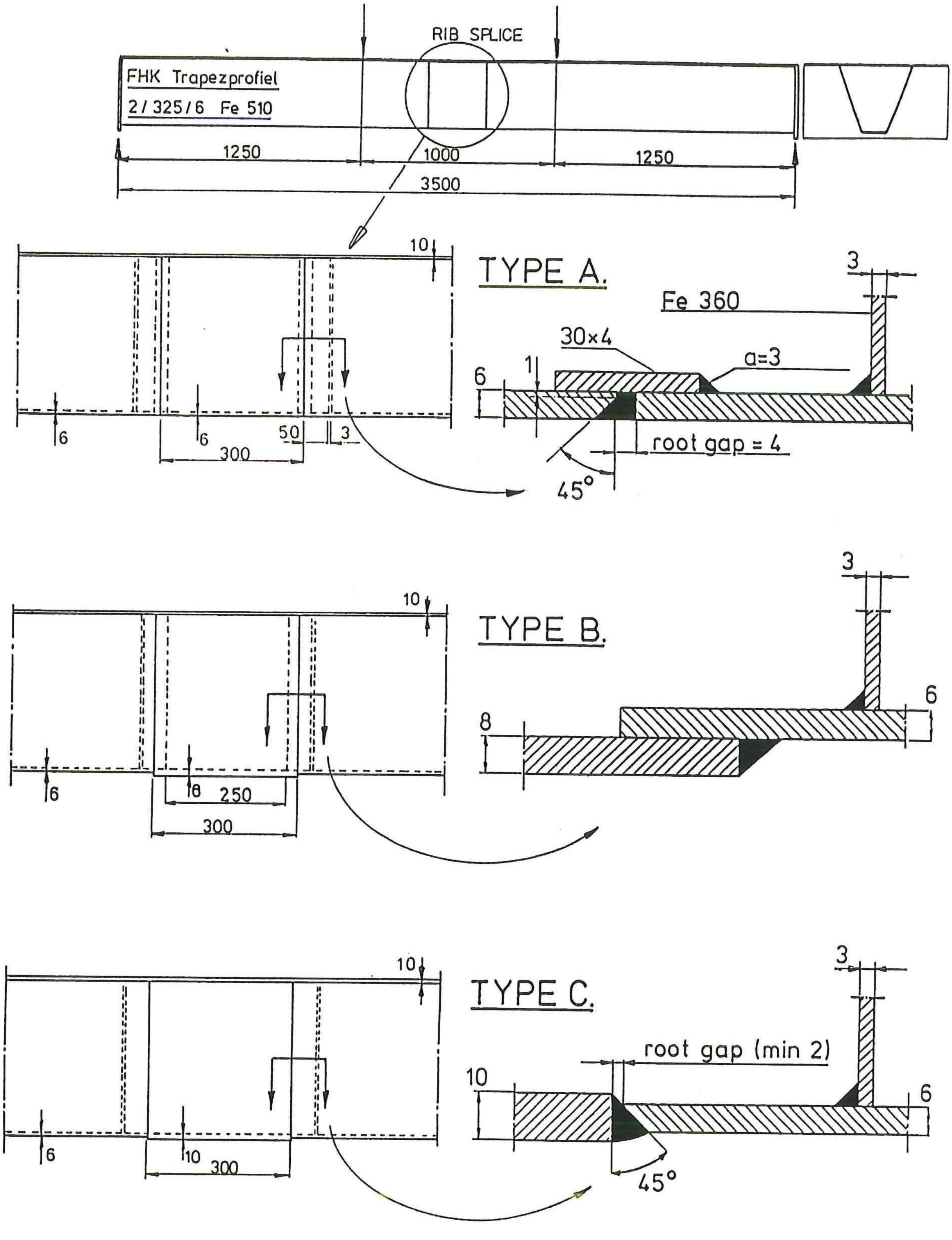

Figure 1 : Tested types of field splices [9].

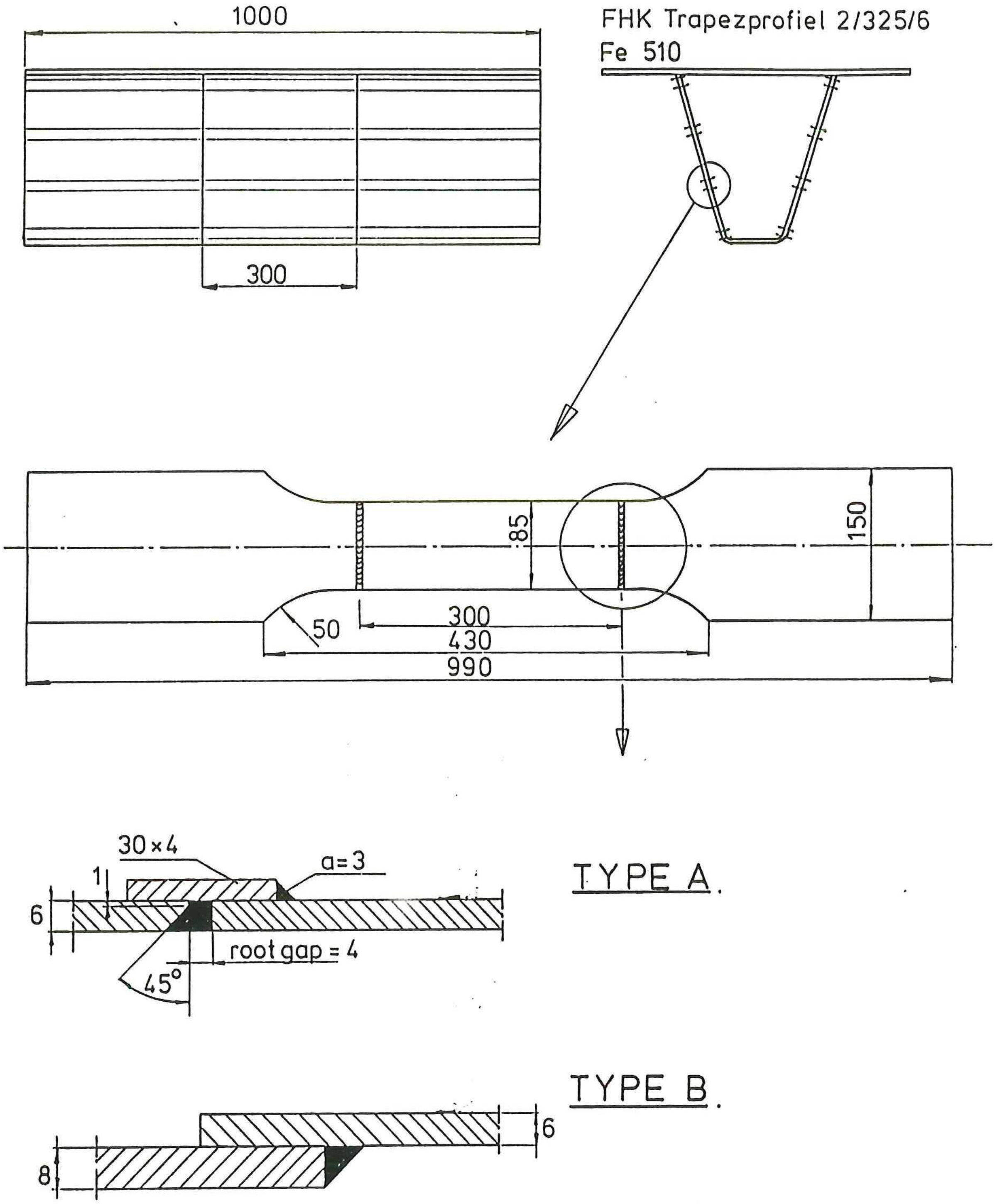

Figure 2 : Dimensions of strip specimens [9].

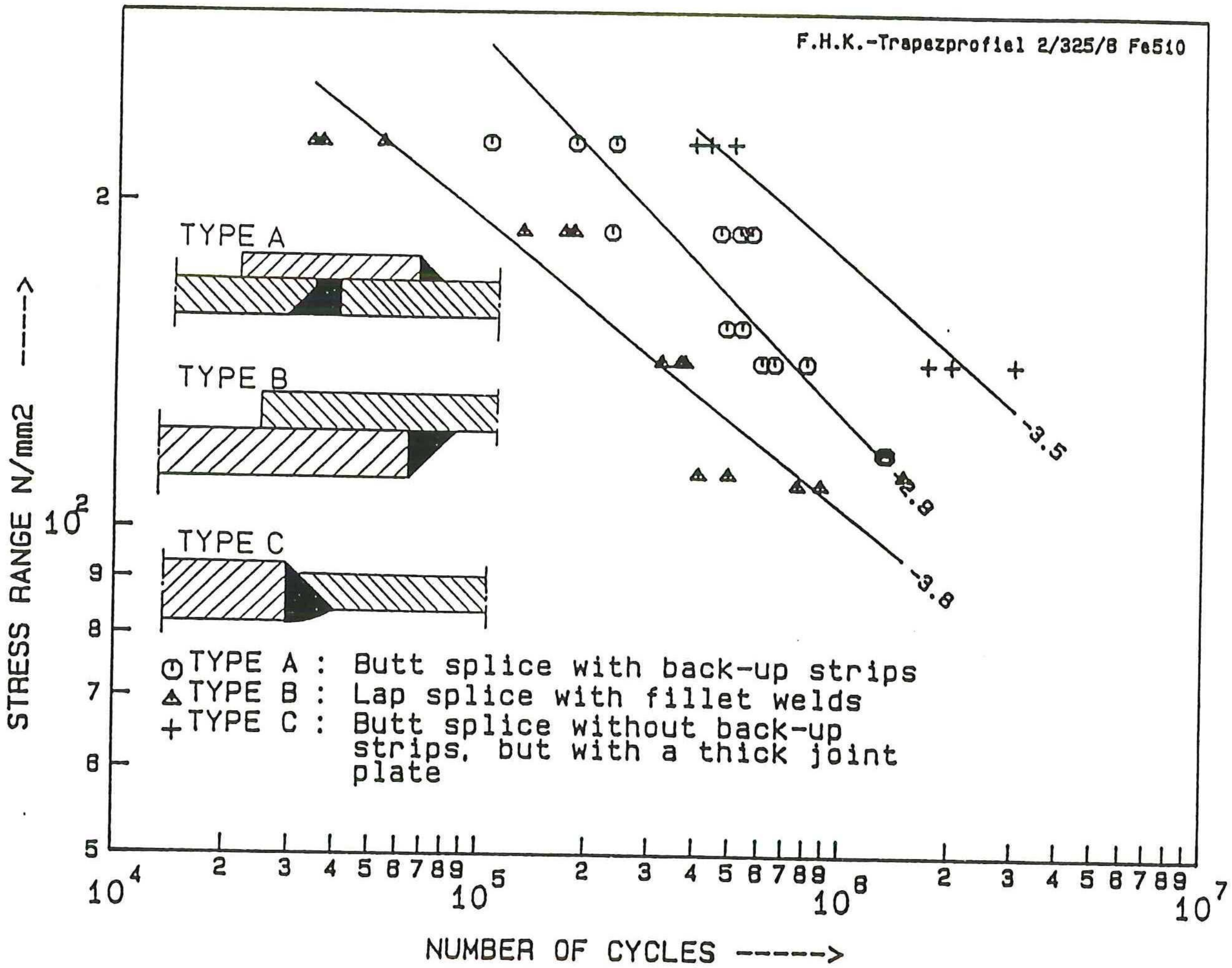

Figure 3 : Results of bending tests [9].

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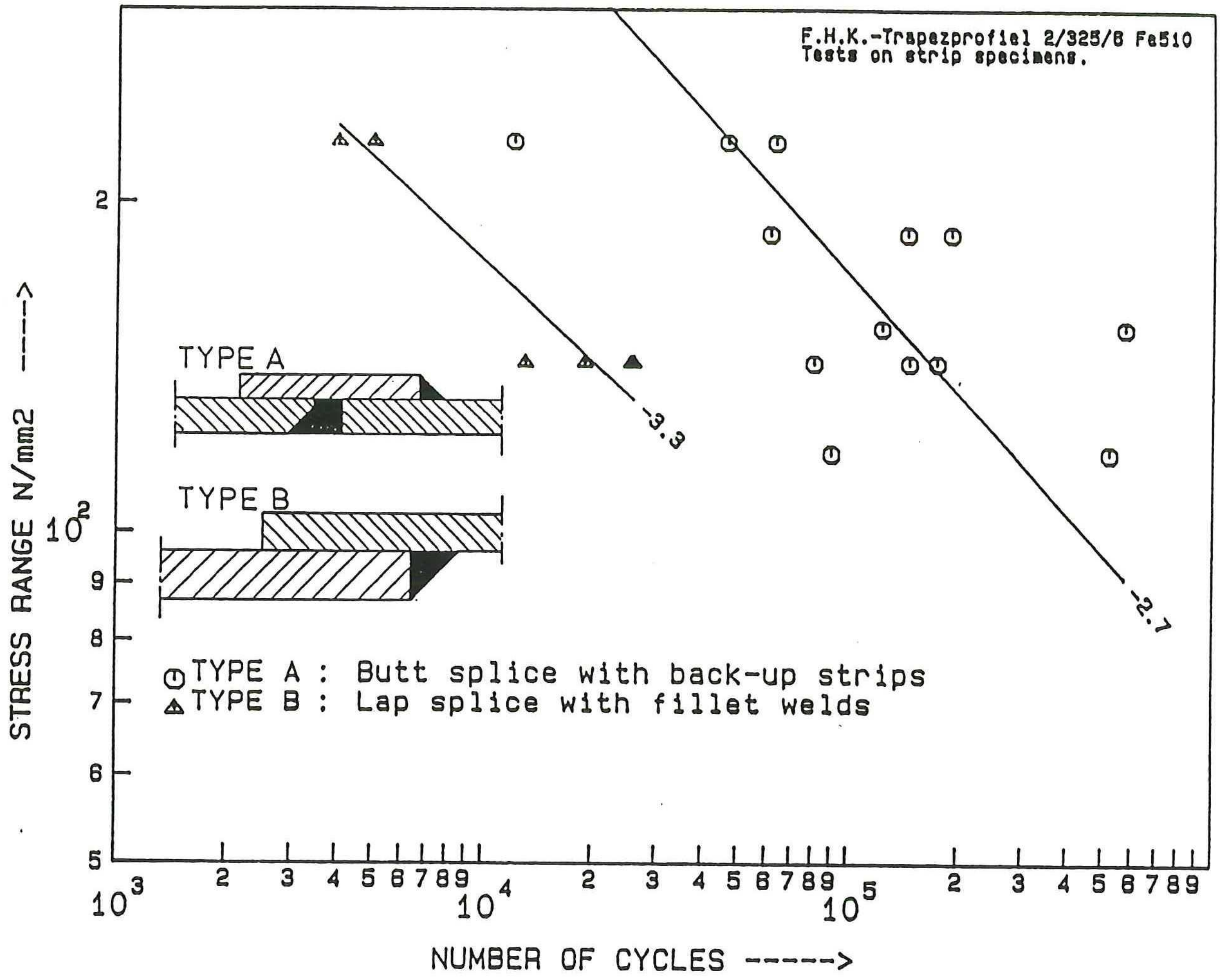


Figure 4 : Results of tests in strip specimens [9].

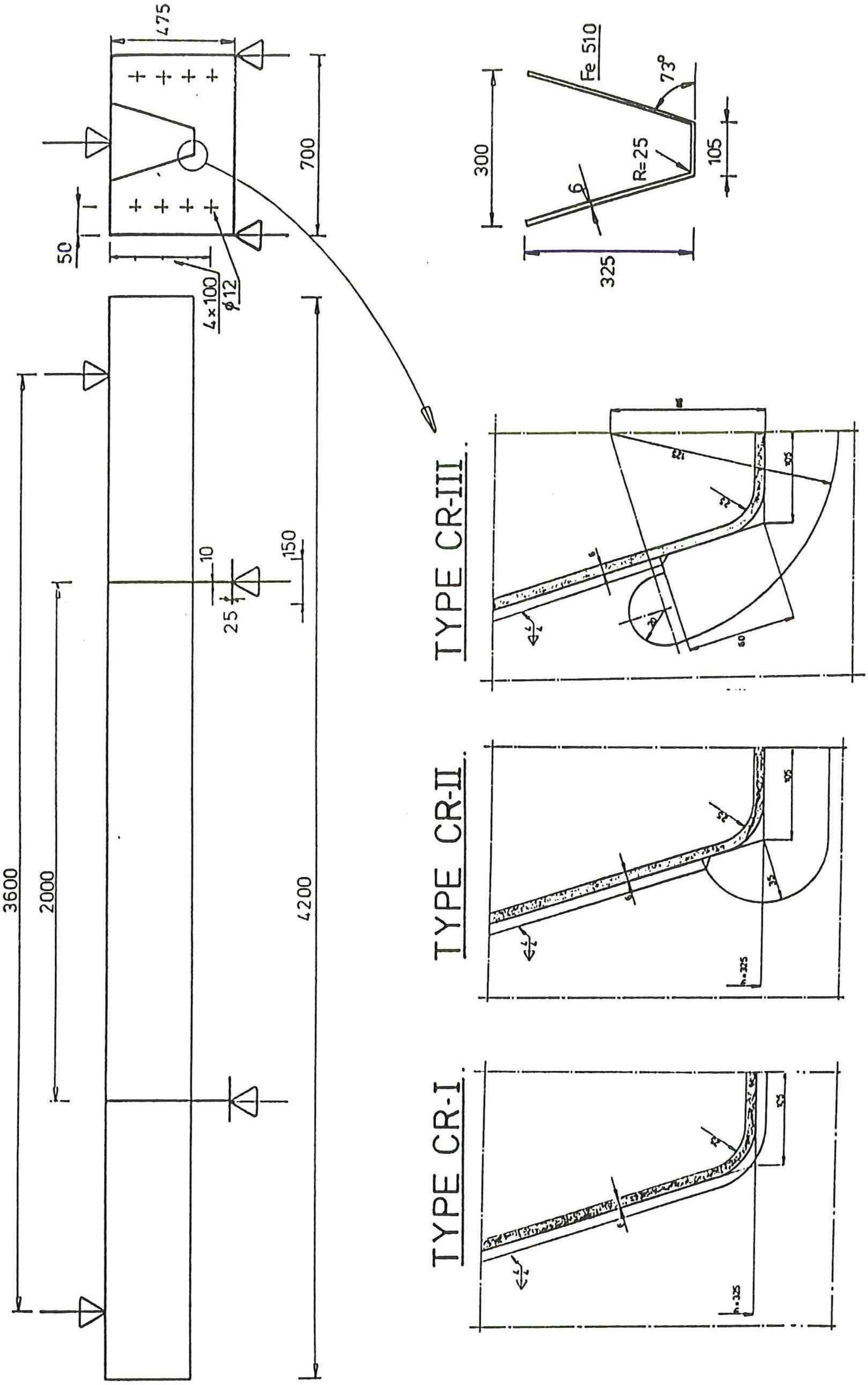


Figure 5 : Selected types of rib to cross girder connections for the ECSC-research program.

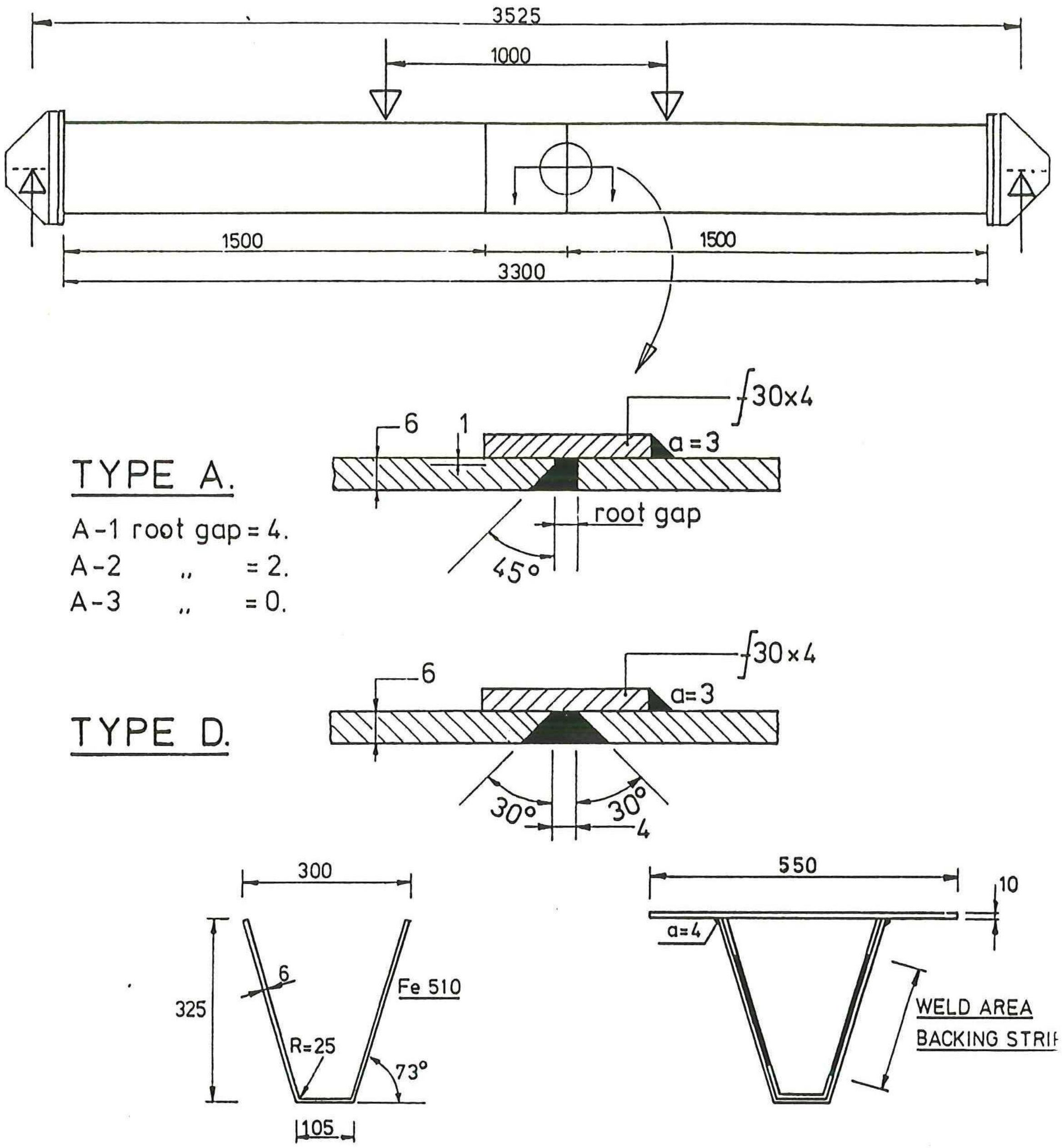


Figure 6 : Selected type of field splices for the ECSC-research program.

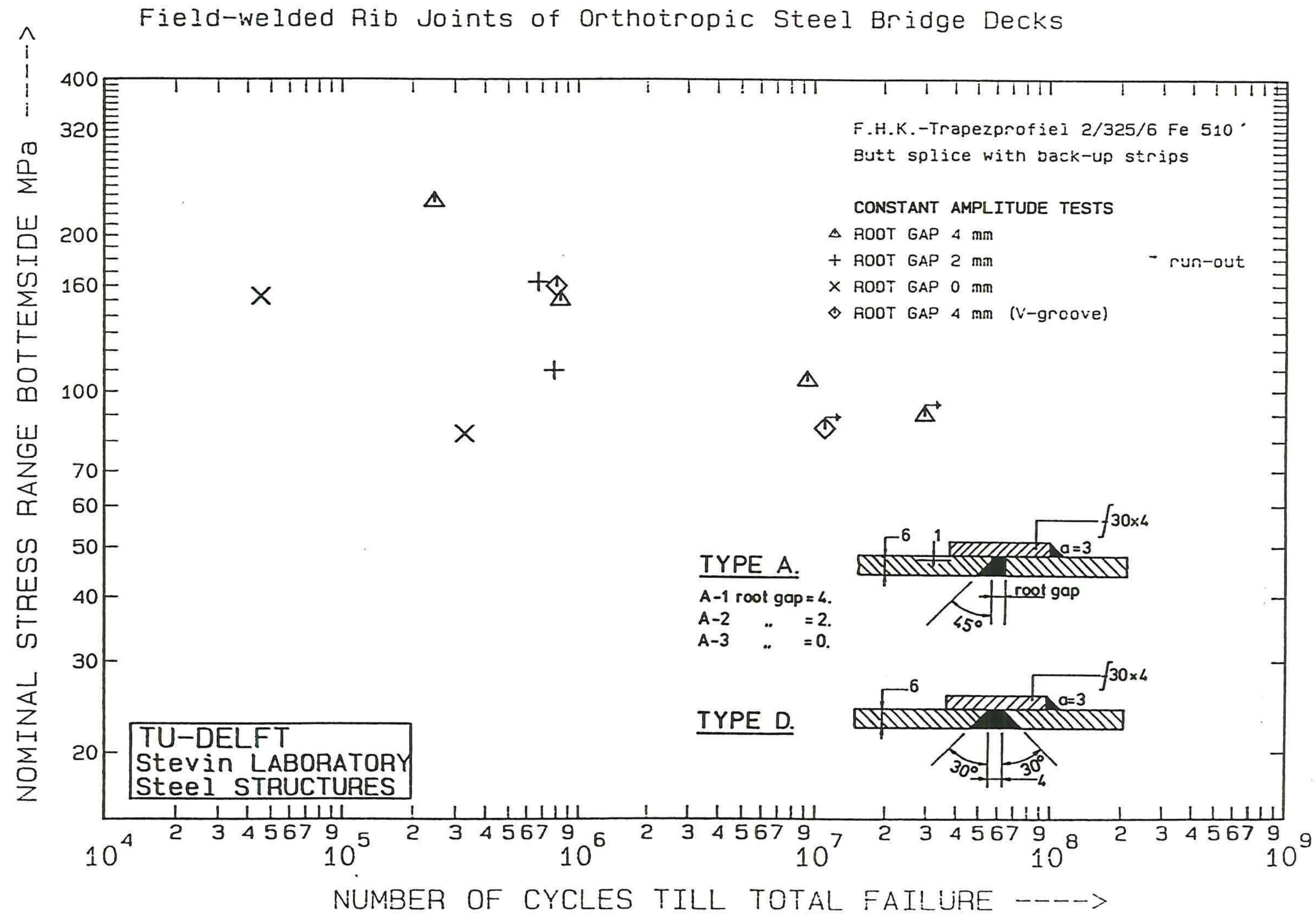


Figure 7 : Results of all constant amplitude tests.

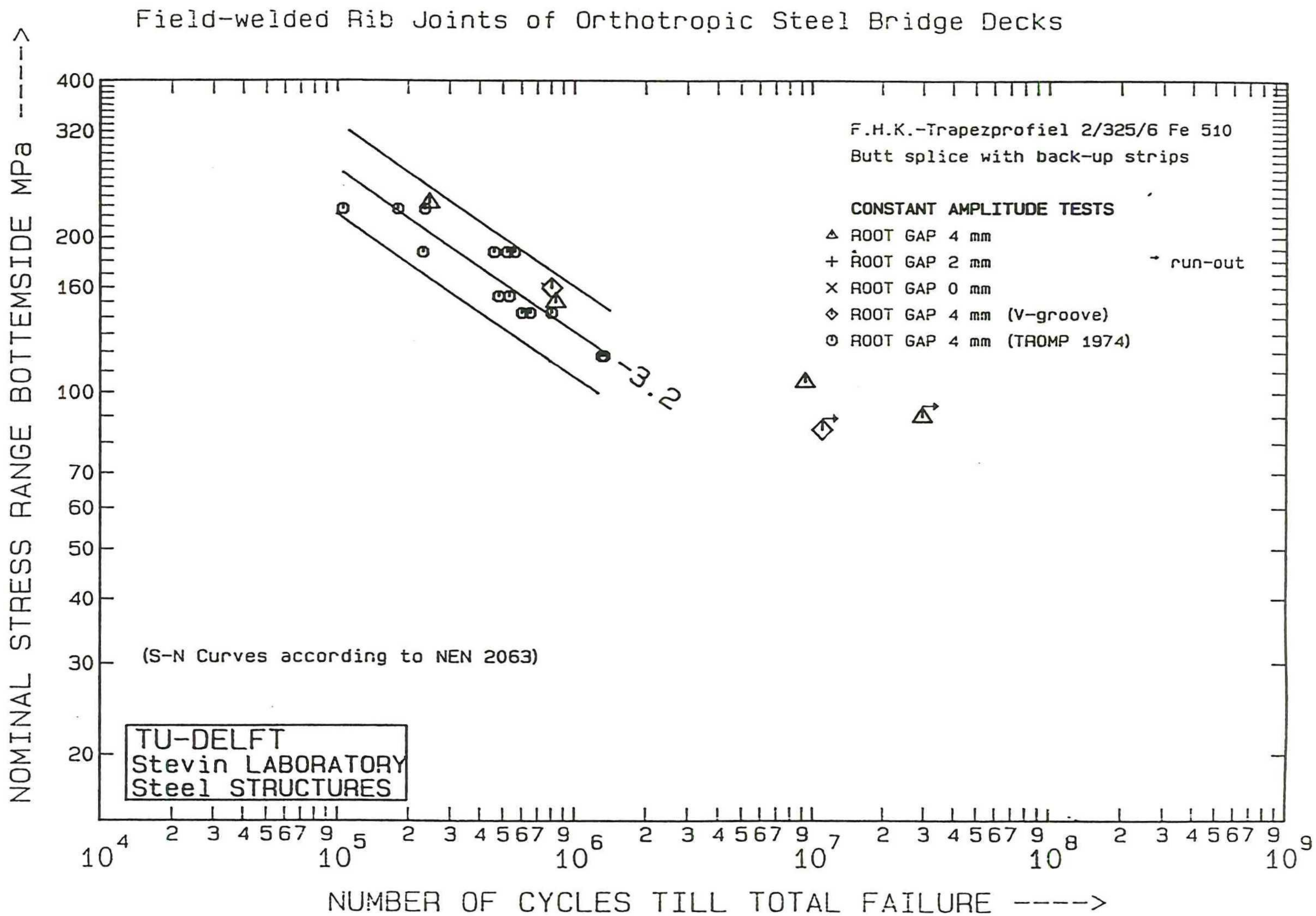


Figure 8 : Results constant amplitude tests-root gap 4 mm.

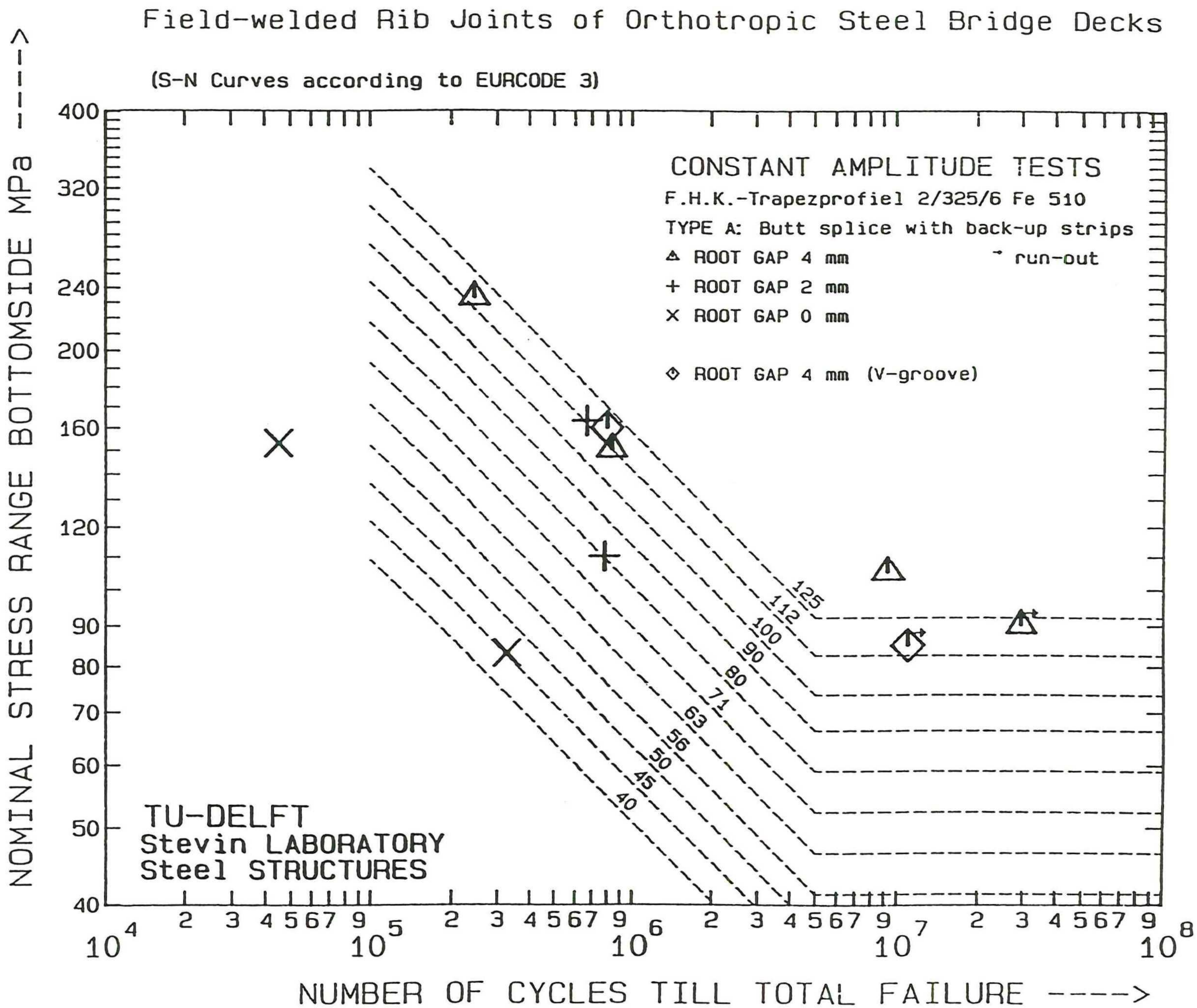


Figure 9 : Fatigue results compared with S-N curves EUROCODE 3

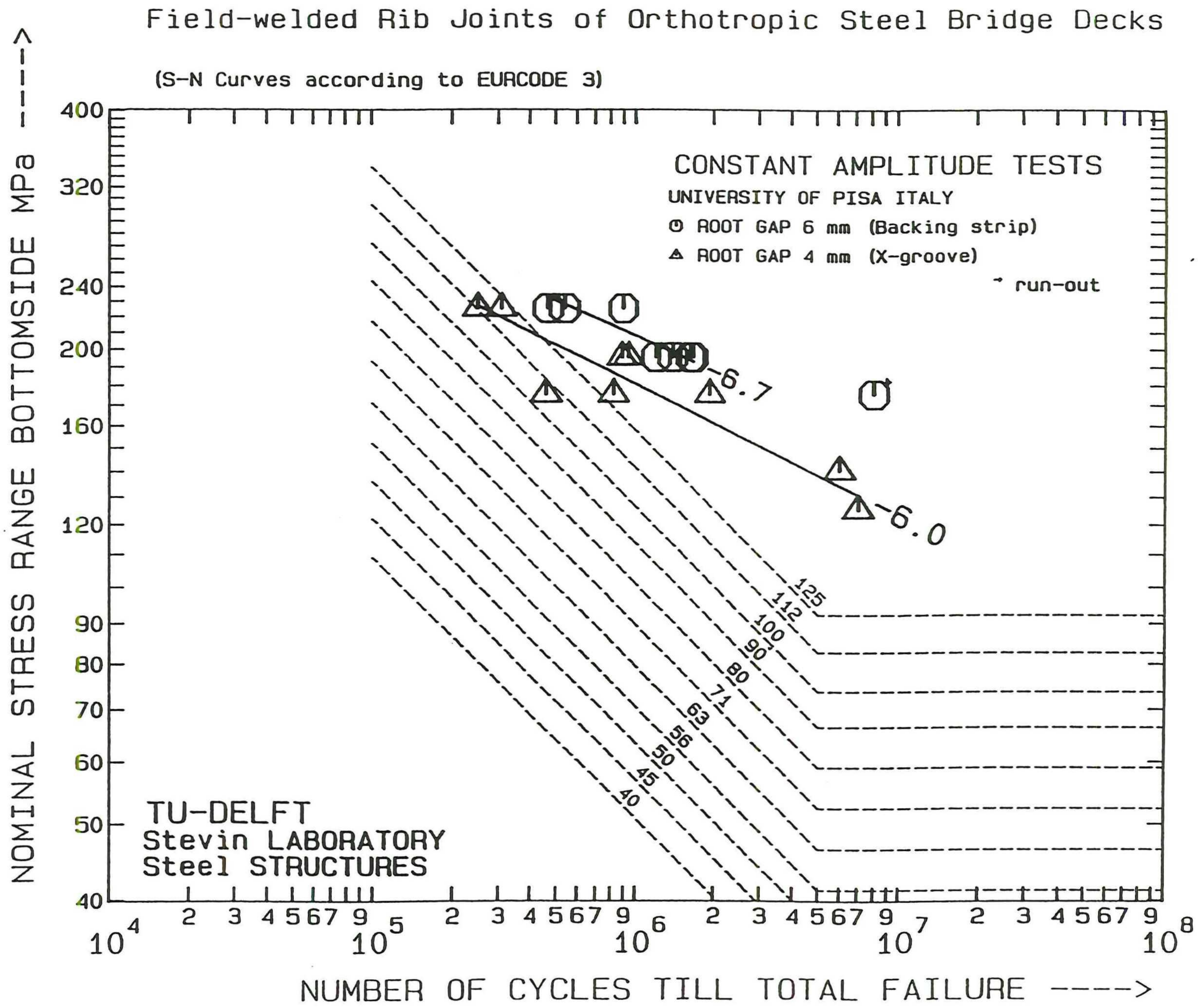


Figure 10 : Comparison with Italian ECSC research.

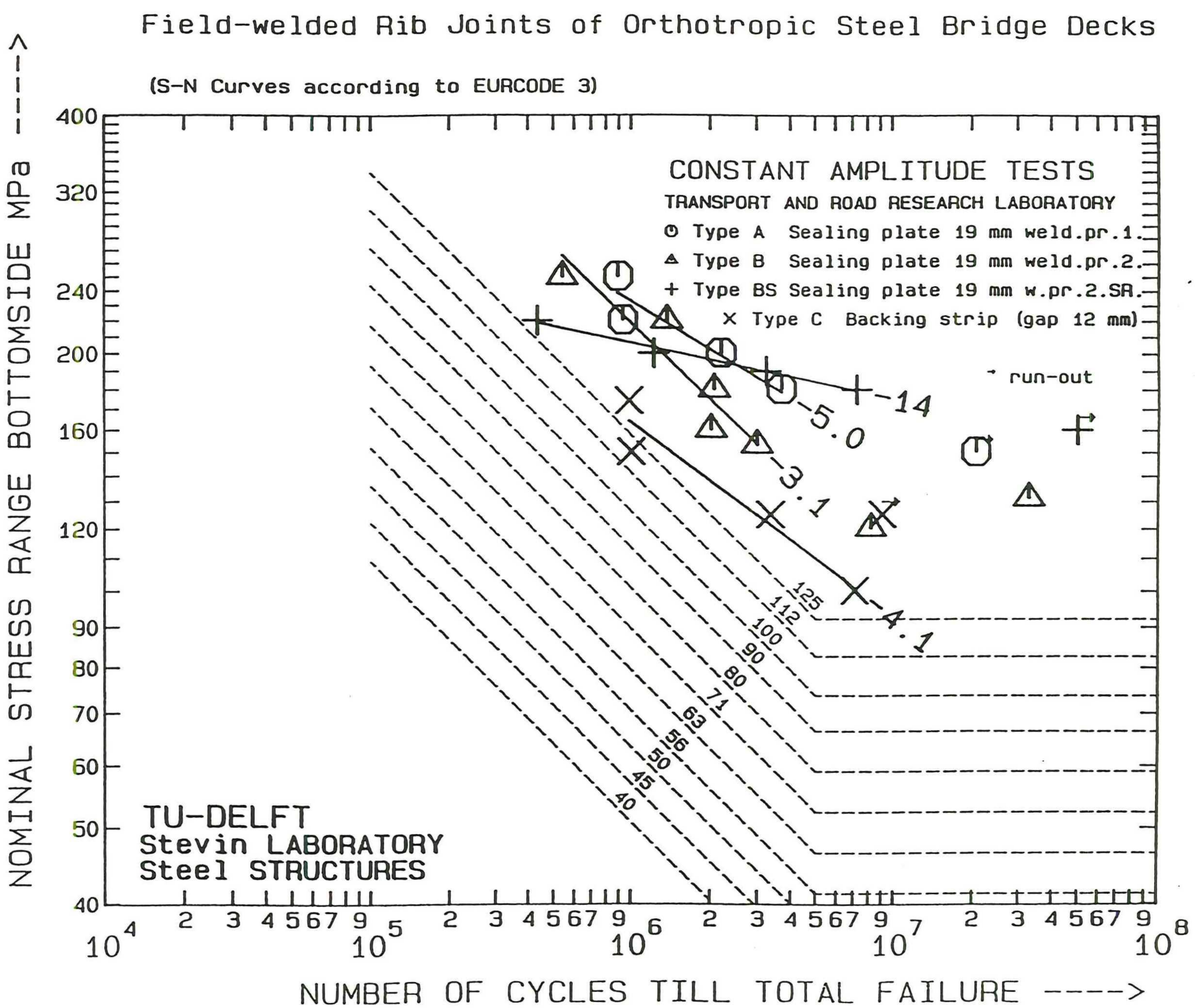


Figure 11 : Fatigue results Cunninghame 1982

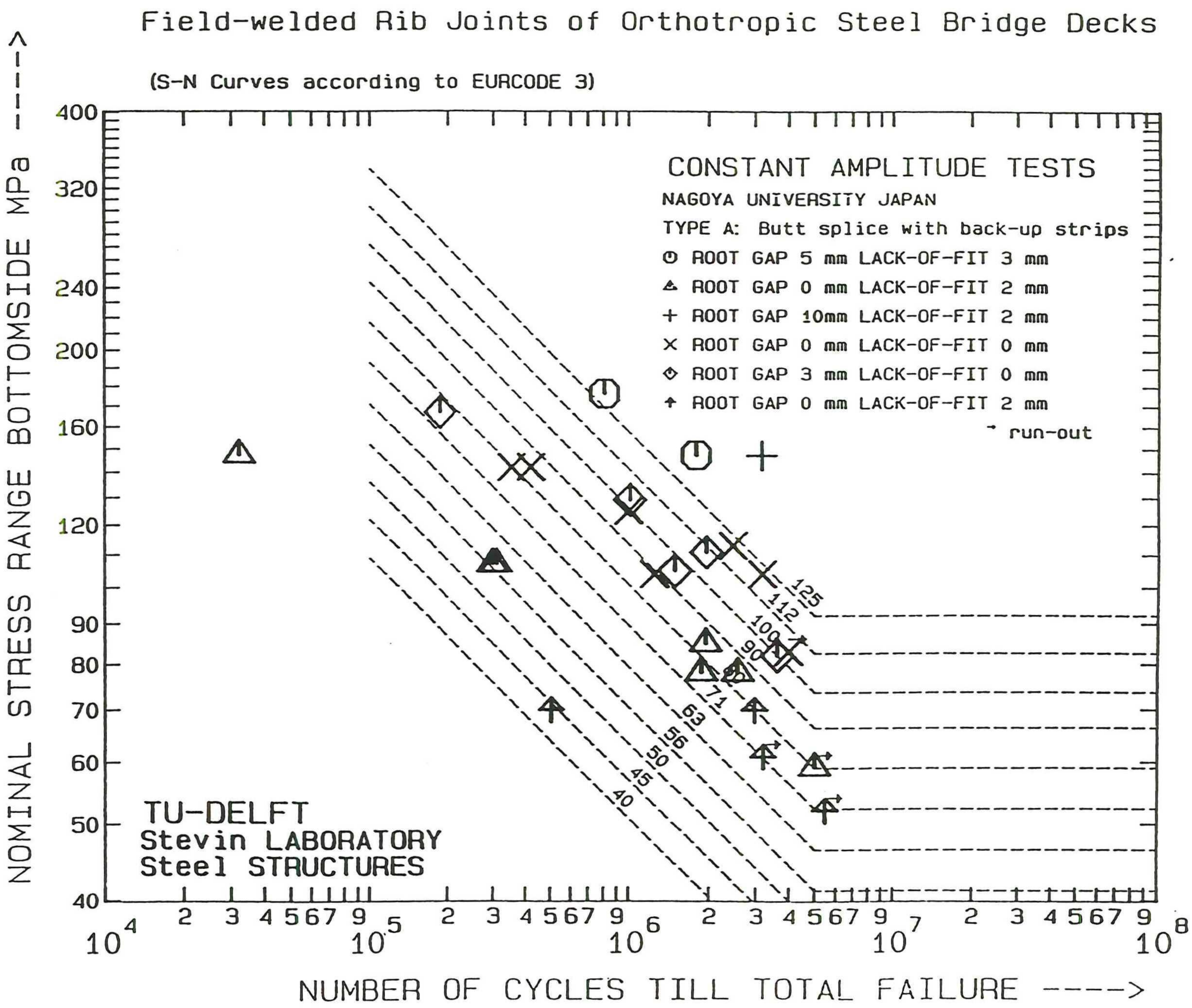


Figure 12 : Fatigue results Yamada 1987

TEST HISTORY SPECIMEN A.1.3						
SPECTRUM		$\Delta\sigma_e$ (N/mm^2)	n_i x10^6	ΔS_m (N/mm^2)	n_m x10^6	REMARKS
CONSTANT AMPLITUDE		90	29	90	29	NO CRACKS
VARIABLE AMPLITUDE	VER 2	104	2.1	114	1.6	NO CRACKS
	VER 2'	143	1.4	145	1.3	WELD FAILURE
	VER 2+2'	123	3.5	134	2.7	
VER2+2'+CA		95	32.5	100	27.1	

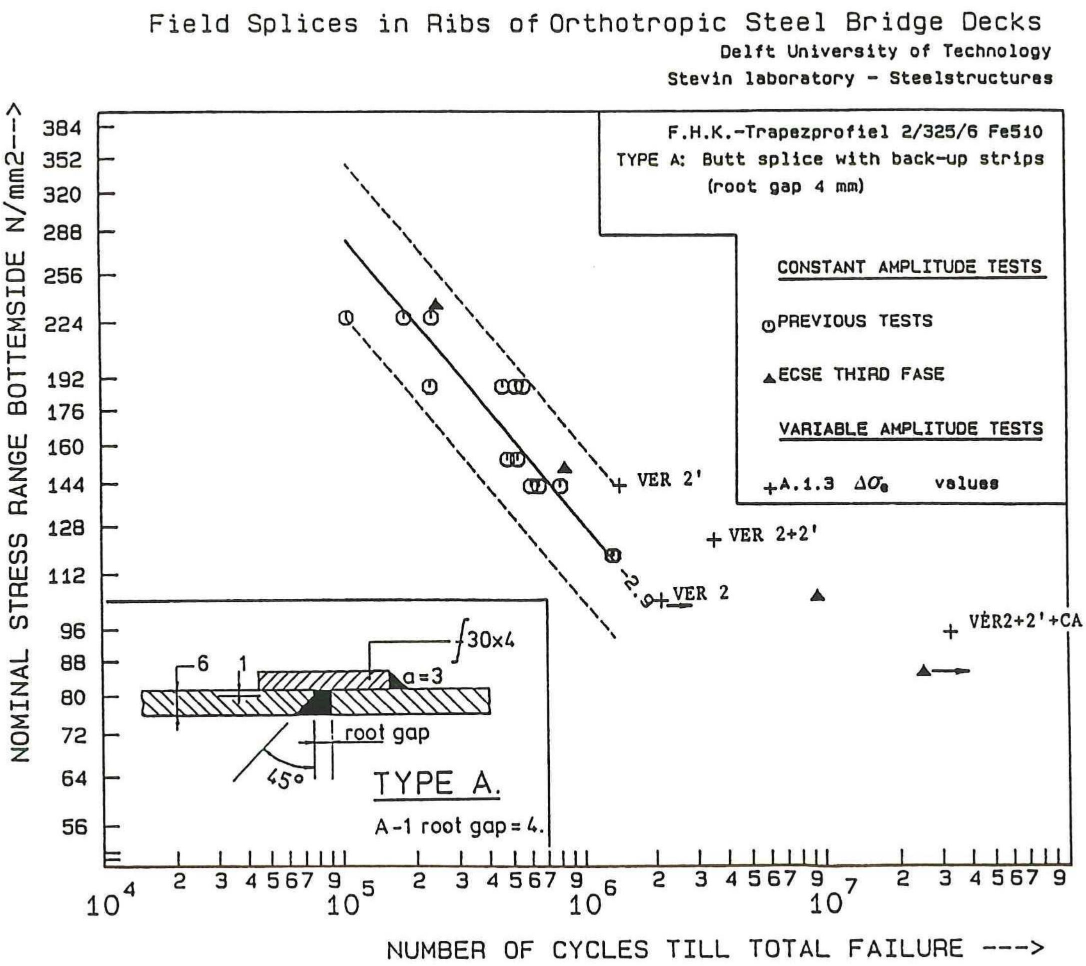
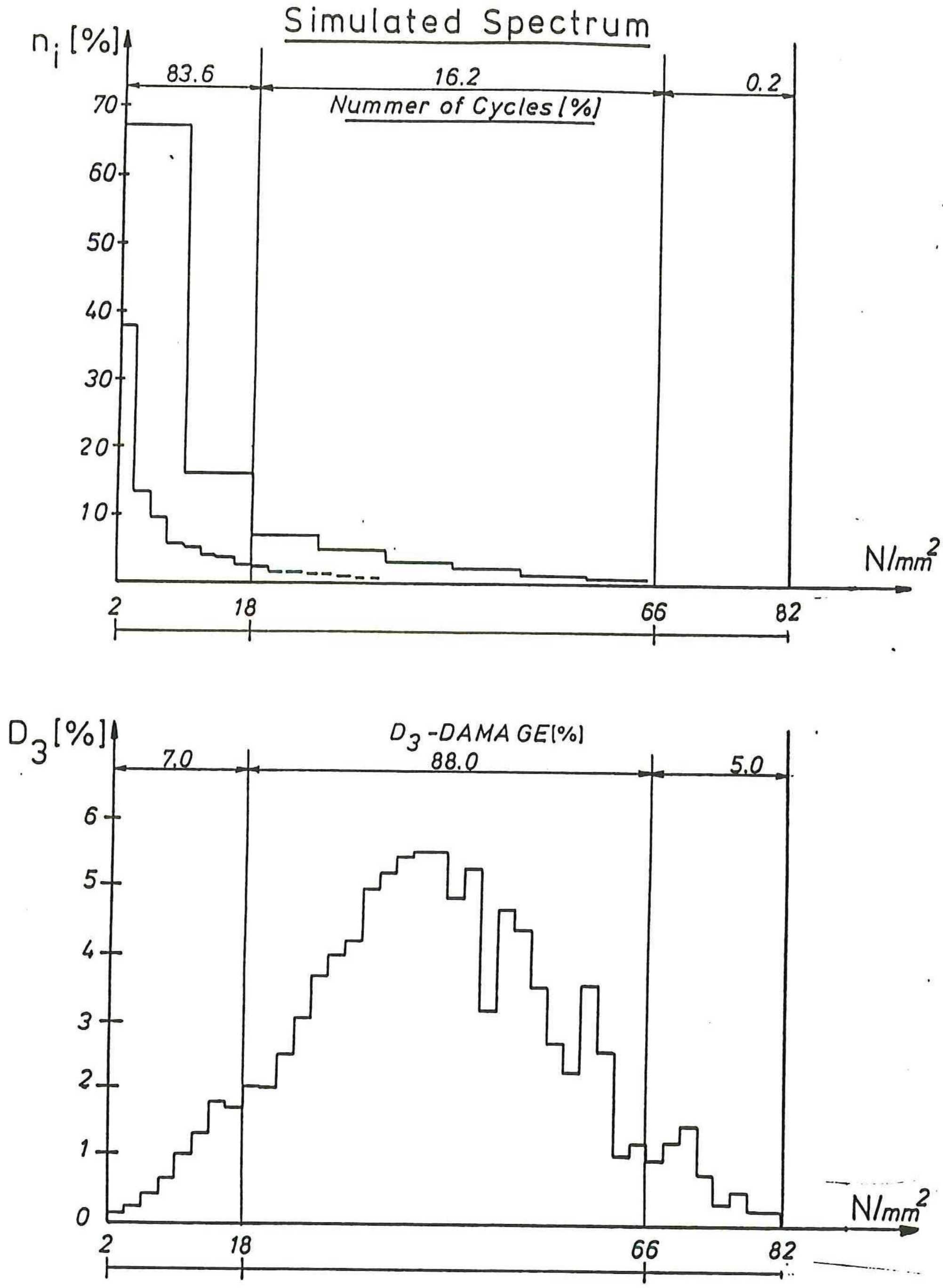


Figure 13 : Results variable amplitude test A.1.3.



$\Sigma n_i = 6138 \quad \Delta\sigma_e = 33.8\ N/mm^2$

$\Delta Sm = 38.3\ N/mm^2, \ n_m = 4450$

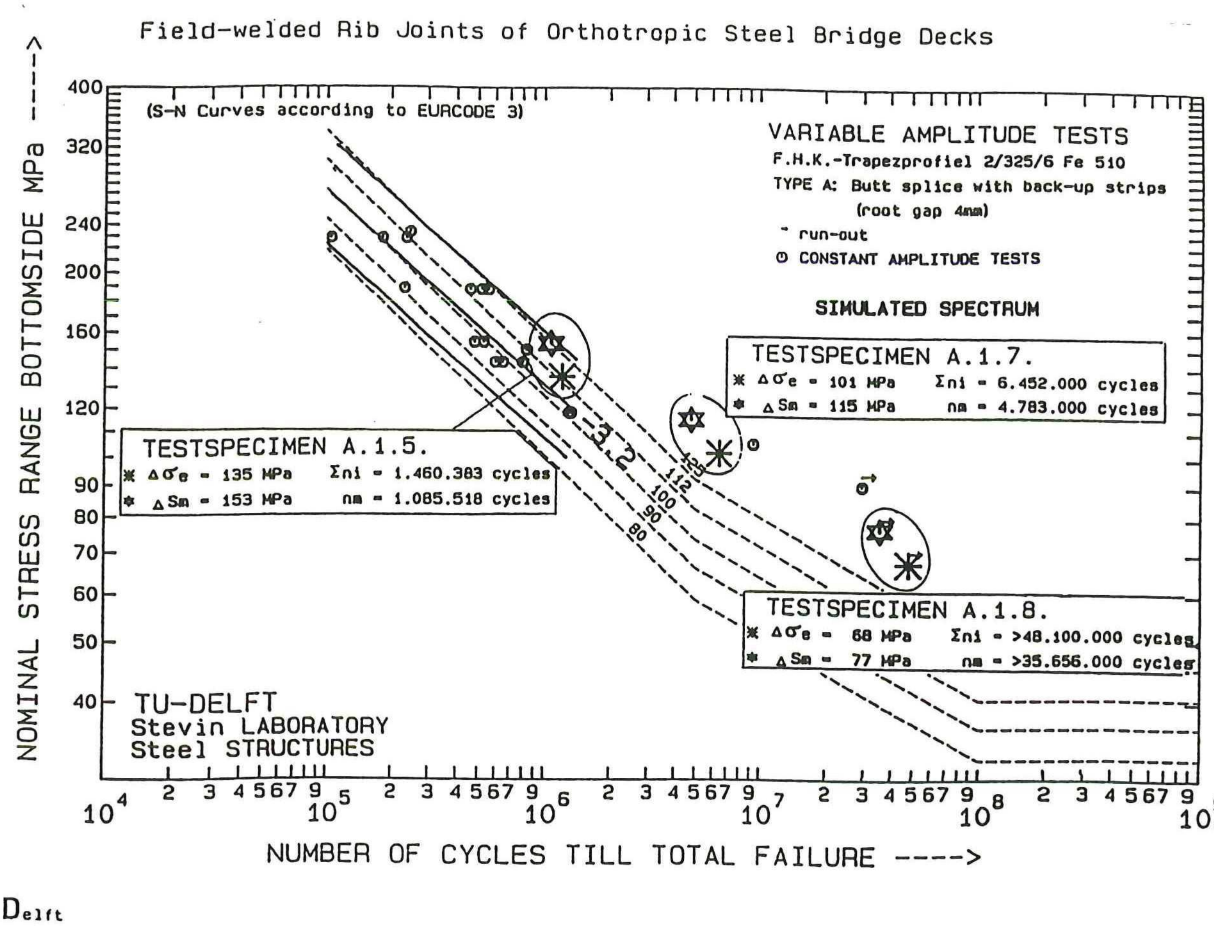
$\Sigma n_i = 37.740 \ \Delta\sigma_e = 18.7\ N/mm^2$

$\Delta Sm = 40.5\ N/mm^2, \ n_m = 4091$

Figure 14 : Simulated spectrum

SIMULATED SPECTRA:				MEASURED SPECTRA:	
NUMBER OF CYCLES	$\Delta\sigma_{i(1000)}$ [MPa] TEST SPECIMEN			NUMBER OF CYCLES	$\Delta\sigma_{i(1000)}$ [MPa] TEST SPECIMEN
n_i	A.1.5. D.1.1.v.	A.1.7.	A.1.8.	n_i	A.1.6.
8130	80	60	40	4086	80
5818	88	66	44	3456	88
5570	96	72	48	3366	96
5410	104	78	52	2502	104
5130	112	84	56	2034	112
4420	120	90	60	1818	120
3900	128	96	64	1818	128
3700	136	102	68	1215	136
3330	144	108	72	1341	144
2900	152	114	76	1044	152
2530	160	120	80	828	160
1970	168	126	84	909	168
1670	176	132	88	810	176
1590	184	138	92	774	184
900	192	144	96	855	192
1090	200	150	100	792	200
900	208	156	104	666	208
640	216	162	108	621	216
440	224	168	112	495	224
330	232	174	116	558	232
480	240	180	120	414	240
310	248	186	124	342	248
110	256	192	128	171	256
120	264	198	132	180	264
Σn_i = 61380				31095	
$\Delta\sigma_e$ [MPa] =	135	101	68		145
ΔS_m [MPa] =	153	115	77		182
n_m = 45610				15840	

Figure 15 : Definite variable amplitude load spectra



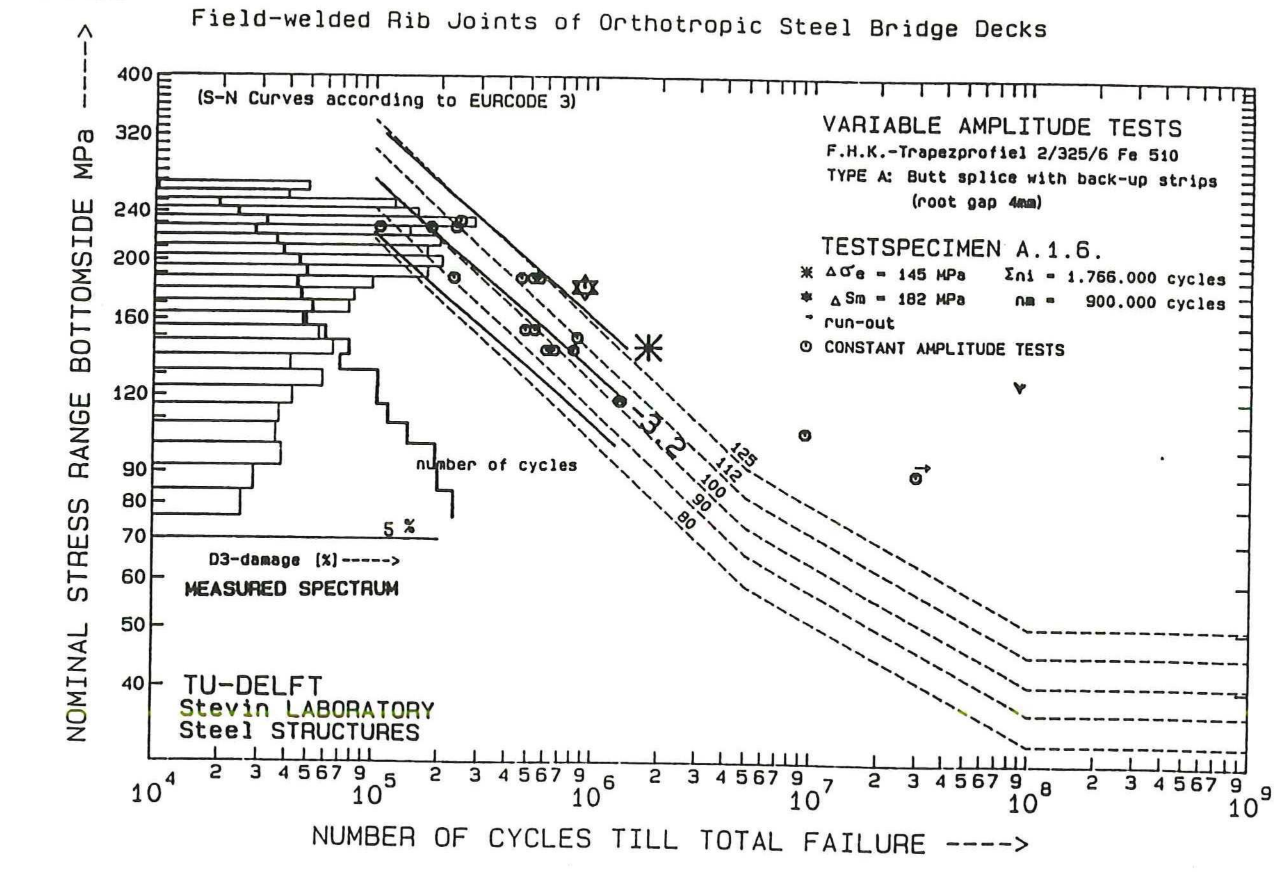
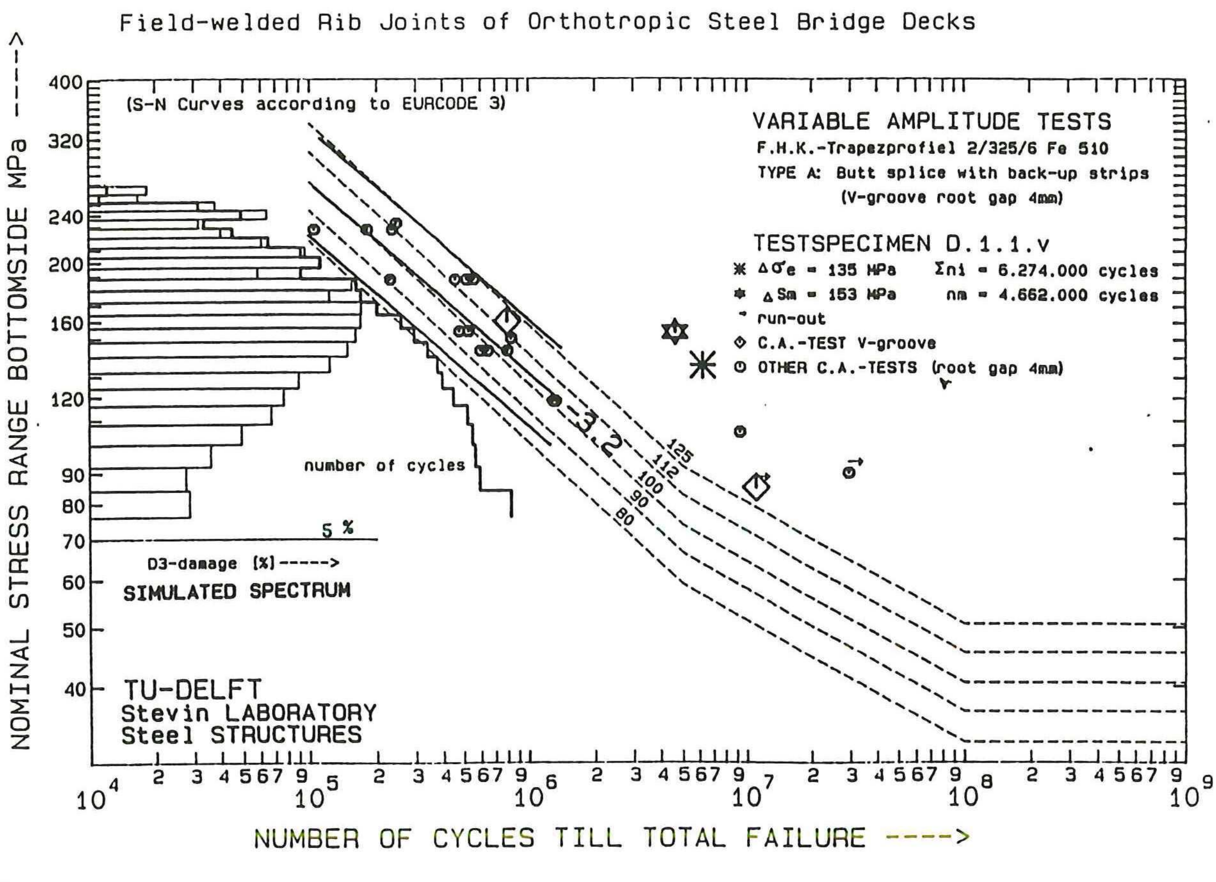


Figure 16 :Results variable amplitude tests root gap 4 mm.



TUDelft

Figure 17 :Results of the variable amplitude test V-groove

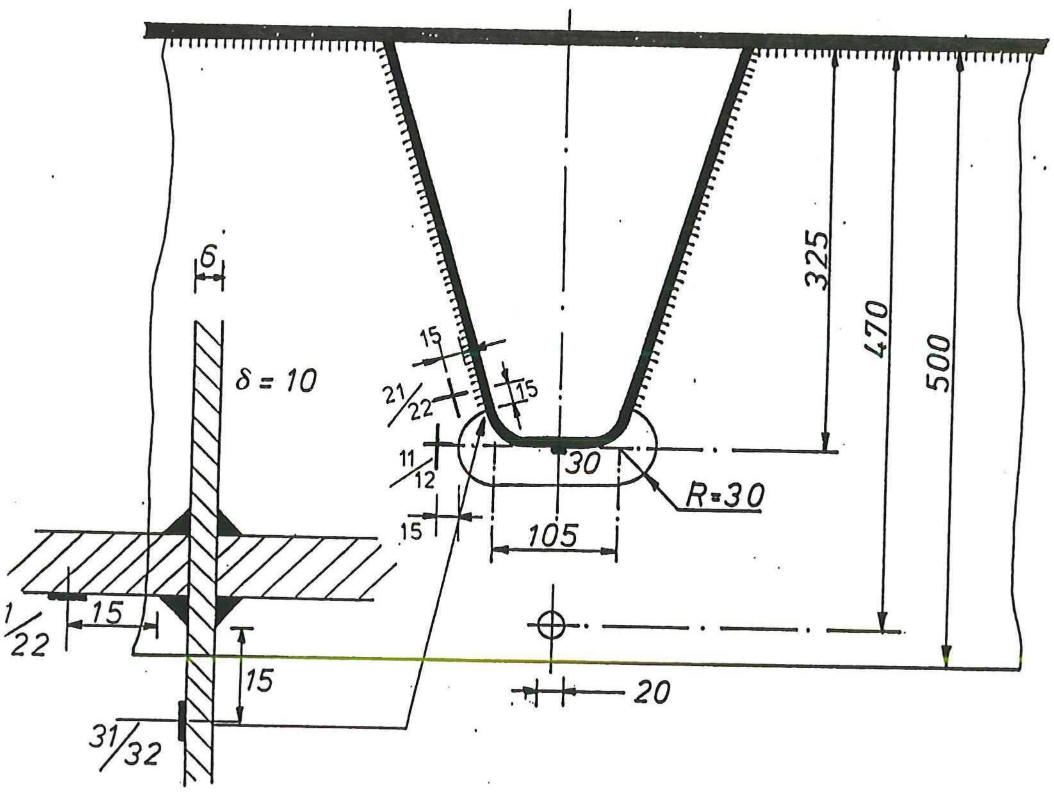


Figure 18 :Strain gauge loacations Moerdijk Bridge

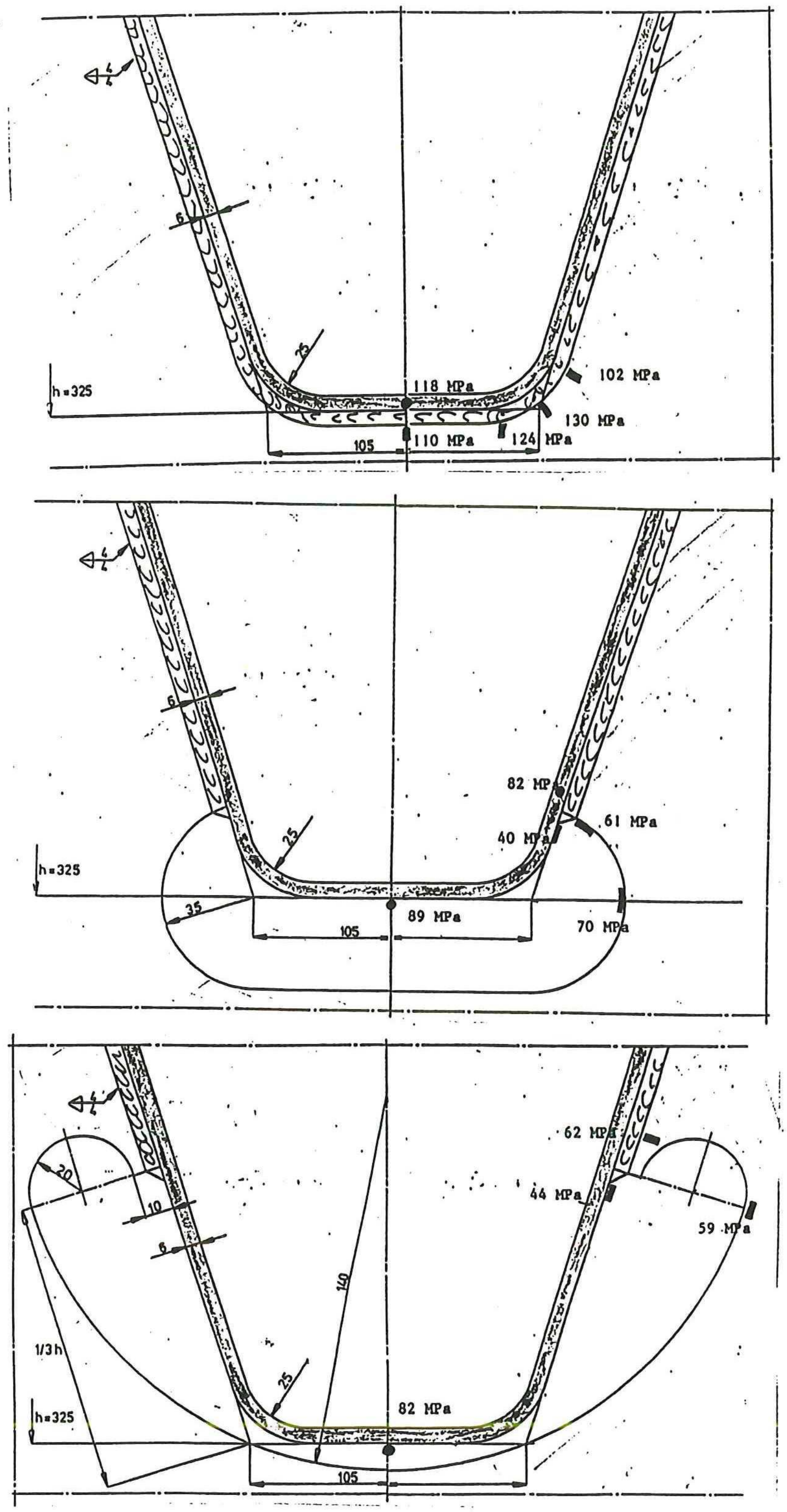


Figure 19 :Measured stresses cross- girder connections.

5. LIST OF TABLES

CONSTANT AMPLITUDE TESTS		
Test specimen	Root gap [mm]	Stress range [MPa]
A.1.1.	4	150
A.1.2.	4	105
A.1.3.	4	90
A.1.4.	4	233
A.2.1.	2	163
A.2.2.	2	110
A.3.1.	0	153
A.3.2.	0	83
D.1.1.(V-groove)	4	85
D.1.2.(V-groove)	4	160

VARIABLE AMPLITUDE TESTS			
Test specimen	Root gap	Spectrum	$\Delta\sigma_e$ [MPa]
A.1.5.	4	Simulated	135
A.1.6.	4	Measured	145
A.1.7.	4	Simulated	101
A.1.8.	4	Simulated	68
D.1.1.v.(V-groove)	4	Simulated	135

Remarks: Stress range = nominal stress range bottomside trough

$$\Delta\sigma_e = \left(\frac{1}{\Sigma\, n_i} \Sigma\, n_i \Delta\sigma_i^3\right)^{1/3} \text{ MPa}$$

Table I - Testing programme

SPECIMEN	WELD	n_1	n_2	n_3	n_4	$\Delta\sigma_r$ [MPa]	GAP [mm]
A.1.1.	1.	-	-	-	> 0.833	150	4
A.1.1.	2.	-	-	-	0.833	150	4
A.1.2.	1.	-	9.085	9.085	9.298	105	4
A.1.2.	2.	-	-	-	> 9.298	105	4
A.1.3.	1.	-	-	-	>29.582	90	4
A.1.3.	2.	-	-	-	>29.582	90	4
A.1.4.	1.	-	0.180	0.180	0.245	233	4
A.1.4.	2.	-	0.239	0.244	> 0.245	233	4
A.2.1.	1.	-	-	-	> 0.668	163	2
A.2.1.	2.	-	-	-	0.668	163	2
A.2.2.	1.	-	-	-	> 0.779	110	2
A.2.2.	2.	-	0.713	-	0.779	110	2
A.3.1.	1.	-	0.035	0.039	0.445	153	0
A.3.1.	1.	-	0.035	0.039	0.445	153	0
A.3.1.	2.	-	-	-	> 0.445	153	0
A.3.2.	1.	0.157	0.175	0.180	0.327	83	0
A.3.2.	1.	-	0.180	-	0.327	83	0
A.3.2.	1.	-	0.203	0.253	0.327	83	0
A.3.2.	2.	-	0.223	0.240	> 0.327	83	0
D.1.1.	1.	-	-	-	>10.990		
D.1.1.	2.	-	-	-	>10.990		
D.1.2.	1.	-	0.728	-	0.798		
D.1.2.	2.	-	-	-	> 0.798		

$\Delta\sigma_r$ = nominal stress range bottomside trough.

n = number of cycles x 10^6.

n_1 = initiation by strain gauge.
n_2 = first visual crack.
n_3 = length of crack equal to 50 mm.
n_4 = failure of the weld.

Table II - Results constant amplitude tests

SPECIMEN	WELD	n_1	n_2	n_3	n_4	$\Delta\sigma_e$ [MPa]	ΔS_m [MPa]	n_m
A.1.5.	1.	1.000	1.230	1.270	1.460	135	153	1.086
A.1.5.	2.	1.200	1.350	-	> 1.460	135	153	> 1.086
A.1.7.	1.	-	-	-	> 6.500	101	115	> 4.783
A.1.7.	2.	4.750	5.400	6.250	6.500	101	115	4.783
A.1.8.	1.	-	-	-	>48.100	68	77	>35.656
A.1.8.	2.	-	-	-	>48.100	68	77	>35.656
D.1.1.v.	1.	-	-	-	> 6.270	135	153	> 4.662
D.1.1.v.	2.	4.760	5.150	5.190	6.270	135	153	4.662
A.1.6.	1.	1.420	-	-	> 1.770	145	182	> 0.900
A.1.6.	2.	1.200	1.470	1.620	1.770*	145	182	0.900

$\Delta\sigma_e$ = equivalent stress range bottomside trough.

$$\Delta\sigma_e = \left(\frac{1}{\Sigma n_i} \Sigma n_i \Delta\sigma_i^3\right)^{1/3} \text{ MPa}$$

n = number of cycles x 10^6.

n_1 = initiation by strain gauge.
n_2 = first visual crack.
n_3 = length of crack equal to 50 mm.
n_4 = failure of the weld.

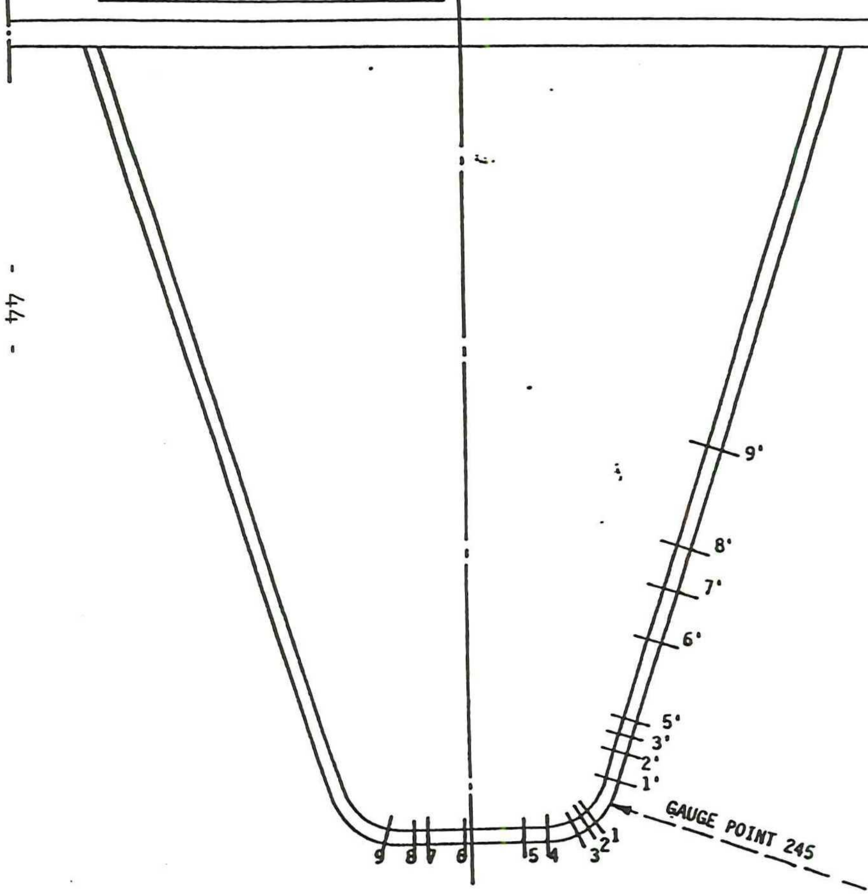
$$\Delta S_m = \frac{\Sigma n_i \Delta\sigma_i^4}{\Sigma n_i \Delta\sigma_i^3} \text{ MPa} \qquad n_m = \frac{\Sigma n_i \Delta\sigma_i^3}{\Delta\sigma_m^3}$$

in which: $\Delta\sigma_i$ = individual stress range
n_i = individual number of cycles belonging to $\Delta\sigma_i$
ΔS_m = equivalent stress range
n_m = number of cycles belonging to $\Delta\sigma_m$.

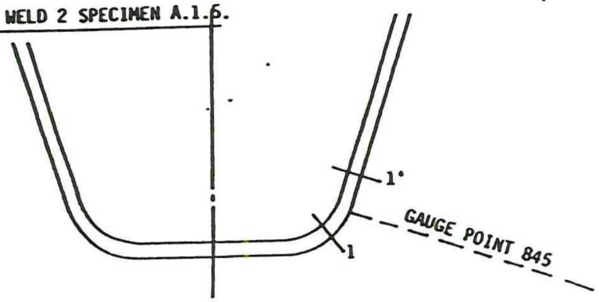
* no further crack growth, due to failure in the parent material.

Table III : Results of the variable amplitude tests

CRACK MARKS WELD 1 SPECIMEN A.1.5



CRACK MARKS WELD 2 SPECIMEN A.1.6.



CRACK GROWTH CHART SPECIMEN A.1.5
WELD I
SIMULATED-SPECTRUM: EQUIVALENT STRESS RANGE 'BOTTEMSIDE' 135 MPa

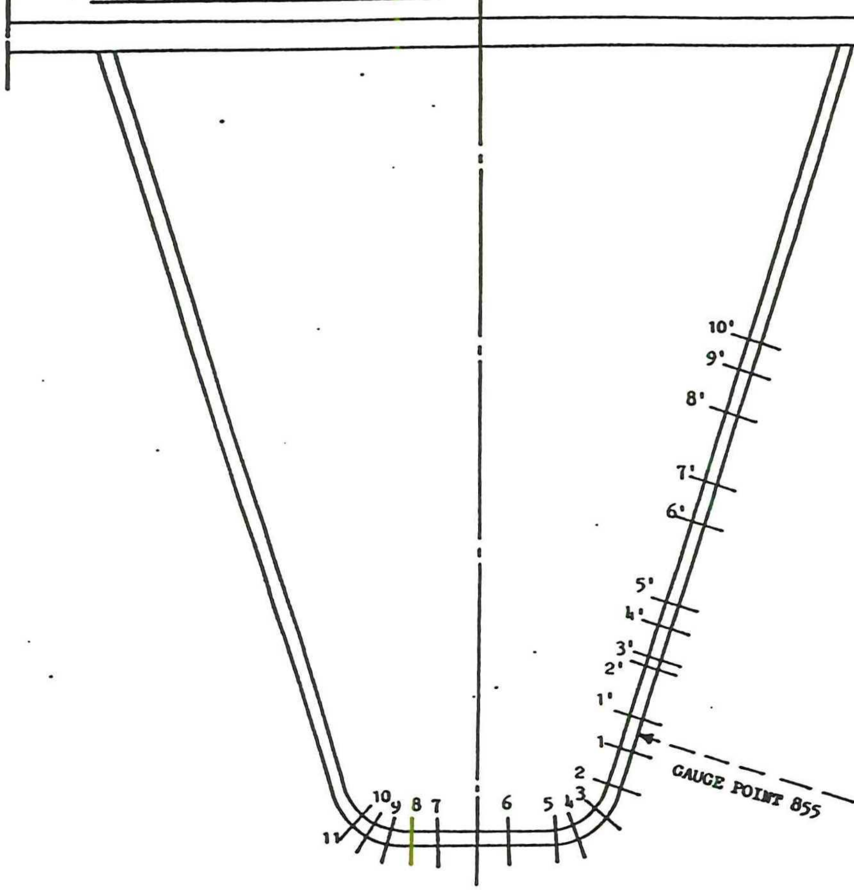
NUMBER OF CYCLES [*10⁶]	GROWTH 1st CRACK			GROWTH 2nd CRACK		
	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]
1.233	0 -1'/0-1	7/7	14			
-.---	0 -2'/0-2 1'-2'/1-2	13/22 6/15	35			
1.270	0 -3'/0-3 2'-3'/2-3	18/27 5/5	45			
-.---	0 -4'/0-4 3'-4'/3-4	33/27 15/0	60			
-.---	0 -5'/0-5 4'-5'/4-5	42/35 9/8	77			
-.---	0 -6'/0-6 5'-6'/5-6	67/68 25/33	135			
-.---	0 -7'/0-7 6'-7'/6-7	82/92 15/24	174			
-.---	0 -8'/0-8 7'-8'/7-8	88/111 6/19	199			
1.460	0 -9'/0-9 8'-9'/8-9	99/150 11/39	249			

CRACK GROWTH CHART SPECIMEN A.1.5
WELD II
SIMULTED-SPECTRUM: EQUIVALENT STRESS RANGE 'BOTTEMSIDE' 135 MPa

NUMBER OF CYCLES [*10⁶]	GROWTH 1st CRACK			GROWTH 2nd CRACK		
	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]
1.460	0 -1'/0-1	13/17	30			

WELD I FAILED AT 1.460.838 CYCLES

CRACK MARKS WELD 2 SPECIMEN A.1.6.



CRACK GROWTH CHART SPECIMEN A.1.6
WELD II
MEASURED-SPECTRUM: EQUIVALENT STRESS RANGE 'BOTTEMSIDE' 145 MPa

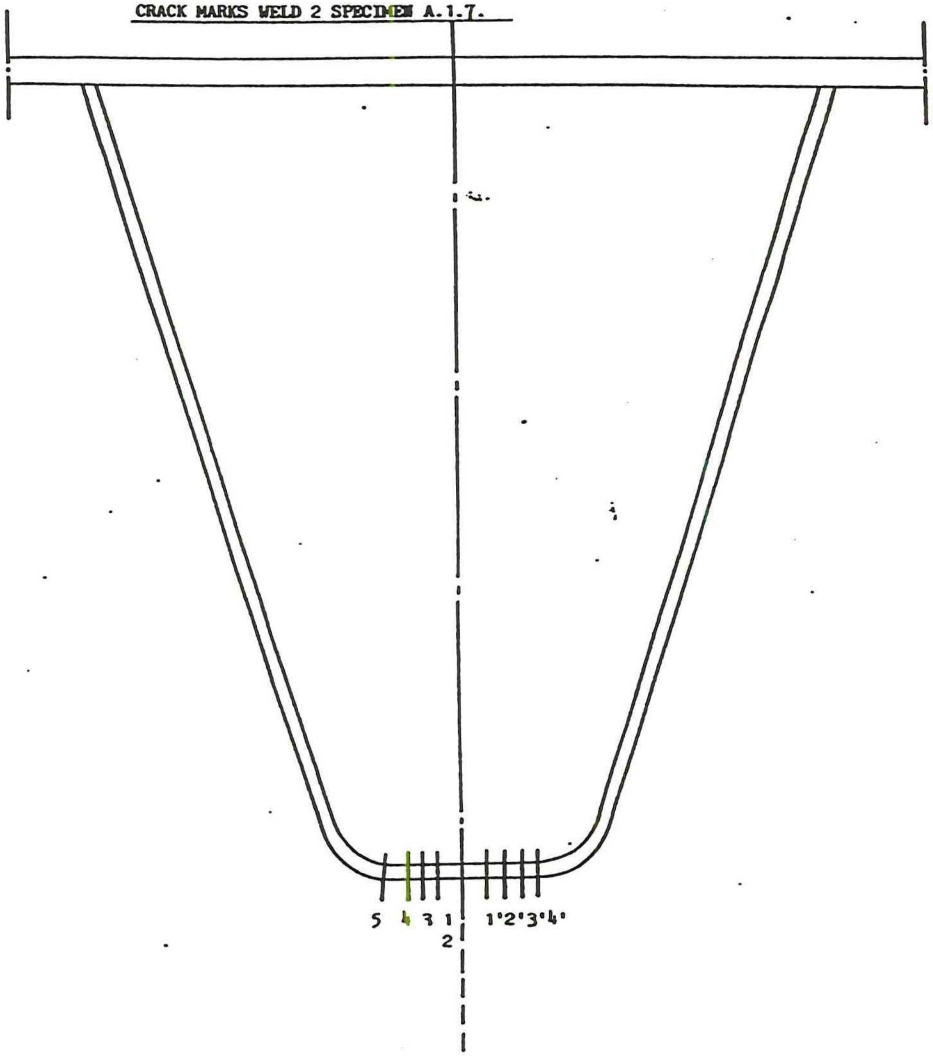
NUMBER OF CYCLES [*10⁶]	GROWTH 1st CRACK LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]	GROWTH 2nd CRACK LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]
1.467	0 -1'/0-1	7/7	14			
1.620	0 -2'/0-2 1'-2'/1-2	28/23 21/16	51			
1.663	0 -3'/0-3 2'-3'/2-3	32/36 4/13	68			
1.666	0 -4'/0-4 3'-4'/3-4	45/51 13/15	96			
1.678	0 -5'/0-5 4'-5'/4-5	55/62 10/11	117			
-.---	0 -6'/0-6 5'-6'/5-6	90/81 35/19	171			
1.746	0 -7'/0-7 6'-7'/6-7	108/112 18/31	220			
1.755	0 -8'/0-8 7'-8'/7-8	137/122 29/10	259			
1.763	0 -9'/0-9 8'-9'/8-9	155/132 18/10	187			
-.---	0 -10'/0-10 9'-10'/9-1	168/141 13/9	309			

CRACK GROWTH CHART SPECIMEN A.1.6
WELD I
MEASURED-SPECTRUM: EQUIVALENT STRESS RANGE 'BOTTEMSIDE' 145 MPa

NUMBER OF CYCLES [*10⁶]	GROWTH 1st CRACK LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]	GROWTH 2nd CRACK LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]
1.763	0 - 1'/0 -1	5/5	10			

WELD II FAILED AT 1.766.000 CYCLES

CRACK MARKS WELD 2 SPECIMEN A.1.7.



CRACK GROWTH CHART SPECIMEN A.1.7
WELD I
SIMULATED-SPECTRUM: EQUIVALENT STRESS RANGE 'BOTTEMSIDE' 101 MPa

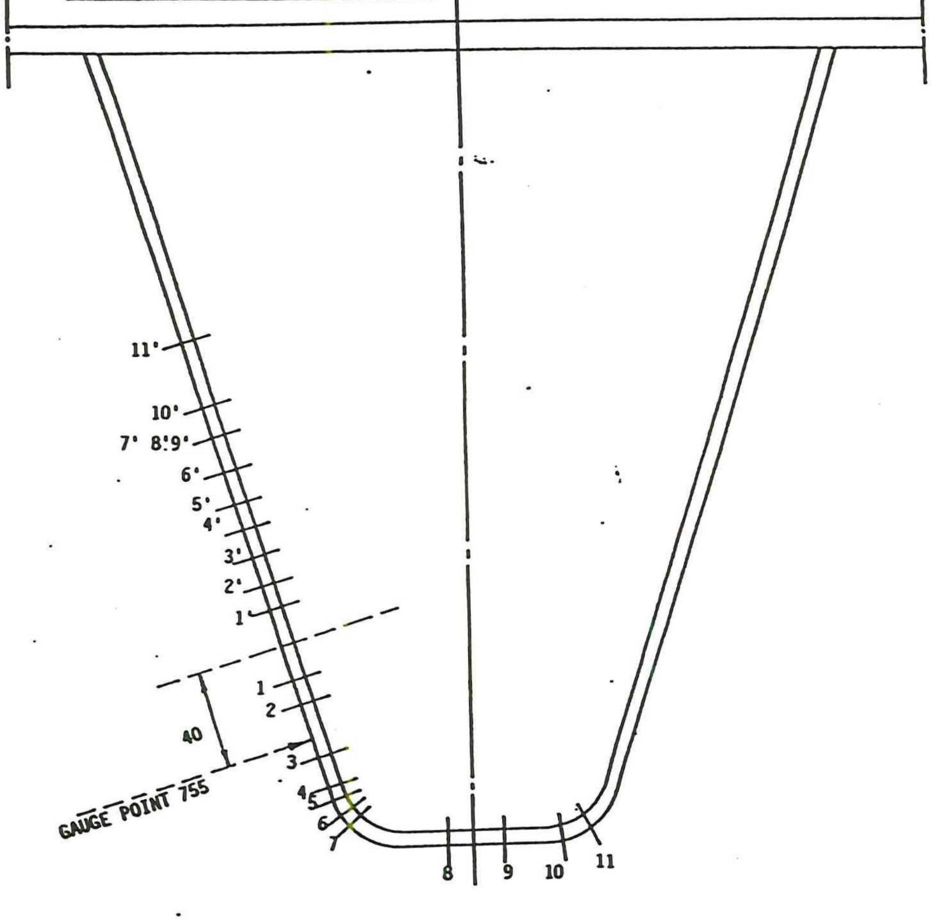
NUMBER OF CYCLES [*10⁶]	GROWTH 1st CRACK			GROWTH 2nd CRACK		
	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]

CRACK GROWTH CHART SPECIMEN A.1.7
WELD II
SIMULATED-SPECTRUM: EQUIVALENT STRESS RANGE 'BOTTEMSIDE' 101 MPa

NUMBER OF CYCLES [*10⁶]	GROWTH 1st CRACK			GROWTH 2nd CRACK		
	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]
5.385	0 -1'/0-1	10/10	20			
5.582	0 -2'/0-2 1'-2'/1-2	17/10 7/6	27			
6.063	0 -3'/0-3 2'-3'/2-3	24/16 7/6	40			
6.250	0 -4'/0-4 3'-4'/3-4	30/22 6/6	52			

WELD II FAILED AT 6.452.000 CYCLES

CRACK MARKS WELD 2 SPECIMEN D.1.1.v.



CRACK GROWTH CHART SPECIMEN D.1.1.v
WELD I

SIMULATED-SPECTRUM: EQUIVALENT STRESS RANGE 'BOTTEMSIDE' 135 MPa

NUMBER OF CYCLES [*10⁶]	GROWTH 1st CRACK			GROWTH 2nd CRACK		
	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]

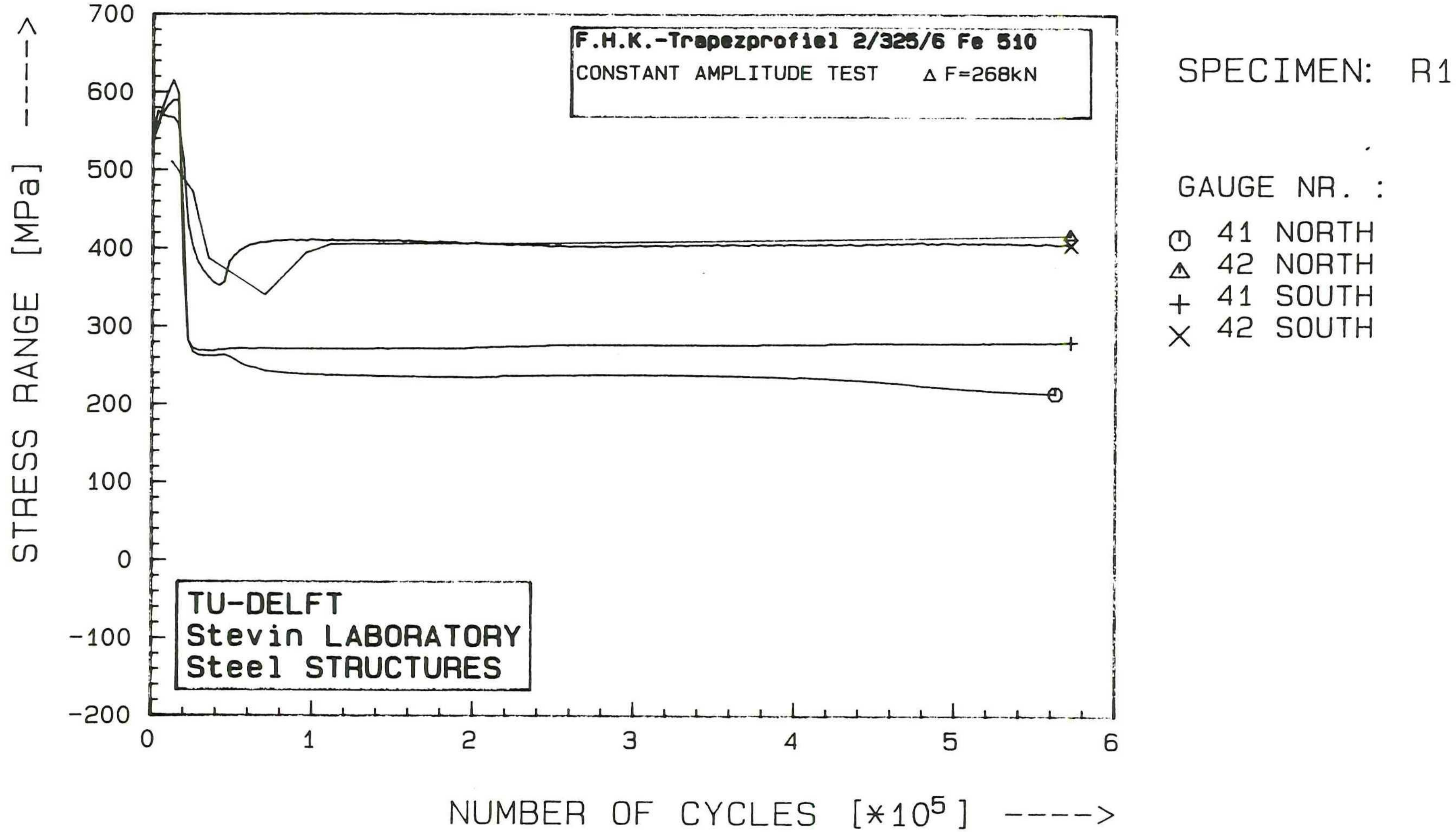
CRACK GROWTH CHART SPECIMEN D.1.1.v
WELD II

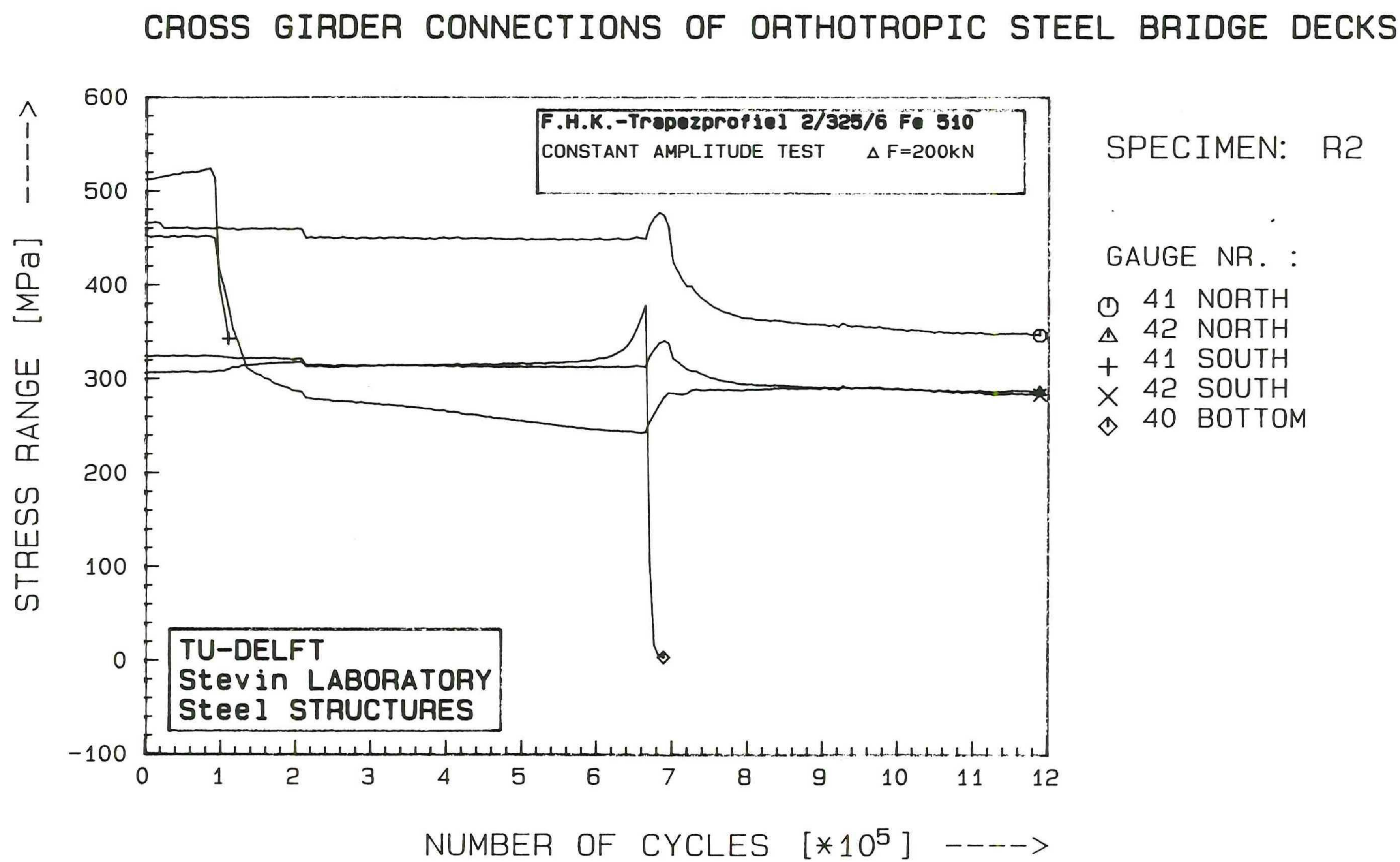
SIMULATED-SPECTRUM: EQUIVALENT STRESS RANGE 'BOTTEMSIDE' 135 MPa

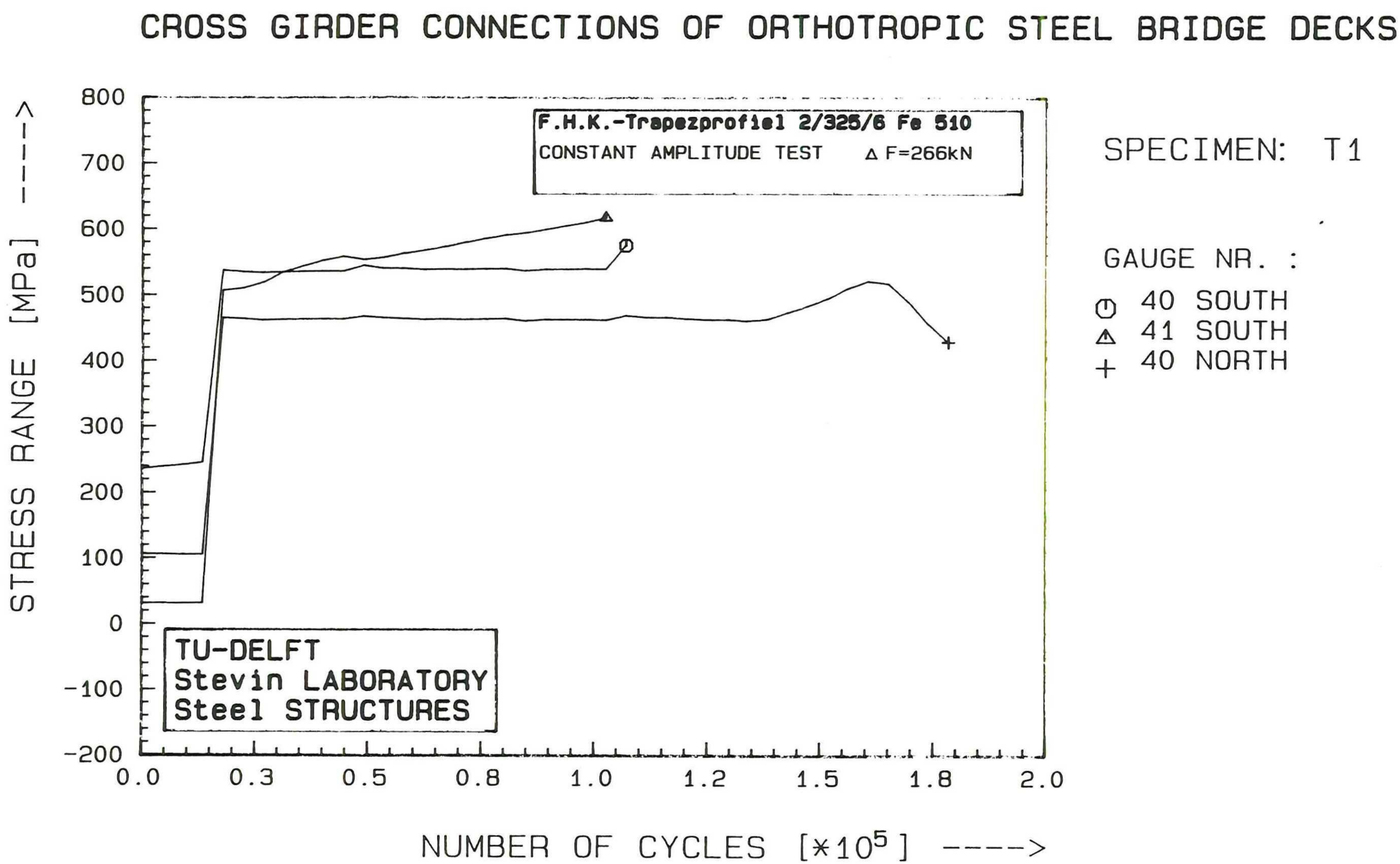
NUMBER OF CYCLES [*10⁶]	GROWTH 1st CRACK			GROWTH 2nd CRACK		
	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]	LOCATION	LENGTH [mm]	TOTAL LENGTH [mm]
5.130	0 -1'/0-1	15/15	40			
5.190	0 -2'/0-2 1'-2'/1-2	26/25 11/10	50			
5.620	0 -3'/0-3 2'-3'/2-3	38/48 12/23	60			
-.---	0 -4'/0-4 3'-4'/3-4	50/61 12/13	70			
5.960	0 -5'/0-5 4'-5'/4-5	61/67 11/6	80			
6.090	0 -6'/0-6 5'-6'/5-6	75/75 14/8	95			
-.---	0 -7'/0-7 6'-7'/6-7	90/79 15/4	115			
6.200	0 -8'/0-8 7'-8'/7-8	90/122 0/43	125			
6.220	0 -9'/0-9 8'-9'/8-9	90/146 0/24	236			
6.230	0 -10'/0-10 9'-10'/9-10	103/171 13/25	274			
-.---	0 -11'/0-10 10'-11'/10-11	130/184 27/13	314			

WELD II FAILED AT 6.274.000 CYCLES

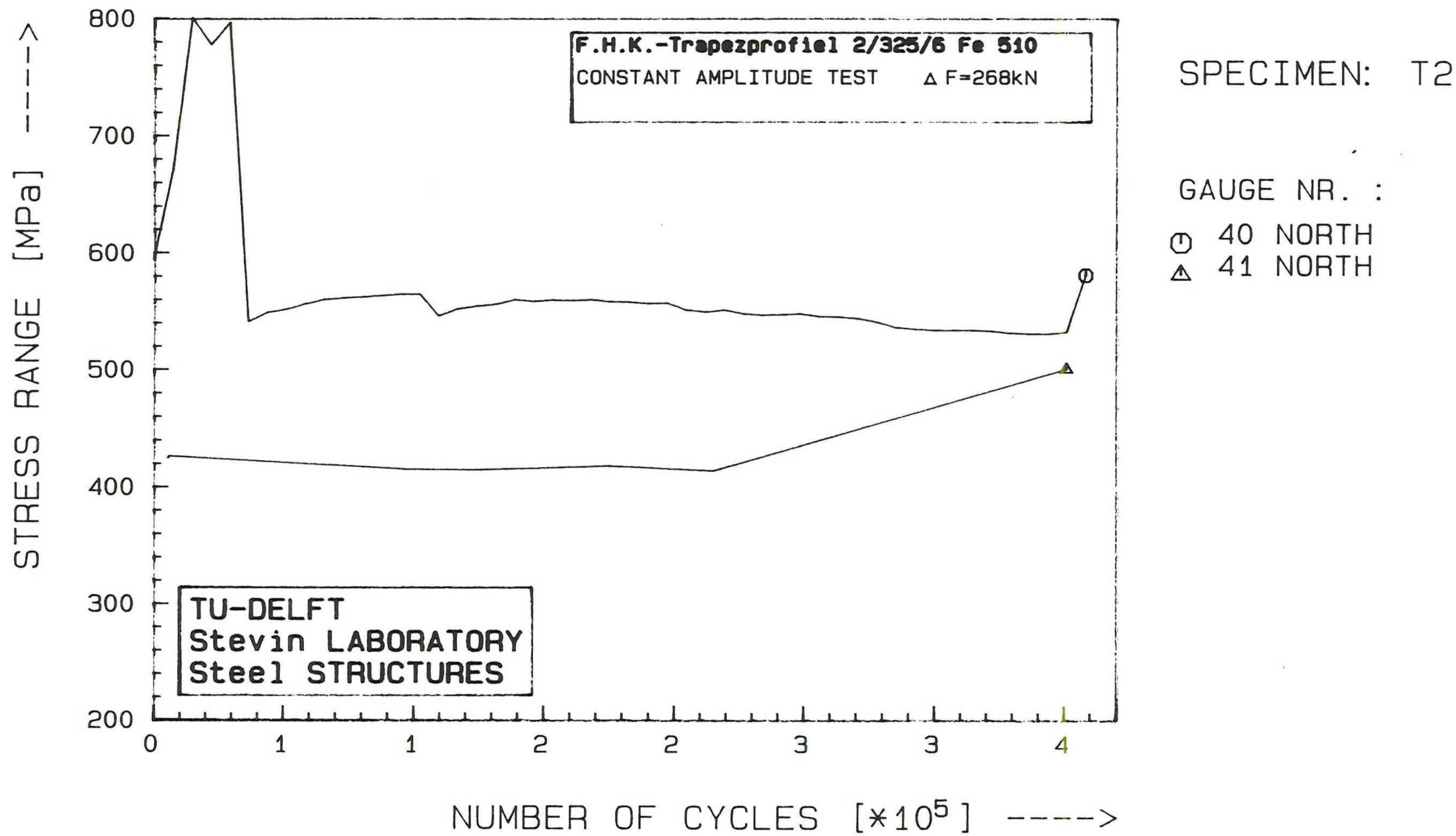
CROSS GIRDER CONNECTIONS OF ORTHOTROPIC STEEL BRIDGE DECKS




CROSS GIRDER CONNECTIONS OF ORTHOTROPIC STEEL BRIDGE DECKS
F.H.K.-Trapezprofiel 2/325/6 Fe 510
CONSTANT AMPLITUDE TEST   Δ F=200kN
SPECIMEN: R2
GAUGE NR. :
41 NORTH
42 NORTH
41 SOUTH
42 SOUTH
40 BOTTOM
TU-DELFT
Stevin LABORATORY
Steel STRUCTURES
STRESS RANGE [MPa] ---->
600
500
400
300
200
100
0
-100
0
1
2
3
4
5
6
7
8
9
10
11
12
NUMBER OF CYCLES [*10^5] ---->


CROSS GIRDER CONNECTIONS OF ORTHOTROPIC STEEL BRIDGE DECKS
F.H.K.-Trapezprofiel 2/325/6 Fe 510
CONSTANT AMPLITUDE TEST Δ F=266kN
SPECIMEN: T1
GAUGE NR. :
40 SOUTH
41 SOUTH
40 NORTH
TU-DELFT
Stevin LABORATORY
Steel STRUCTURES
STRESS RANGE [MPa] ---->
800
700
600
500
400
300
200
100
0
-100
-200
0.0
0.3
0.5
0.8
1.0
1.2
1.5
1.8
2.0
NUMBER OF CYCLES [*10^5] ---->

CROSS GIRDER CONNECTIONS OF ORTHOTROPIC STEEL BRIDGE DECKS



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