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TESTS AND CALCULATIONS OF REINFORCED CONCRETE BEAMS SUBJECT TO DYNAMIC REVERSED LOADS

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SUMMARY

This study presents the tests of a reinforced concrete beam conducted by the Department of Mechanical and Thermal Studies at the Centre d'Etudes Nucléaires, Saclay, FRANCE. The actual behavior of nuclear power plant buildings submitted to seismic loads is generally non linear even for moderate seismic levels. The non linearity is specially important for reinforced concrete beams type buildings.

To estimate the safety factors when the building is designed by standard methods, accurate non linear calculations are necessary. For such calculations one of the most difficult point is to define a correct model for the behavior of a reinforced beam subject to reversed loads . For that purpose, static and dynamic experimental tests on a shaking table have been carried out and a model reasonably accurate has been established and checked on the tests results.

1.INTRODUCTION

The study presented here is the first test program intended to study the behavior of a reinforced concrete beam under seismic condition using the shaking table VESUVE at the Saclay Nuclear Center. The actual behavior of nuclear power plant buildings submitted to seismic loads is generally non linear even for moderate seismic levels. To estimate the safety factors when the building is designed by standard methods, accurate non linear calculations are necessary. For such calculations one of the most difficult point is to define a correct model for the behavior of a reinforced beam subject to reversed load. The main objectives of this program of static and dynamic tests were to obtain information of such structures vibrating and to develop new calculation models for the prediction of the structure's inelastic response to an earthquake excitation.

2.THE TEST PROGRAM

2.1. Selection of the test structure.

The selection of the structure to be tested was guided by some preminimary considerations. First it was wished to carry on a fundamental study on dynamic behavior with simple flexion without shearing. As a second point the fundamental frequency of the structure was wanted to be situated in the region of the earthquake's response spectrum for which displacement and velocity are relatively important. There reasons led to the consideration of a reinforced concrete column fixed on its basis and with a mass of two tons fixed on top.

The structure for the test was obtained by reducing the size of a typical column of an actual nuclear power plant building by a factor 3. The height is 1.44 m and the section is a square 17 cm x 17 cm. There are 12 reinforcing steel 8 mm diameter and the stirrups are spaced each 5 cm. The design of the structure was made according to the requirements of the 1968 Uniform building Code available in France (CC BA 68 Rules).

2.2. Material properties.

The mechanical characteristics of the reinforcement and the concrete, used in the computation of the response of the test structure, were determined from their measured stress-strain relationships.

The average values obtained from the different tests are given below :

	Young's Modulus	Yielding limit Sy	Ultimate limit.Su
Steel	20 000 kgF/$_{mm}$2	42 kgf/$_{mm}$2	56 kgf/$_{mm}$2
Concrete	3 200 kgF/$_{mm}$2	1 kgf/$_{mm}$2	4.25 kgf/$_{mm}$2

2.3. Static Tests.

Tests of loading and unloading were conducted on four specimens with record of the force - top displacement relationship on a plotting table. The force was applied by an horizontal hydraulic actuator at 1.m above the basis. Two kinds of tests were carried out : Monotonic loadings giving the Moment-curvature relationship and alternate loading giving a behavior's law.

2.4. Dynamic Tests.

In order to see the influence of the earthquake's characteristics it was choosen two reference earthquakes. The TAFT NS component and the SAN FRANCISCO NS component which has a shorter duration and a central frequency relatively higher than the former. The response spectra of these two earthquakesfor a damping factor of 5 % are shown on figure 1.

To obtain a better representation of an actual building's behavior, subject to seismic load, which has a fundamental frequency three or four times lower that the column test the accelerograms were contracted in time and dilated in acceleration as to conserve the velocities (factor 4 for TAFT and 3 for SAN FRANCISCO)

Eight series of test were conducted on the shaking table with 8 specimens.

Acceleration on top and basis were recorded as relative-displacement of several points of the column as shown on the figure 2. The particularities of each series of test are presented in the table shown figure 3. Succession of earthquakesof increasing level were conduct as succession of earthquakesdirectly at the level gives the collapse in the precedent test.

3. GENERAL TEST RESULTS

3.1. Static Tests.

During the loading test a great number of unloading have been carried out from a load obviously lower than the ultimate load. All these tests have given the same result : the appearance of cracks lead to a stiffness degradation even for a low level of load. And, below the level of load involving the plastic deformation of the steel, a loading, after a stiffness degradation and an unloading, is done like an elastic one with a new stiffness lower than the original one.

3.2. Dynamic tests.

The dynamic tests confirm the results predicted by the static ones : the fundamental frequency of the structure decrease as the level of the load increase. This is clearly apparent on the time-histories top displacement plotted on the figure 5 at low level and high level for Taft and San Francisco. The existence of cracks even at a low level of load is put in evidence by a strain-gage stuck on a steel reinforcement at the basis of the column whose time-history is plotted on figure 6: when the concrete is in traction, only steel works, but when the concrete is in compression steel and concrete work and the absolute value of the strain is obviously lower.

For each earthquake, the fundamental frequency and the damping factor of the structure have been determined by considering its free movement after the end of the earthquake as the movement of a pseudo-periodic oscillator. The results shown on figure 7 shows the decrease of the frequency as a function of the maximum relative deflexion of the beam in the course of the movement. It can also be seen the little increase of the damping as a function of the maximum relative deflexion in the course of the time. On this figure had been marked the Normal limit and Extreme limit corresponding to French regulation for reinforced concrete building design (CC BA 68 Rules). For this beam the Normal limit is the load which leads to the maximum value of 0,66 Sy on wires or 0,6 Su on concrete in compression. The Extreme limit is the load which leads to the maximal value of Sy on wires or 0,9 Su on concrete in compression.

The points of the curves giving frequency and damping before any deflexion were determined by excitation at a very low level with an electromagnetic vibration generator : this was the only mean to insure that no cracks would appear. The fundamental frequency thus obtained agrees very well with this calculated taking in account the actual material and geometrical properties of the beam.

The set of the tests does not show a significant difference of effect between a beam directly subject to high level earthquake and a beam subject to high level earthquake after a series of increasing level earthquake from low to high.

With regard to this fact a test of fatigue had been conducted with one beam which had been submitted 40 times to the same earthquake (TAFT Vmax = 0.13 m/s). The results plotted on figure 8 show the fundamental frequency decreases from 5 Hz to 2.5 Hz during the first earthquake. That is the stiffness is not subjected to a new significant degradation when the level of the following earthquakes does not increase. Also it can be seen that the damping decreases a little after the first earthquake to attain a constant value at the end of about twenty cycles.

As the level of the earthquake was increased it was observed the appearance of cracks at each stirrup level from the basis to the top. The main crack was situated at the fixed end of the beam.

The observation of the deformed shape of the beam shows a rotation at the fixed end of the beam and a quasi-linear shape of the beam in the course of the movement. Then the relative deflexion that is the ratio of top deflection on beam height seemed to be a good parameter of the total deformation.

To indicate the level of each earthquake the maximal velocity of the table during the movement had been chosen. The collapse appeared for TAFT at 0.50 m/s and for SAN FRANCISCO at 0.55 m/s, the difference is not very significant. The results of relative deflexion of two beams for the two earthquakes are represented on the figure 9. The relative deflexion of the beam is less important with TAFT than with SAN FRANCISCO for low and middle level but for high level the collapse appears approximatively for the same maximum velocity.

4. INTERPRETATION

4.1. Dynamical characteristics.

It had been observed that the frequency of the beam after an earthquake only depends on the maximum relative deflexion reached during the movement. The figure 7 shows the experimental points for several beams are very close to a unique curve giving frequency as a function of maximum displacement. No influence of the duration of the earthquake (3.5 second for contracted TAFT and 1.9 second for contracted SAN FRANCISCO) had been observed on the frequency-deflexion relationship.

The results for the damping factor are more scattered but there is a little increase of the mean value with the maximum relative deflection. But it can be observed the measured values are significantly lower than the usual values taking in account for seismic building design. An explanation of this is perhaps that the damping was determined with the free movement of the beam when the relative deformation is lower than during the earthquake. It must also be remarked that the concrete was made in laboratory and with a granulometry at scale one third of the usual one in order to respect the similarity conditions.

4.2. Comparison between test and usual calculation methods.

The usual calculation method is to calculate the frequency of the structure and to determine with the response spectrum of the earthquake, for the damping of the structure, the maximum acceleration to which each mass is submitted. This gives the load and the moment in the section of the beam. With this moment the crack section concrete in traction is determined, then the actual neutral axis and finally the maximum compression stress on concrete and the maximum traction stress on steel wires. Those values are compared with the normal and the extreme limit. That is that we call "elastic analysis". On figures 11 and 12 it can be seen that the safety factor between normal limit and collapse is about 3 to 5. The safety factor between extreme limit and collapse is about 2.5 to 4. But on an other point the calculated displacements are lower than the actual ones and that set a problem for buildings which are very close one from the other : shocks may appear that have not been predicted by calculations. Consequently the floor response spectra given by this method may be very different of the actual ones. The error can be great when are computed the seismic loads on the equipments resting on the structure.

Seeing the results of the tests which have been conducted a simple calculation method can be suggested. For computing the stiffness of the beam it is supposed that only half of the concrete section works so as the steel wires (because of the cracks which would rapidly appear in case of earthquake). The frequency of the beam is then lower and because of the shape of the response spectrum the displacements are greater. Such an analysis called "half cracked section analysis" had been carried out for the beams and the normal and extreme limits are plotted on figures 11 and 12. The safety factor is approximatively the same than in elastic analysis but the displacements are rather more important than the actual ones. These considerations are available for this type of test and for this range of frequencies.

5. CONCLUSION.

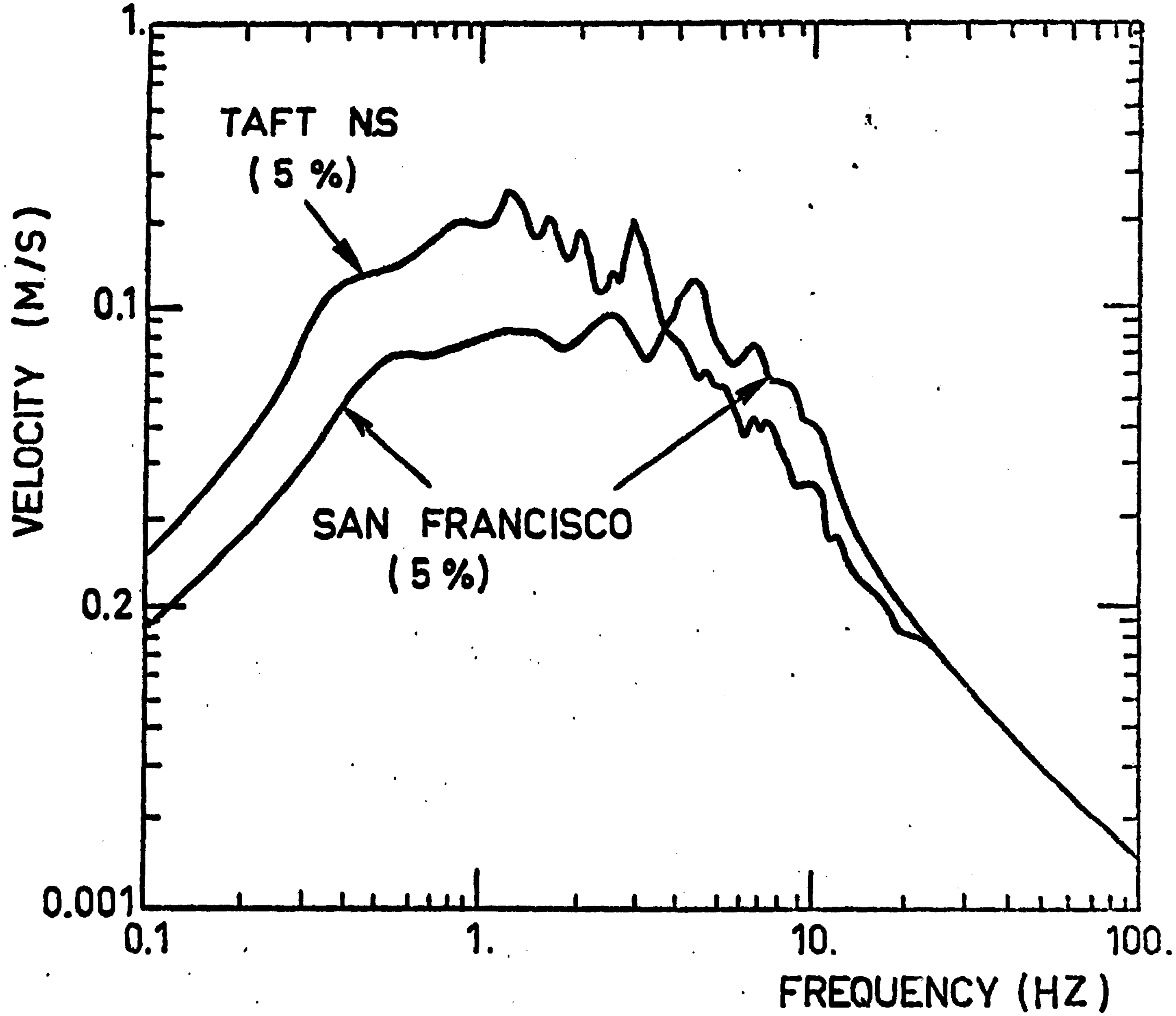
The series of test of a reinforced concrete beam had been carried out. Tests are carried on to study the biaxial effect with a beam submitted to earthquake in two perpendicular directions.

The further developments concern porticos to study the behavior of a complete structure similar to a building. Then walls will be tested in sollicitation in their plan to study the shearing.

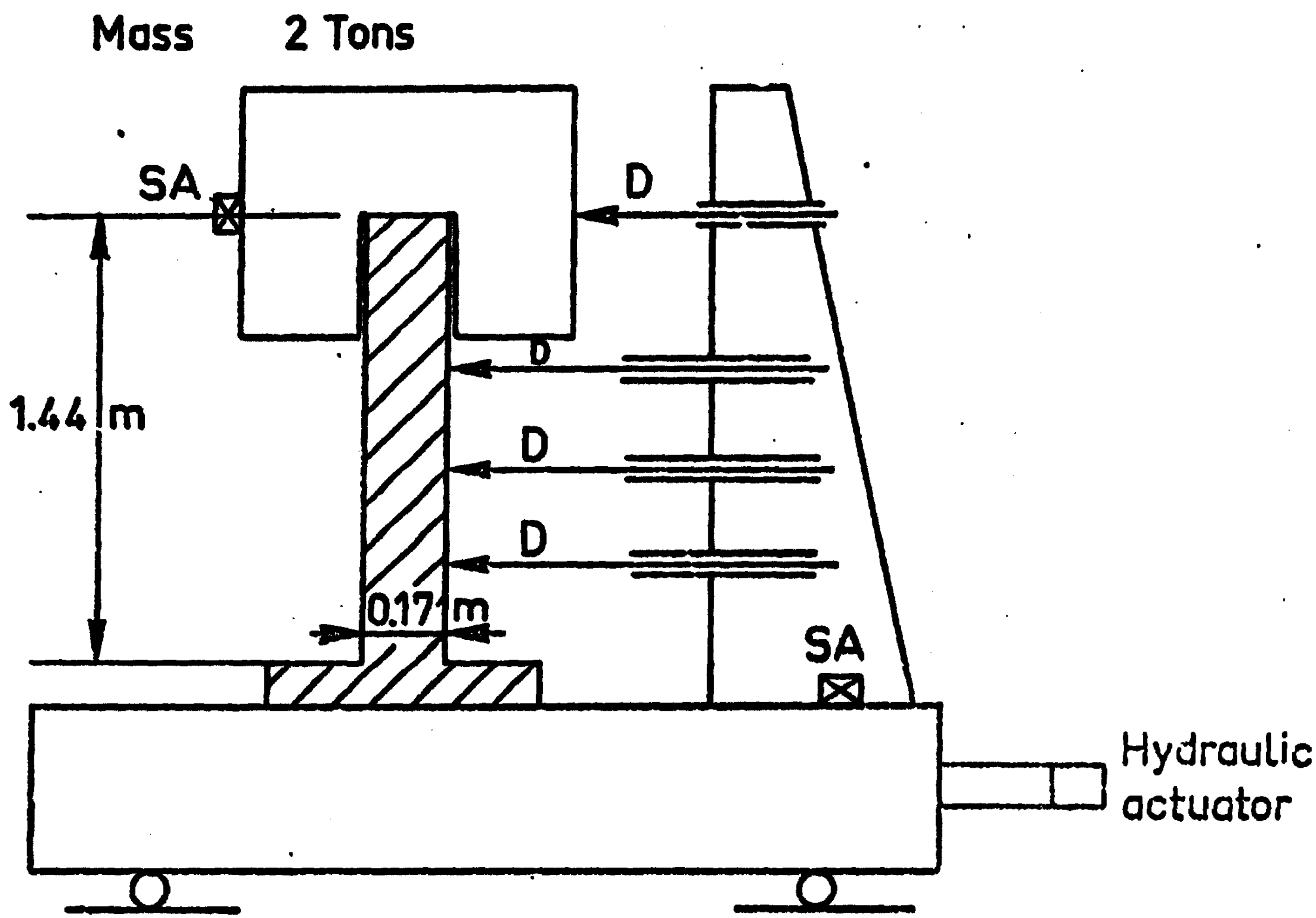
The main result of the test conducted is the stiffness degradation of a reinforced concrete structure even when the earthquake level is relatively low. That involves the fundamental frequency of the structure decreases when the maximum deflexion increases as far as the section is completely cracked. It is necessary to take a good prediction of the displacements of the structure submitted to seismic load. That leads to develop non linear computation models which may be of two kinds. A first model is to build an iterative process between stiffness, dynamic load and non-cracked section by using the classical techniques of response spectrum. A more accurate model is being developed taking in account the actual behavior of the reinforced concrete. The cracking involves a stiffness degradation and plasticity of concrete and steel is also taken in account. This model involves a direct dynamic calculation using an accelerogram. Such a model was introduced in the TEDEL program of the CEASEMT system (presented at the same conference) and the first results seem to be excellent.

FIGURES CAPTION

1. Pseudo-velocity response spectra for TAFT NS and SAN FRANCISCO NS with 5 % damping.
2. Dynamic test installation diagram.
3. Table of the dynamic tests.
4. Force - Top displacement diagram.
5. Time-history of top displacement at low and high level with TAFT US and SAN FRANCISCO.
6. Time-history of a strain-gage at a low level.
7. Frequency and damping as a function of the maximal relative deflection.
8. Influence of fatigue on frequency and damping.
9. Maximal relative deflexion as a function of the maximal earthquake velocity.
10. General view of the installation.
11. Relative deflexion with TAFT NS
12. Relative deflexion with SAN FRANCISCO.



TAFT NS
(5 %)
SAN FRANCISCO
(5 %)
VELOCITY (M/S)
1.
0.1
0.2
0.001
0.1
1.
10.
100.
FREQUENCY (HZ)



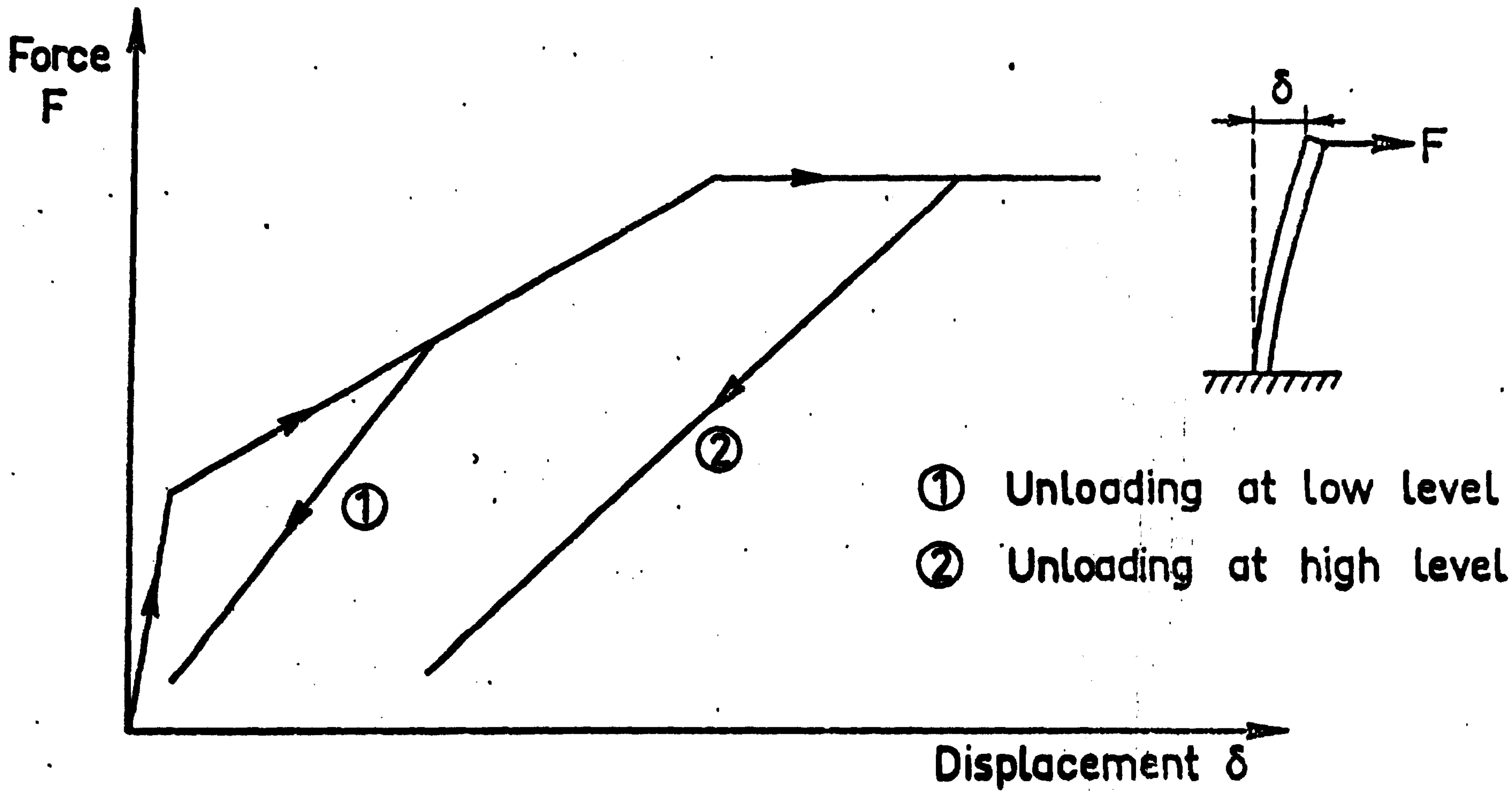
Shaking table

SA Servo - accelerometer

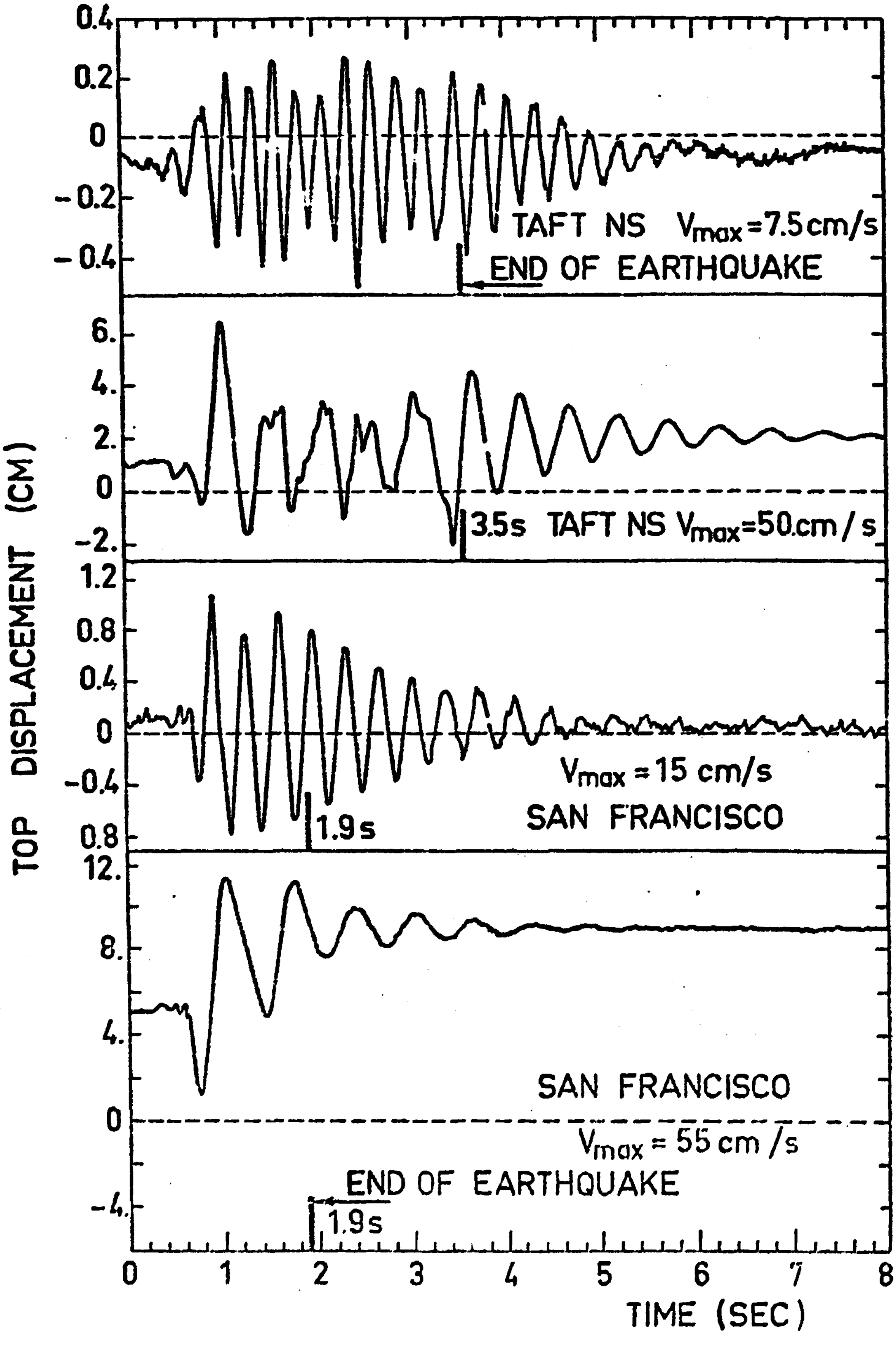
D Relative displacement transducer

Earthqvake	Beam Ref.	Number of Earthqvake	Max. speed of table	Observations
TAFT	JT 9.	8	0.075 m/s ↗ 0.50m/s	increasing levels
	JT 10	4	0.50 m/s	high level
	JT 14	40	0.13 m/s	effect of fatigue
	JT 15	8	0.075m/s ↗ 0.50m/s	increasing level
	JT 16	10	0.075m/s ↗ 0.50m/s	biaxial effect
SAN FRANCISCO	JT 11	13	0.06m/s ↗ 0.55m/s	increasing level
	JT 12	8	0.55 m/s	high level
	JT 13	13	0.06m/s ↗ 0.55m/s	increasing level

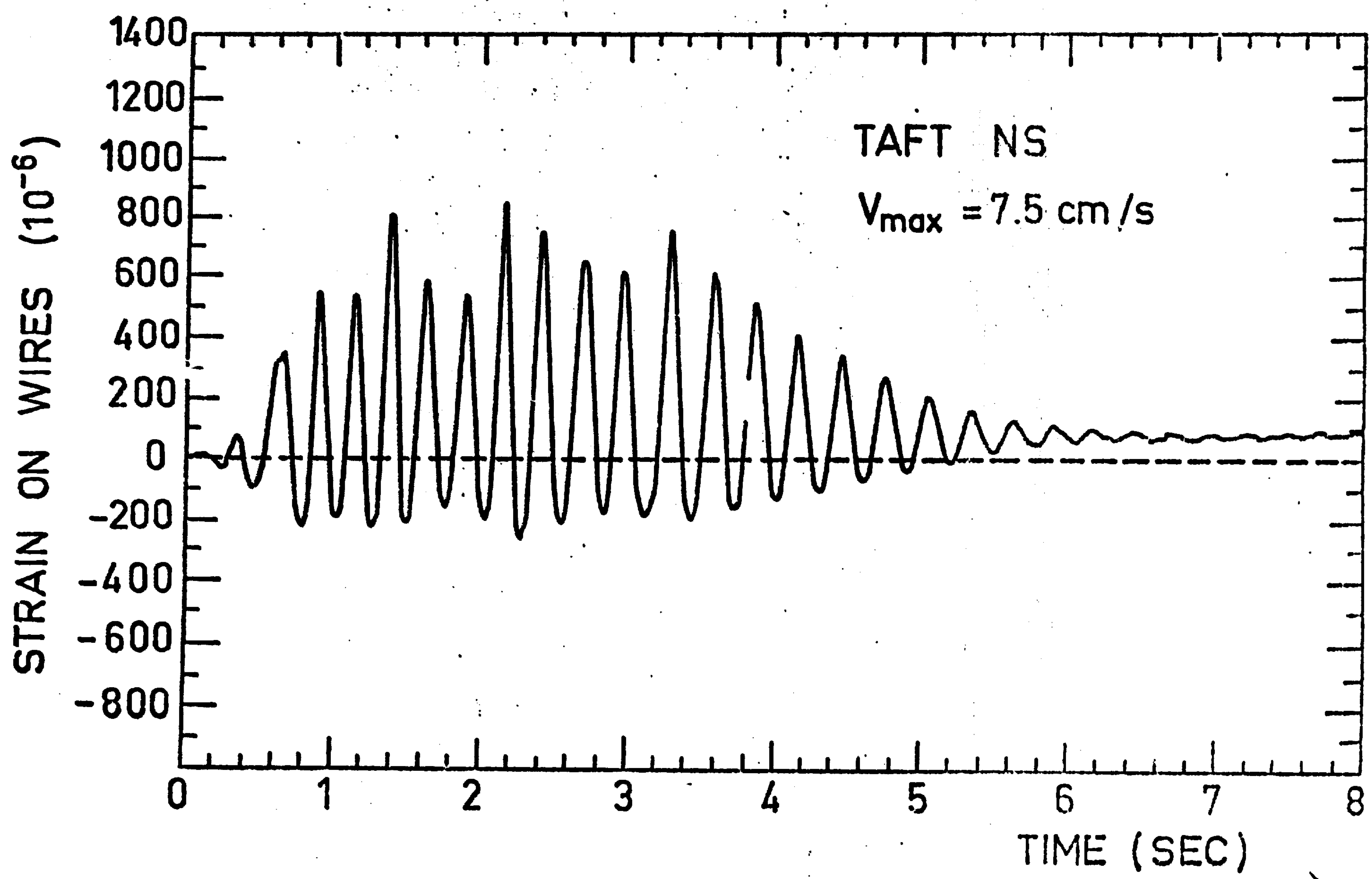
Table of tests

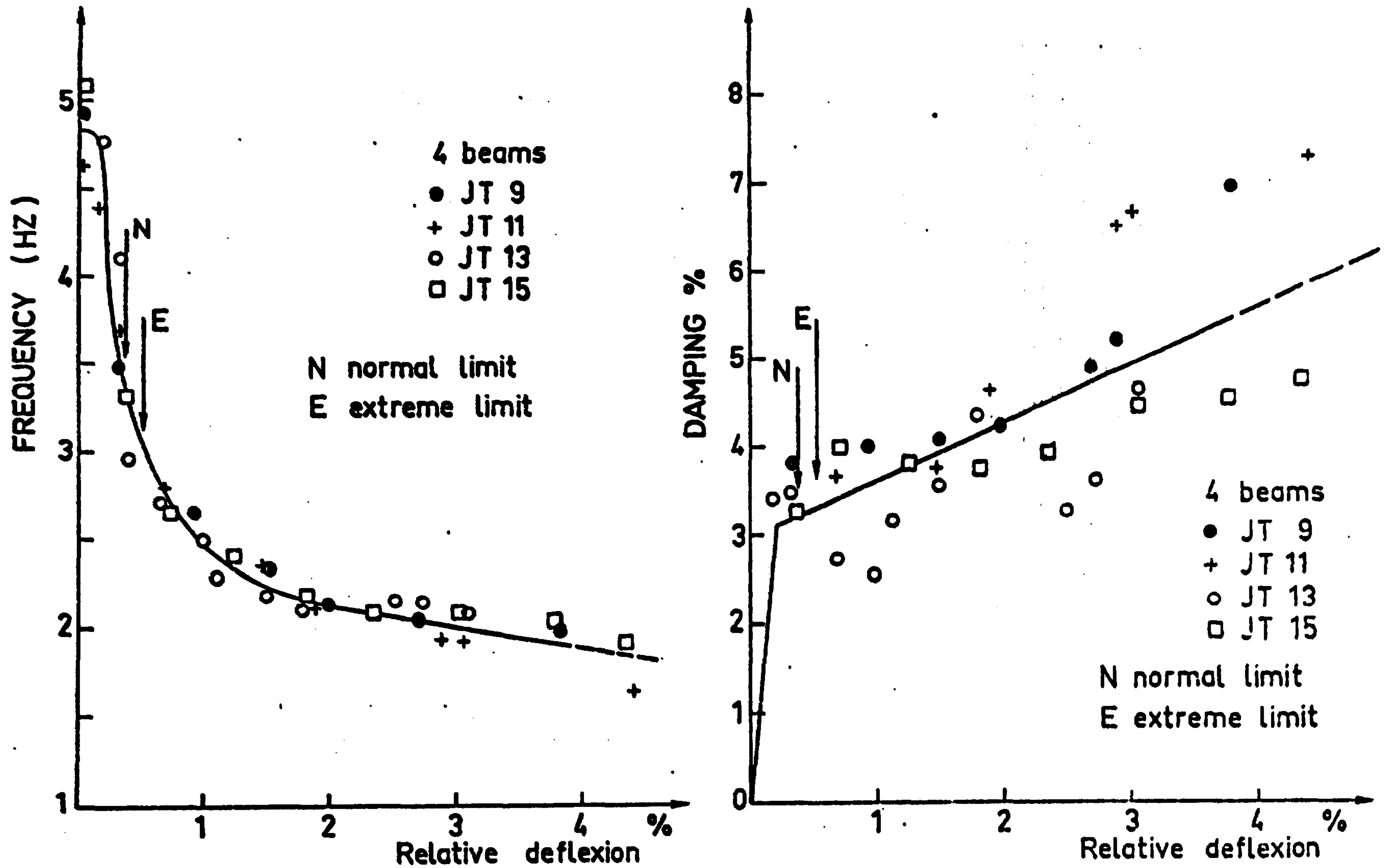


Tests of loading – unloading

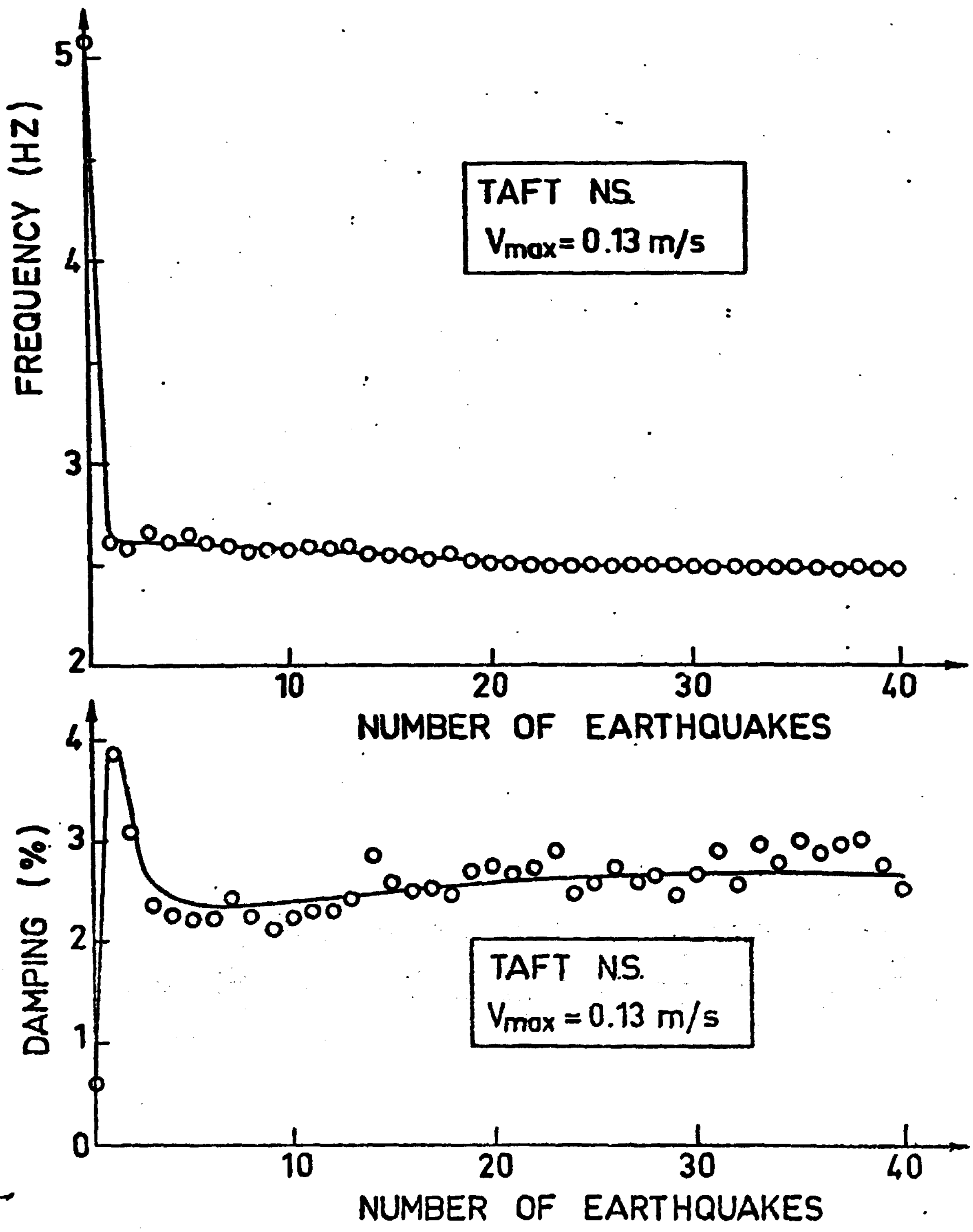


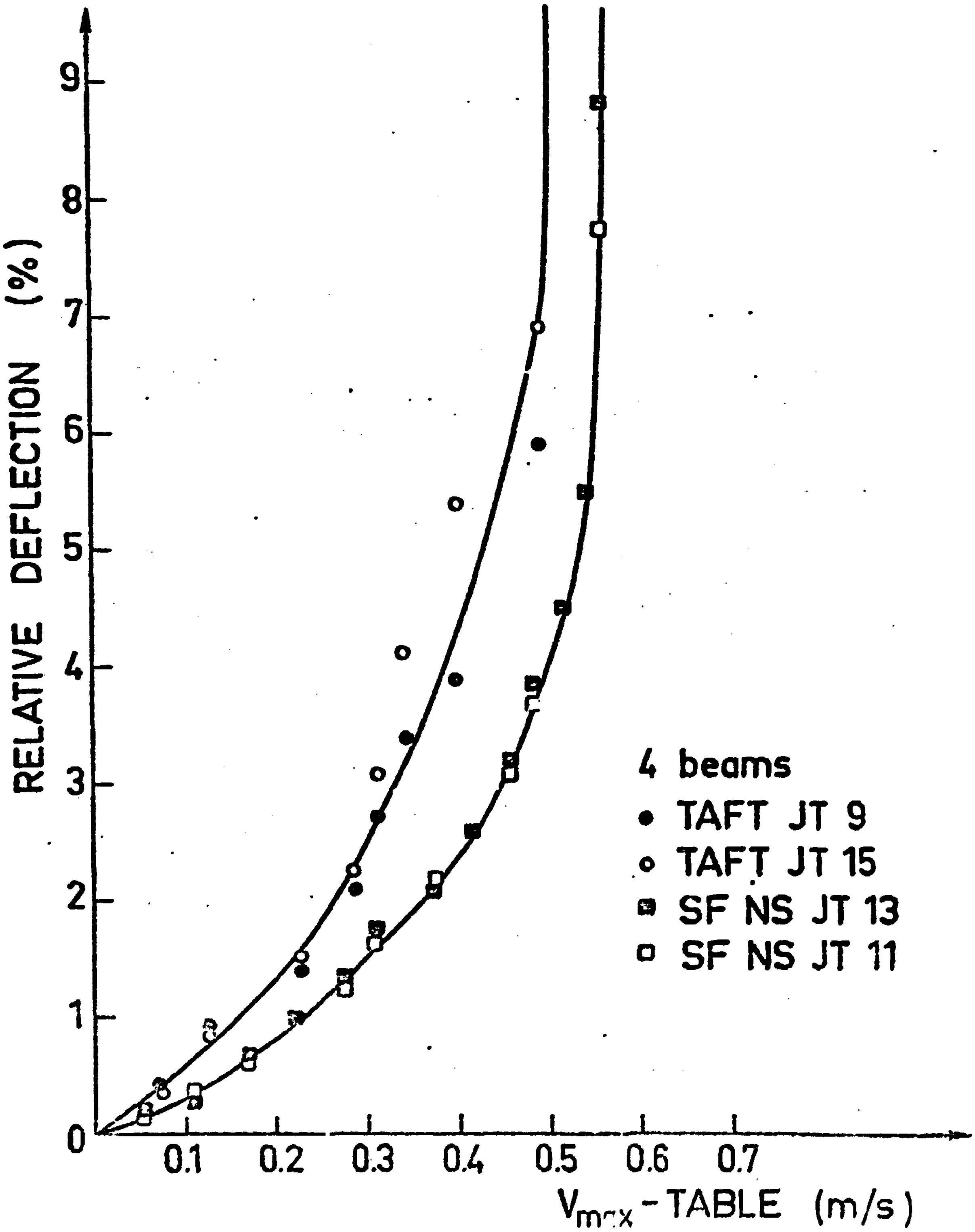
TAFT NS V_{max}=7.5 cm/s
END OF EARTHQUAKE
3.5s TAFT NS V_{max}=50.cm/s
V_{max} = 15 cm/s
SAN FRANCISCO
1.9s
SAN FRANCISCO
V_{max} = 55 cm/s
END OF EARTHQUAKE
1.9s
TOP DISPLACEMENT (CM)
TIME (SEC)


TAFT NS
V_{max} = 7.5 cm/s
STRAIN ON WIRES (10^{-6})
1400
1200
1000
800
600
400
200
0
-200
-400
-600
-800
0
1
2
3
4
5
6
7
8
TIME (SEC)

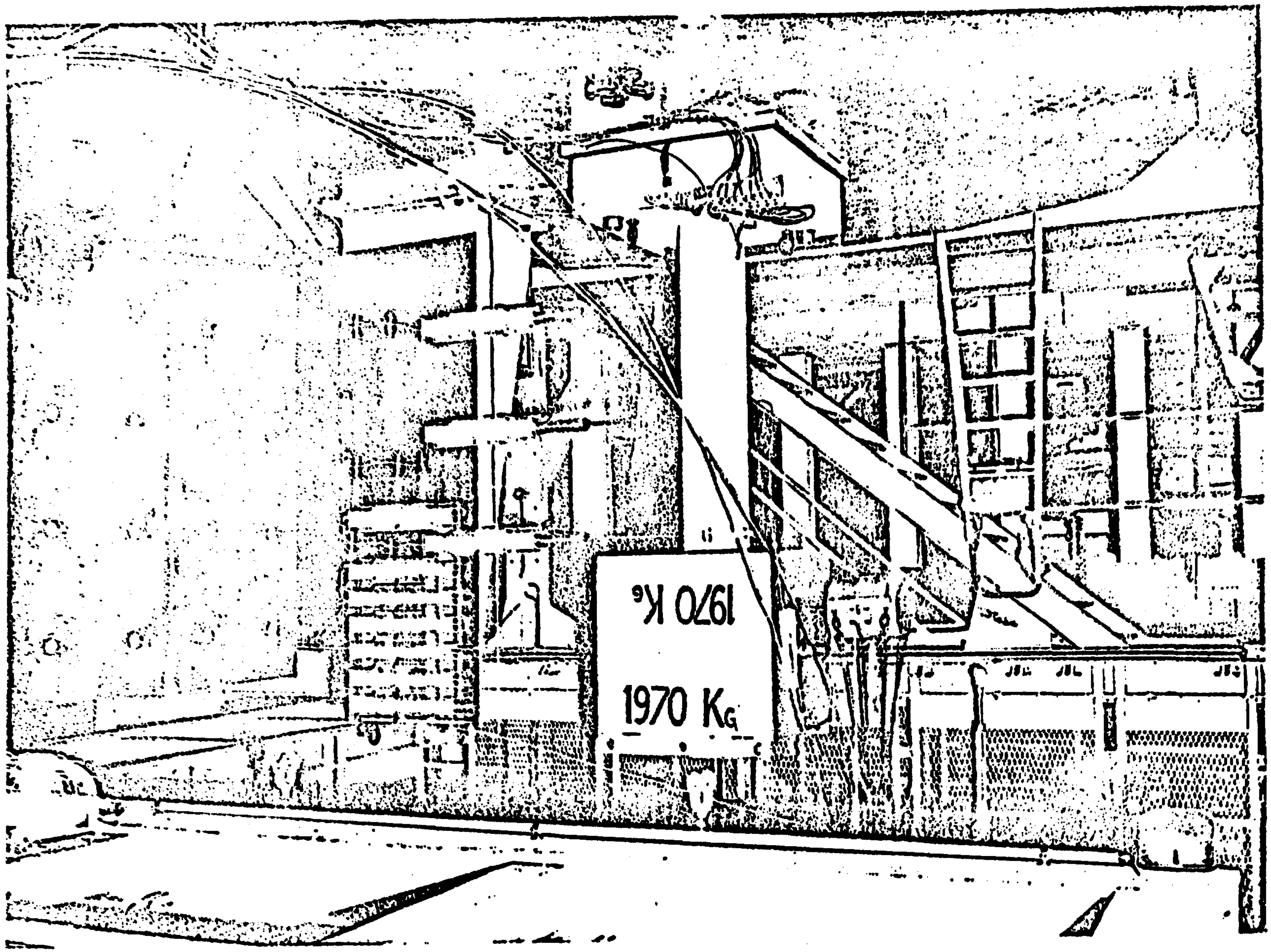


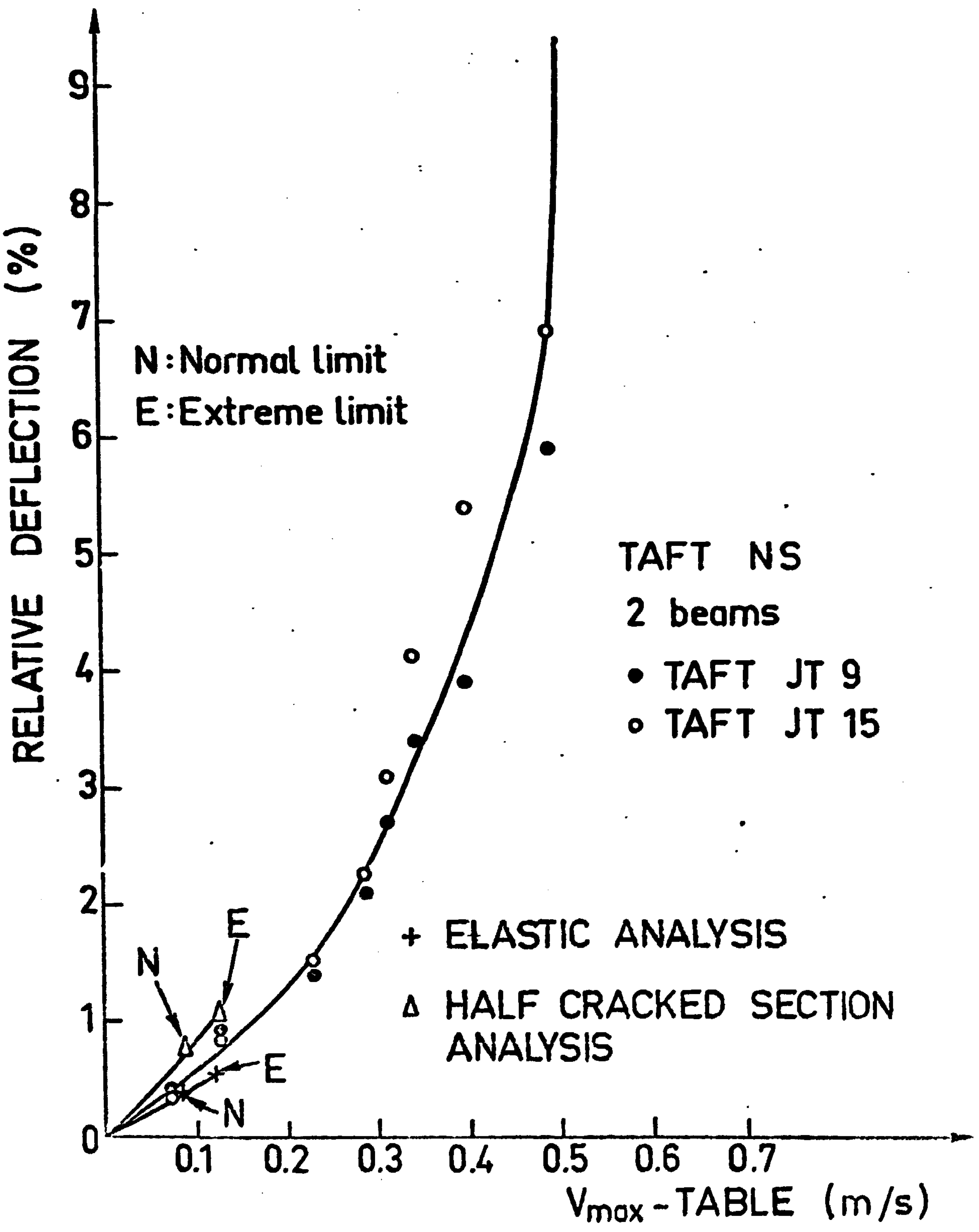
RESULTS ON FREQUENCY AND DAMPING


FREQUENCY (HZ)
5
4
3
2
TAFT N.S.
Vmax = 0.13 m/s
10
20
30
40
NUMBER OF EARTHQUAKES
DAMPING (%)
4
3
2
1
0
TAFT N.S.
Vmax = 0.13 m/s
10
20
30
40
NUMBER OF EARTHQUAKES

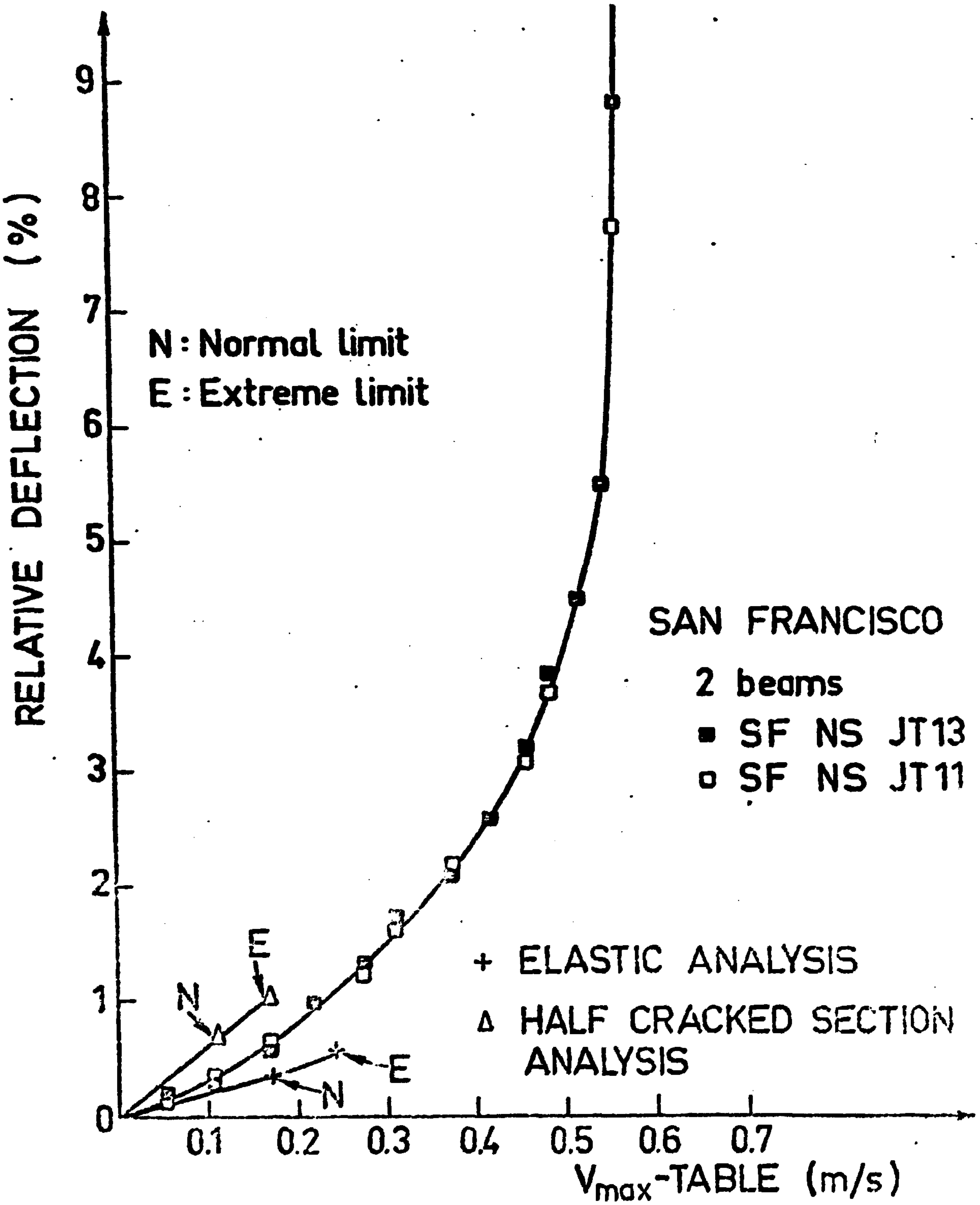


DEFLECTION RESULTS


1970 Kg



RELATIVE DEFLECTION (%)
9
8
7
6
5
4
3
2
1
0
N : Normal limit
E : Extreme limit
TAFT NS
2 beams
• TAFT JT 9
o TAFT JT 15
+ ELASTIC ANALYSIS
Δ HALF CRACKED SECTION ANALYSIS
N
E
E
N
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Vmax - TABLE (m/s)


RELATIVE DEFLECTION (%)
9
8
7
6
5
4
3
2
1
0
N : Normal limit
E : Extreme limit
SAN FRANCISCO
2 beams
SF NS JT13
SF NS JT11
+ ELASTIC ANALYSIS
Δ HALF CRACKED SECTION ANALYSIS
N
E
0.1
0.2
0.3
0.4
0.5
0.6
0.7
V_{max}-TABLE (m/s)