THE SEISMIC DESIGN OF AN INDUSTRIAL CHIMNEY WITH ROCKING BASE

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ABSTRACT:

An industrial reinforced concrete chimney, of cruciform cross-section and thirty-five metres tall, has been designed and built with the high degree of seismic protection afforded by allowing the base to rock during large earthquakes. As a precaution a pair of purpose-made yielding steel dampers have been installed to assist in the control of the most violent response that might occur.

This paper discusses the advantages in the adoption of this revolutionary design and describes the hand dynamic analysis carried out during the design process. Subsequent time-history computer analysis carried out to confirm the design method is reported on.

1.0 INTRODUCTION:

During the design of the new hangar and maintenance facilities at the Air New Zealand Engineering Base at Christchurch there was a requirement for an industrial chimney to be provided. As this chimney would be close to, and associated with, the prestigious hangar and maintenance complex an architectural solution was sought which would be more appropriate than the conventional guyed circular flue. Acceptable architectural and engineering criteria were met with the development of a reinforced concrete structure of cruciform cross-section to which the flues could be attached. The chosen form allows the eventual duplication of the chimney capacity while its freedom to rock gives it the ability to withstand very large earthquakes without requiring the substantial repairs that most reinforced concrete structures would need.

2.0 DEVELOPMENT OF THE DESIGN:

The preliminary architectural proposals showed a reinforced concrete tower of uniform cross-section with height and integral with its foundations. The cruciform shape gave four quadrants in each of which there could be attached one flue. The application of conventional engineering design practices would mean that this reinforced concrete structure would either have to be detailed to behave as a single ductile cantilever shear wall or be heavily penalised by having to have very large reserves of elastic strength provided. The provision of a ductile region, although it would be indeed difficult to detail, implies that damage might be expected whenever the earthquake induced loads approach those of the design level. In order to make use of the protection inherent in a ductile design of such a simple cantilever large deformations are required and the tower may be left with a permanent set.

The original shape was therefore modified to allow the chimney to rock on its foundations. Over the bottom half the tower was splayed out to provide greater resistance to over-turning and flanges were proposed for the outer edges of the four legs of the cruciform to increase their stability. The final dimensions of the chimney are shown in Figure 1.

By being able to rock, the chimney superstructure is protected from attracting larger seismic moments and shears than those required to produce lift-off. The superstructure may then be safely designed to remain elastic under these limited loads. Extra damping is provided by two steel hysteretic dampers, each of 120 kN yield force, placed at opposite corners of the base.

Any torsional stiffness of the chimney is greatly enhanced by the addition of horizontal diaphragms or ring beams at three intermediate levels.

3.0 DETAILS OF THE ROCKING MECHANISM:

It will be seen from an inspection of a cross section at the base of the chimney (Figure 1) that there are two possible modes of rocking of the superstructure. Rocking about the axis A-A will require a lesser ground acceleration than that about axis B-B as the latter mode involves a greater lever-arm to the centre of gravity of the chimney. The base is, in its 'at rest' position, therefore supported at each of four points, one under each of the wall flanges at the corners of a square of side 7050 mm. The supports are lead sheets of 380 x 380 x 16 mm. To produce a positive transfer of horizontal shear between foundation and superstructure a

50 mm dia. stainless steel vertical dowel, wrapped in a soft proprietary tape, protrudes from the foundation through each lead pad into the chimney base. Thus, for the expected rocking displacements, the chimney structure is prevented from walking off the foundation.

The size of the lead plates is such that stresses due to the chimney's dead load are not high enough to induce creep strains that would be significant in the life of the structure.

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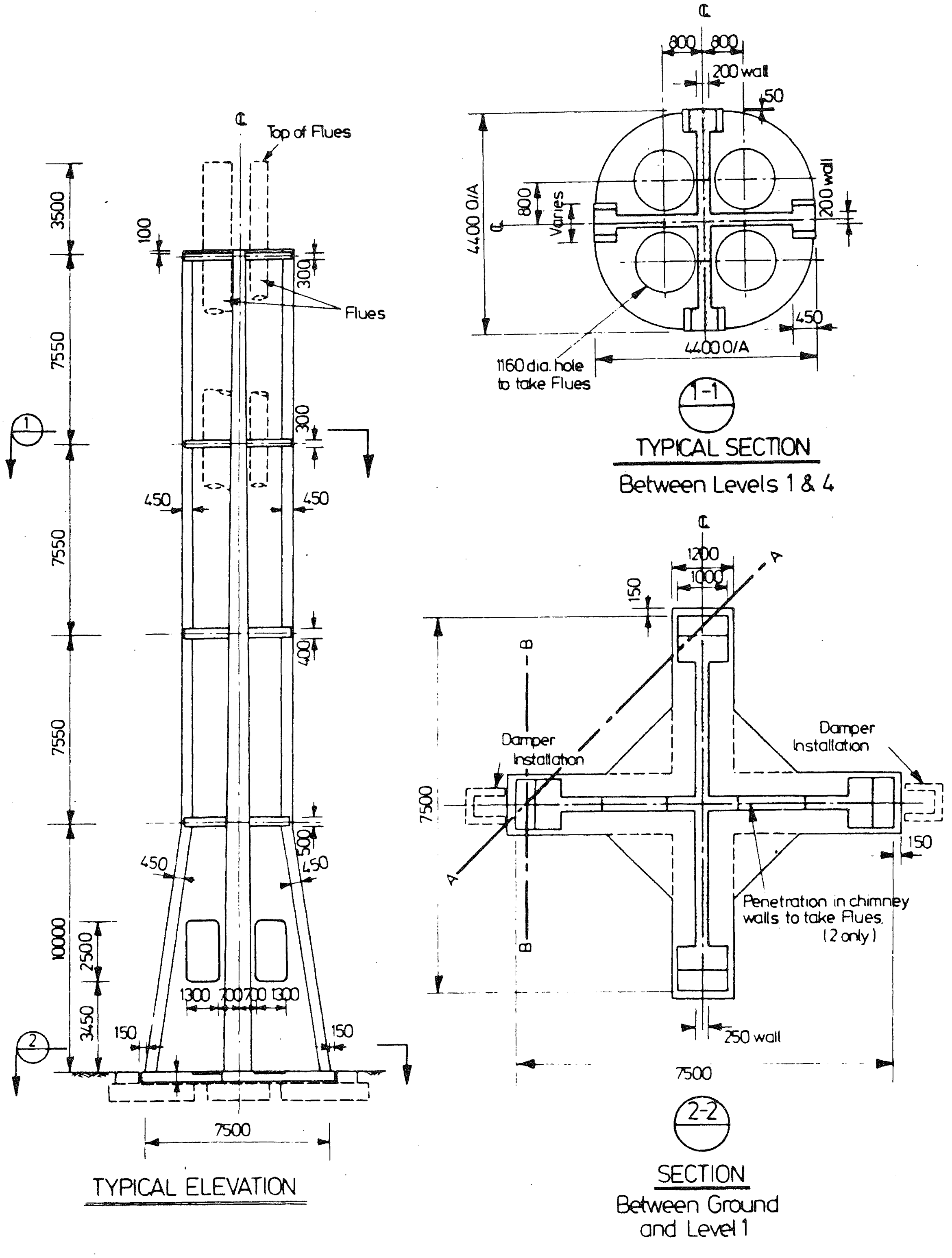

Figure 1 : CHIMNEY SUPERSTRUCTURE LAYOUT

4.0 HYSTERETIC DAMPERS:

Apart from the inherent material damping there is an energy loss to the system each time one of the rocking 'feet' comes down on its lead pad and radiates energy into the half-space below. Extra damping is achieved by placing two hysteretic dampers (Figure 2) diametrically opposite each other at the ends of one wall of the cruciform. With this choice of positions at least one damper will be effective whatever the direction of rocking. Developed by the NZ DSIR and manufactured from mild steel plate the hysteretic dampers are tapered cantilevers so that yielding is promoted along the taper where the moment capacity varies at the same rate as the moment. Each damper requires a force of 120 kN to yield it and so assists in preventing the chimney rocking prematurely during comparatively common wind storms.

The detailing of the dampers is such that the holding down bolts are positioned away from the critical yielding area during their operation.

5.0 WIND LOADING:

As well as being checked by conventional empirical methods for possible susceptibility to vortex shedding and galloping, a scale model was treated by the NZ DSIR in their wind tunnel without any unusual effects being recorded.

6.0 METHOD OF ANALYSIS:

6.1 Internal Forces -

Although true normal modes do not exist in a non-linear system, such as a rocking tower, modal concepts may be used to obtain approximate seismic forces and deformations for a relatively rigid tower. The chimney is thus treated as rigid when calculating forces and the deformations of only this first (rocking) mode are included when calculating sidesway. It is this approach which was used when designing this chimney.

By making the rigid body assumption the shape of the first (rocking) mode may be taken as triangular when computing forces. The higher rocking modes, with their insignificant deformations, have non-resonant responses which are in phase. 1'g' horizontal loads were applied to compute the forces in the first triangular mode (Figure 3). Having assumed that all the rocking action is included in the first mode it is a simple matter to equate the base moment corresponding to the triangular mode shape to the over-turning moment of the rigid chimney.

Under the condition where there is being a constant ground acceleration of 1 g applied to the base of the chimney, it follows that the sum of the chimney's lateral forces arising from modes higher than the first (triangular) mode must be the difference of the 1 g lateral loads and the first mode loads computed assuming a triangular mode shape. The rigid body rocking mode has therefore been divorced from the more conventional modes of the flexible chimney superstructure.

The forces that are acting on the chimney when it is rocking are therefore made up of a fixed component derived from the first mode rocking criterion and a component derived by conventional response spectra considerations applied to all significant modes higher than the first.

6.2 Displacements -

To find the maximum likely displacement of the chimney the approximate rigid rocking model of Figure 4 was used. The first step was to obtain the maximum displacement of the equivalent single mass m. It is shown by Newmark et al (1) that an approximate value for the maximum displacement of a single-mass non-linear system can be given by that of an equivalent linear resonator for which appropriate values of equivalent period and equivalent damping are defined. This technique has been used by Skinner et al (2) and the approximate correctness of the method has been confirmed by detailed analysis.

The lateral resistance of the model of Figure 4 is of a constant stiffness form up to the point at which pivoting about one support commences and then is of constant force for greater lateral displacements. This load-displacement relationship is the same as that for an elastoplastic single-mass resonator. Newmark (1) shows its effective period to be

$$T_1 = T_0 \frac{1}{3\mu} \left(1 + 2\mu^{\frac{3}{2}}\right) \qquad \text{(i)}$$

where T_0 = period for deformations less than those which will cause rocking.

and $\mu = \dfrac{\text{maximum lateral displacement of single mass}}{\text{lateral displacement at commencement of rocking}}$

Equation (i) may be rearranged to give

$$T_1 = \frac{4\pi}{3\sqrt{g}} \sqrt{\frac{D}{f_0}} + \frac{g}{12\mu^2} \frac{f_0}{D} T_0{}^3 \qquad \text{(ii)}$$

where D = maximum lateral displacement of mass

and $f_0 = \dfrac{\text{lateral force at commencement of rocking}}{\text{weight of single mass}}$

The hysteretic dampers for the stepping chimney were chosen to give an effective damping of five percent of critical for the rocking 'mode' of the chimney and this, therefore was the value assumed for the equivalent linear resonator to represent the model of Figure 4.

6.3 Hysteretic Damper Size -

The vertical coulomb damper force required to give an effective damping factor of five percent of critical was obtained by noting that a rectangular hysteresis loop gives a damping factor of 63% (Newmark (1)). This may be expressed as

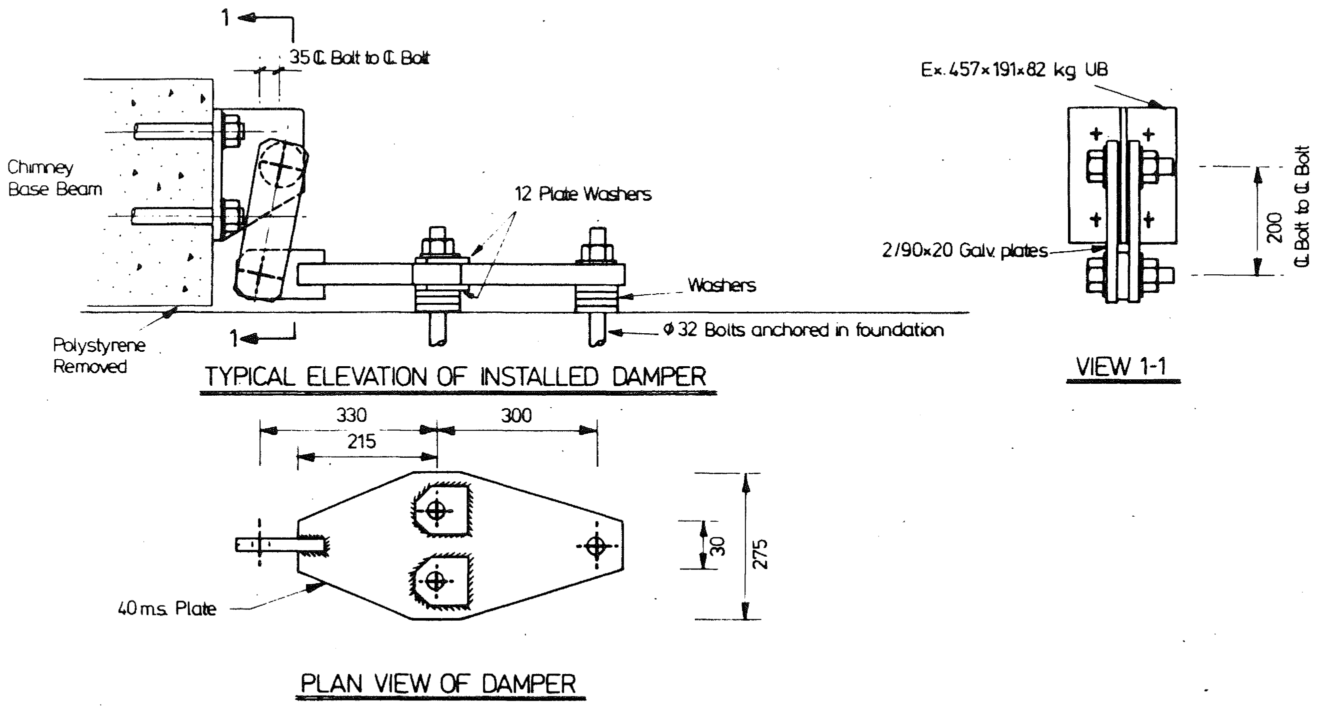

Figure 2 : DETAILS OF HYSTERETIC DAMPERS

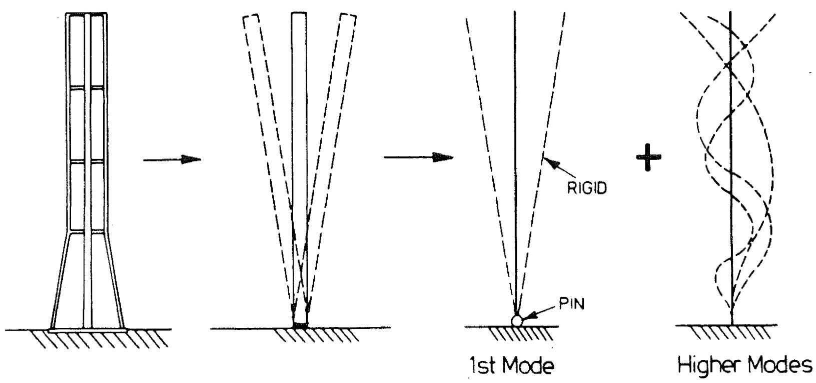

Figure 3 : IDEALISATION OF ROCKING MODES

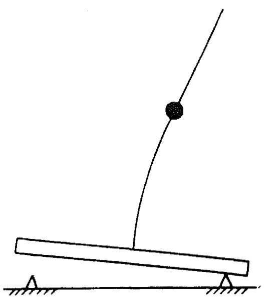

Figure 4 : MODEL OF SINGLE MASS ROCKING CHIMNEY

AXIS	LATERAL FORCE FOR UPLIFT, $f_o W$	MAX. MASS DISPLACEMENT (mm)	DAMPER UPLIFT (mm)
A-A (minor)	0.165 W	± 104	25
B-B (major)	0.233 W	± 69	23

TABLE 1 : MAXIMUM DISPLACEMENTS AND UPLIFTS FOR EL CENTRO SIZED EARTHQUAKES

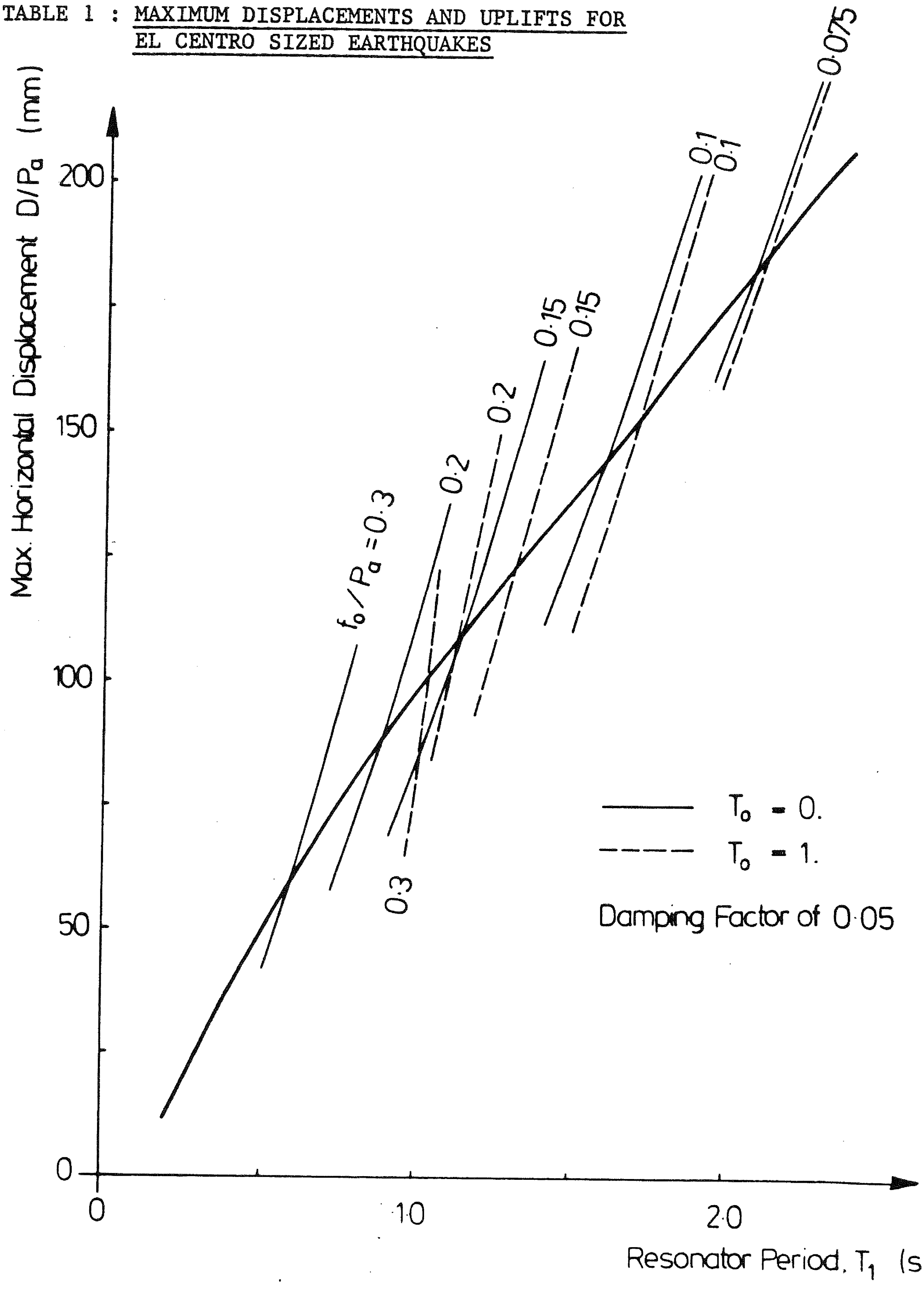

Figure 5: AVERAGE DISPLACEMENT RESPONSE SPECTRUM

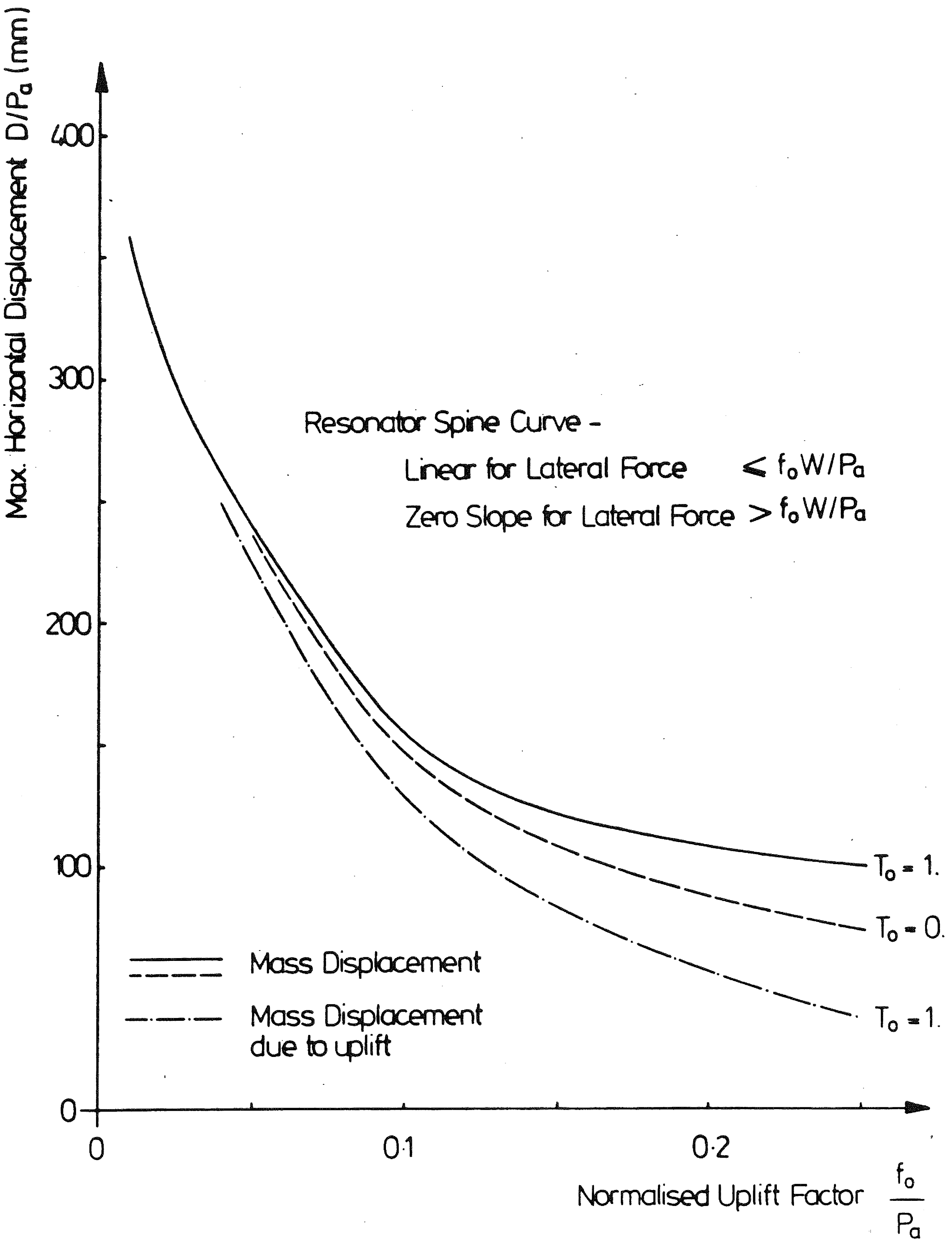

Figure 6 : MAXIMUM DISPLACEMENT vs UPLIFT FACTOR

$$\frac{F_D}{F_D+F_V} = \frac{\lambda}{63} \qquad \text{(iii)}$$

where F_V = vertical force at damper to lift weight of chimney

and F_D = hysteretic damper force

λ = effective damping percentage

Approximately then

$$F_D \cong (1+\frac{\lambda}{63}) \ \frac{\lambda}{63} F_V \qquad \text{(iv)}$$

Assuming that the chimney and foundation hysteresis provide an effective damping of 5% for motion up to the onset of stepping then equation (iv) reduces to

$$F_D = 0.086 \ F_V$$

The value of F_D can then be found simply by consideration of the overturning moment to calculate F_V.

6.4 Spectral Modal Response

Spectral curves which relate, for a specified recorded earthquake, the maximum displacements that would have been experienced by single mass resonators with various periods and damping factors are available (3). For the purposes of this design the weighted and averaged response spectra (Table 1 (3)) of a number of recorded earthquakes were used. The weighting was intended to give each of the constituent earthquakes approximately the same 'size' as that which occurred at El Centro, California on May 18, 1940 (N-S component). Figure 5 shows the resulting displacement spectrum for 5% of critical damping. Also plotted on this figure are the relationships of equation (ii) for $T_0 = 0$ and $T_0 = 1$ second. These relationships are given for various values of $\frac{f_0}{P_a}$ where P_a is a linear factor by which the averaged earthquake and its resulting displacement spectrum might be scaled.

Intercepts with the spectral curve give the relationships plotted in Figure 6.

The first natural period of the chimney (before rocking) was found to be $T_0 = 0.434$ seconds in both principal directions. The geometry of the chimney dictates the dimensionless forces f_0 to just produce rocking to be 0.165 and 0.233 for the minor and major axes respectively.

An estimate of the likely displacement response to the averaged earthquakes may then be found from Figure 6. The results found apply to the equivalent single-mass rocking model of Figure 4. Assuming again the approximation of the chimney's rocking mode shape being triangular, it is then a simple matter to produce equivalent maximum lateral displacements for the chimney top. These are plotted for both principal directions of rocking in Table 1.

The likely displacement for any recorded earthquake for which there is a displacement spectrum available may be similarly calculated for the rocking system.

It should be noted that loads increase only slowly with increased maximum acceleration (i.e. with scaling factor P_a) while displacement and rocking increase substantially with increased spectral velocity.

For this chimney, displacements of twice those given by Table 1 were adopted because of the possibility that the Christchurch area may experience ratios of spectral velocity to maximum acceleration somewhat greater than those for the El Centro May 18, 1940 type earthquakes.

7.0 TIME-HISTORY ANALYSIS:

Although it was thought that the previously described hand analysis was sufficiently accurate for the design of the rocking chimney, there later arose an opportunity to perform a computer analysis to give the time-history response of the structure subjected to the N-S component of time-history response of the structure subjected to the N-S component of the El Centro, May 18, 1940 earthquake. For simplicity the chimney was modelled as a two-dimensional structure. The particular elevation chosen was that corresponding to the superstructure rocking about its major axis.

7.1 The Computer Model -

The inelastic dynamic analysis program DRAIN-2D developed by Kanaan and Powell (4) was used to model a simple representation of the proposed chimney. This model, depicted in Figure 7, is made up of an assemblage of simple beam type members whose individual stiffnesses are combined to give an overall stiffness for the structure. Mass is apportioned to the model in a lumped form at the joints between members where appropriate. This mass will attract both horizontal and vertical inertia loadings. The ability to rock is given to the model by modelling the supports with a device which will resist compression loads only. This is achieved by including one of the computer program's truss elements at each support. These elements can be specified to as to resist tensile forces only. An inspection of Figure 7 will show how this feature can be effectively reversed.

No attempt was made to include the restoring effect of the potential energy associated with the small lifting of the centre of total mass of the chimney during rocking. Likewise, the dissipation of energy on the impact of the foundation beam on the lead pads was ignored. Because the expected magnitude of the period of rocking would imply reasonable slow impact velocities, an economical integration timestep of 0.01 s was selected.

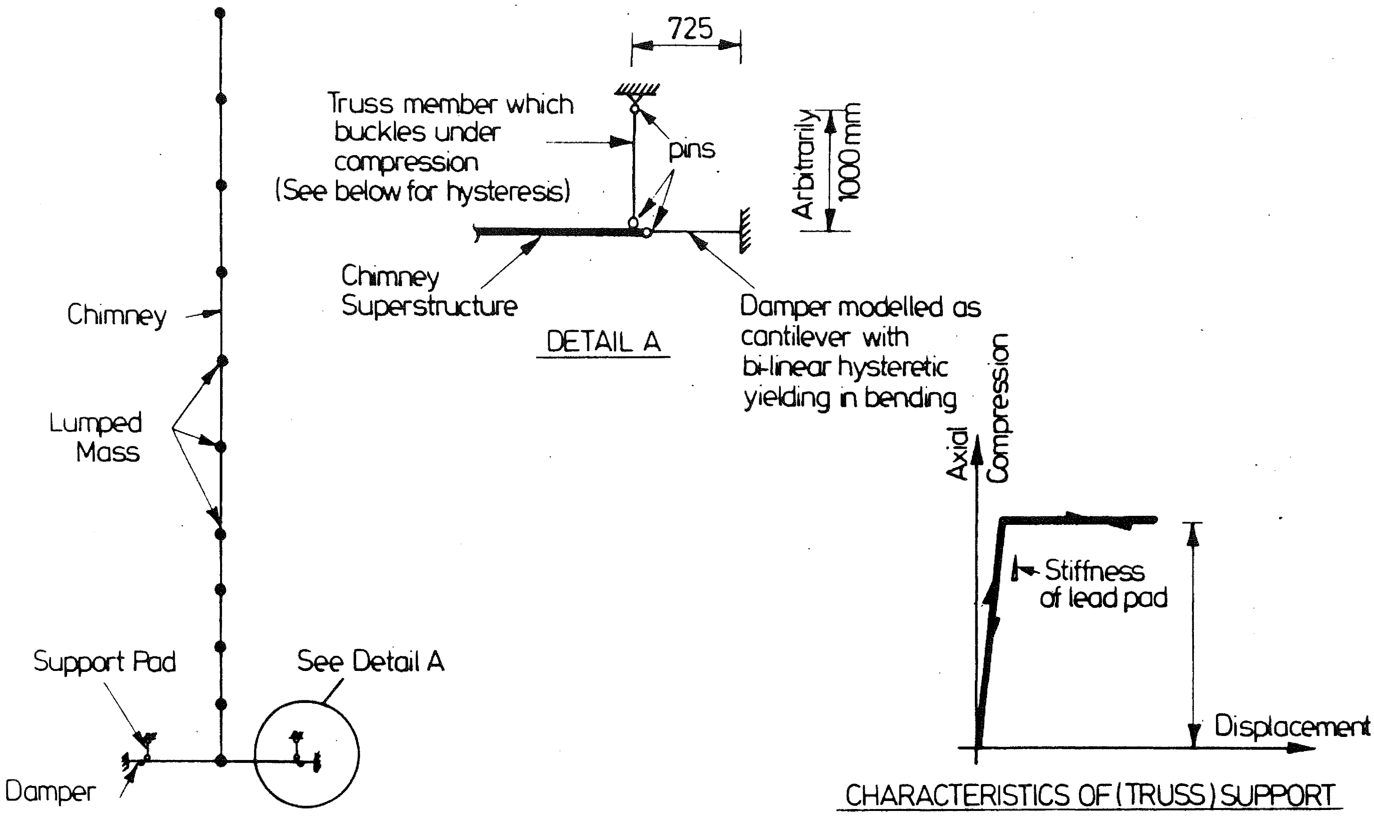

Figure 7 : ROCKING CHIMNEY MODEL FOR TIME-HISTORY ANALYSIS

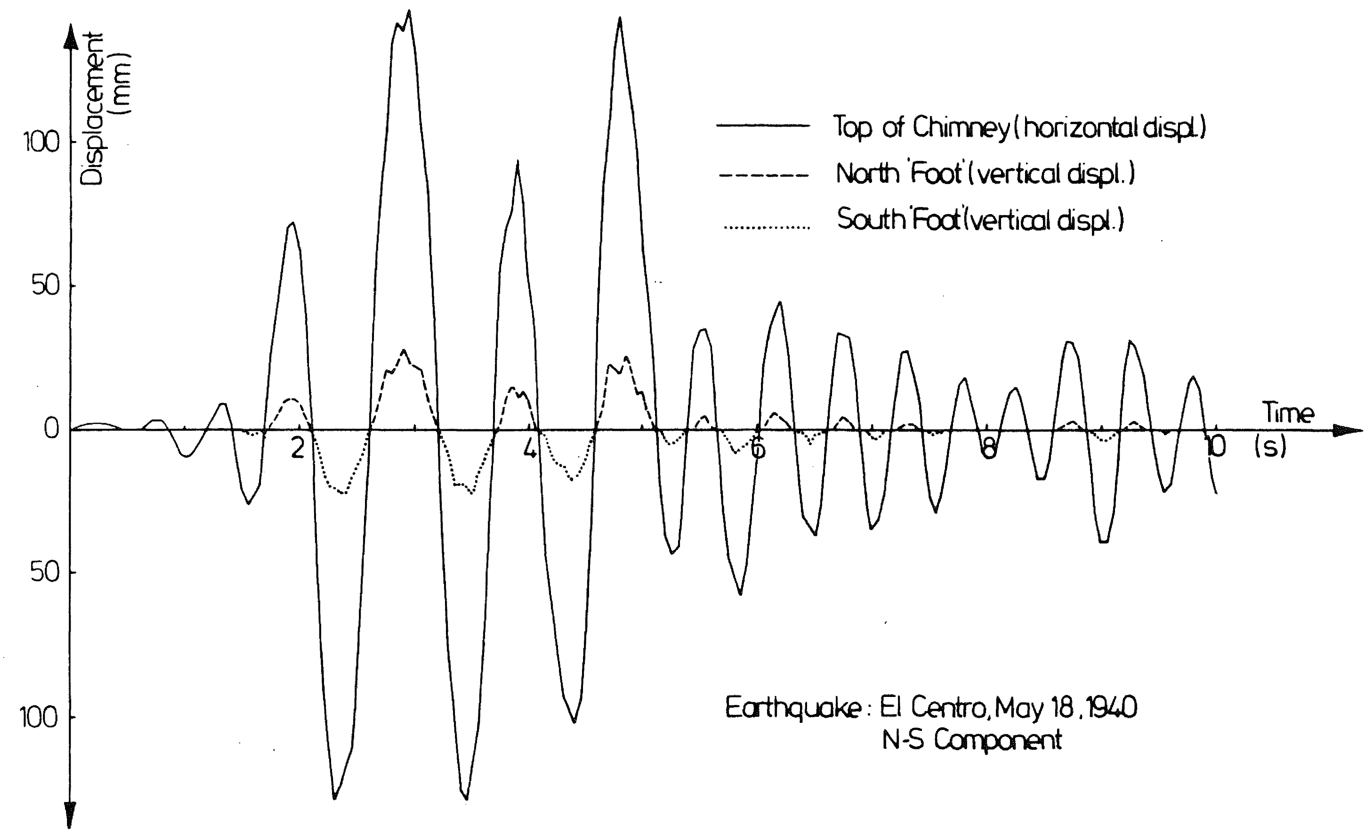

Figure 8 : DISPLACEMENT RESPONSE OF CHIMNEY SUPERSTRUCTURE

7.2 Time-History Analysis Results -

The essential results of the time-history analysis are presented in Figure 8 which shows the displacement response of both the chimney top and the base. The response had a fundamental period of approximately one second during the period of maximum response. This shortened to approximately 0.6 seconds as the earthquake decayed and the rocking motion diminished. The solution of equation (ii) for half-amplitude displacements of 140 mm and 30 mm gives rocking periods of 1.2 s and 0.6 s respectively. The maximum displacement response (top of chimney) due to the N-S component of the El Centro, May 18, 1940 earthquake is approximately 40% greater than that predicted for the averaged earthquakes represented by the displacement response spectrum used in the hand analysis. The flexural response of the chimney structure itself accounts for much of the difference between the observed displacement and that predicted from the use of the average earthquake displacement system.

8.0 CONCLUSIONS:

The described hand spectral analysis, while making some approximations, allows the general character of a wide range of earthquakes to be considered in design. The selection of a structural form which allows rocking during a severe earthquake provides a high degree of protection because the initial loads in the chimney then increase only slowly with increased maximum displacement. The displacements, however, increase substantially with increased spectral velocity (Figure 5).

A simple analytical rocking model has been presented which is compatible with available inelastic time-history analysis computer programs. Such a model may be used to make parametric studies of the rocking response of a proposed structure during the design process.

ACKNOWLEDGEMENT:

The detailed design of the chimney was undertaken by the Earthquake Engineering Group of Beca Carter Hollings & Ferner Ltd. The second author acted as a consultant to that team in the areas of the hand dynamic analysis, wind loading and damper specification.

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