EARTHQUAKE DESIGN

A STRUCTURAL ENGINEERS VIEWPOINT

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INTRODUCTION

The following is a brief review of seismic concepts, structural concepts and the footing/soil interface. It will touch on:

i) Basics/Fundamentals

- Reponse spectra.
- Ductility.
- N.B.C. Code Factors

ii) Building Structures

- Moment Frames.
- Walls.

iii) Footing Design

- Sliding/overturning.
- Working stress design.
- Factored load design.

In particular, the question of how exactly to size the footing to resist overturning will be raised. This review will not address problems of soil liquifaction, force amplification due to soils, etc.

BASICS/FUNDAMENTALS

Commentary J of the "Supplement to the National Building Code of Canada, 1985" (Ref. 3), starts off by stating that the main purpose of the code requirements for seismic design is to prevent loss of life and major structural failures, (i.e. - collapse), of buildings. It accepts that severe damage may occur during the design earthquake such that the building may have to be demolished after.

It states the current estimate for this occurring is one chance in ten over a 50 year building life, based on a 471 year return period for the design earthquake.

This acceptance of potential major damage is common to most building codes for seismic areas (i.e. - the U.S.A. and New Zealand, for example). It is based on the recognition that the potential elastic seismic forces can be orders of magnitude larger than wind loads etc., and that it would be extremely costly to design all buildings such that no damage occurred during the design earthquake.

The basis for the code approach is discussed in this section. The first three figures are taken directly from the Portland Cement Association book "Design of Multistorey Reinforced Concrete Buildings for Earthquake Motions" by Blume, Newmark and Corning (Ref. 1).

SEISMIC DATA

Recorded ground seismic data for the 1940 El Centro Earthquake is shown in Fig. 1. Note that the length of strong motion activity is about 15 seconds duration, the maximum recorded acceleration is about 30% of gravity, the maximum velocity is about 13 inches/sec., and the maximum displacement is about 8".

Records such as this are collected for a variety of earthquakes. However, as raw data they are not directly useful to structural designers. The next step is to try to determine how a building might react to these base ground motions. This is discussed next.

RESPONSE SPECTRUM

Since a building has a series of natural periods or modes of vibration, its response to an earthquake can be thought of as a "summation" of how it responds to the earthquake in each of its natural modes of vibration. This determination can be assisted by the creation of a "response spectrum" for various earthquakes by doing the following:

- take a single degree of freedom oscillator of natural period "T" and damping "B".

- feed the control earthquake to it at its base (analytically or experimentally).

- record its maximum response (i.e. - a single value) for acceleration, velocity, displacement.

- repeat for different natural periods "T" and damping values.

- plot these maximum values as functions of the periods.

The resulting curve of these maxima is the "response spectrum" for that earthquake. Fig. 2 is the acceleration response spectrum for the El Centro 1940 record for various damping ratios. Fig. 3 is a log-log plot on three different axes for acceleration, velocity and displacement for the same earthquake. By doing this for several earthquakes and smoothing the plot, a "design elastic response spectrum" can be generated, as is done in the N.B.C. commentary (Ref. 3).

Essentially, the elastic building response can be "built up" by assuming that each natural mode responds in a similar fashion as an oscillator with the same period.

The interesting thing to note in Fig. 2 is the response varies dramatically with period. This means, for instance, that a 2 second mode type building collects much less force than a short period .3 second mode building. Also note that for a .3 second period oscillator at 5% critical damping, the maximum lateral elastic acceleration response is about 90% of gravity for a maximum base input acceleration of 30% gravity. This means that at some point a force equal to 90% of the oscillator weight is being applied laterally. This is a large force and is difficult to design for. Codes deal with this by designing for reduced loads and allowing inelastic behaviour and thus damage.

DUCTILITY

It was noted that systems that were allowed to yield rather than remain elastic exhibited the following behaviour when excited by known earthquake records:

i) The maximum acceleration (i.e. - force) response was lowered.
ii) The maximum displacement response was about the same.

This led to an idealized structural behaviour shown in Fig. 4. Basically, the structure could be <u>designed</u> to lower force levels provided it was <u>detailed</u> in such a manner that would allow the required inelastic (plastic) deformation.

The lower the force level the greater the non-linear demand and hence the greater the detailing requirements. The result of this increased non-linear behaviour is often increased levels of damage. The elastic structure would return the zero displacement, but the inelastic structure might end up permanently displaced, hence damaged.

The ability of the structure to deform inelastically is an example of ductile (as opposed to brittle) behaviour. It is this concept of ductility that the codes use in determining design load levels.

CODE FACTORS

The 1985 National Building Code (Ref. 2) design force level is expressed as:

$$V = v*S*K*I*F*W*LF$$

where:

V = Design base shear.

v = Expected maximum ground velocity, based on curves similar to those in Fig. 1.

S = Response factor varying with fundamental building period. Basically reflects the response behaviour illustrated in Fig. 2.

I = Importance factor of building. Usually equals 1.0 with post disaster buildings using a value of 1.3.

F = Foundation factor. This varies from 1.0 to 1.5 and is meant to allow for site soil amplification effects.

W = Building weight.

LF = Load factor. This is 1.5 in the N.B.C.C. 1985 (Ref 2) and takes the working load and makes it an "ultimate" or "yield" load.

K = .7 to 1.3 - This factor reflects structure type and "ductility" requirements. K = 1.0 implies that the maximum or total deflection is about 3 times the "yield" deflection. Higher K values imply higher force levels and as such lower ductility requirements.

BUILDING STRUCTURES

There are four fundamental structural types used to resist lateral seismic loads:

i) Moment frames.
ii) Braced frames.
iii) Shear walls.
iv) Coupled shear walls.

Only two will be discussed here; walls and moment frames, which will serve to illustrate the concepts behind seismic force resisting structures.

FRAMES

Frames are systems of moment connected beams and columns, and are illustrated in Fig. 5A. Basically, the columns resist the lateral shearing forces and try to bend and displace sideways. This deformation causes the beams to bend and resist the column displacements, and to induce large vertical axial forces in the end columns. The net effect at the foundation is:

i) All the columns resist the lateral shearing force.

ii) The end columns resist the lateral overturning moments by developing a tension in one end column and a compression in the other. These forces must go into the footings and into the soil.

Because of the concept of "ductility", the frames will probably yield at various locations, and the preferred configuration is beam yielding as shown in Fig. 5B. This produces a multitude of "plastic yielding" regions in the frame beams and a relatively stable structure.

A second mechanism is column yielding as shown in Fig. 5C. This is not as desirable, as it produces fewer regions of hinging. It also produces a relatively unstable sway mechanism at the yielding storey.

SHEAR WALLS

Shear walls are basically large vertical beams cantilevering off the footing. This is illustrated in Fig. 6. The wall resists both the shear and overturning moment and transmits these forces directly to the footing. Because of the concept of ductility, the wall will probably yield and usually does so at or near the base, where the moment in the wall is largest. In this system, the footing is critical to resist overturning. In general, it should be stronger than the wall to force yielding into the wall and keep the footing elastic.

FOUNDATIONS/FOOTINGS

The design of foundations/footings for the structural engineer is an interesting process because of the following:

i) Allowable bearing pressures are usually given in "working stress" values.

ii) Occasionally, a 30% increase in allowable bearing stress is allowed for seismic/wind effects. Sometimes this is limited to a "toe pressure", sometimes not.

iii) The allowable bearing stress may be governed by strength limitations or by deflection criteria, and the deflection criteria may not be so critical under transitory seismic loading.

iv) Current codes tend towards "limit state" loadings which are factored, ultimate loads. These are generally used in the structural design and analyses, which leads to all sorts of number manipulations at the footing in order to use working stress on the soil.

v) For working stress design, the N.B.C.C. requires a factor of safety of 2 against overturning and sliding, with a cautionary note in the commentary about not assuming infinite soil stresses at the footing toe (Ref. 3). This leads to questions about sizing footings under walls for overturning forces.

The New Zealand Code for Loads and Procedures (Ref. 5), 1976 edition, suggests using a factored soil resisting stress of 1.8 times the allowable stress, applied as a uniform block to the toe of the footing to resist seismic overturning moments. This makes the structural designers job much easier.

vi) The N.B.C.C. 1985 (Ref. 2) now requires retaining walls to be designed for seismic effects on the soil pressure. This code, coupled with the new concrete code (Ref. 4), now require higher load factors to be used for retaining wall design. The old concrete code allowed a lower load factor if a soils report was done.

All these items lead to a variety of questions when sizing footings, which might be avoided if a "factored" soil resistance was provided as part of the soils report.

FOOTINGS UNDER FRAMES

For cases where the footings under the end frame column is a pad footing under uniform soil stress, it is relatively simple to convert back to working loads to size the footing. However, questions of interest would still be:

i) If deflection limits govern, could the stress be raised a bit?

ii) If the "usual" 30% increase is allowed, does it still apply if the soil stress is uniform over the footing?

FOOTINGS UNDER WALLS

The design of footings against overturning for walls is a little more complex. The basic distribution of forces is shown in Fig. 6. However, the footing size to satisfy soils stresses and overturning considerations varies depending upon what design criteria are applied. The following example illustrates this.

A footing will be sized for width for the following conditions:

- Footing length 33.8M (111).
- Dead load on footing - 225,500 kN (50,000 kips).
- Overturning moment - 1,356,000 kN:M (1,000,000 K-ft).
- Allowable working soils stress - 1436 kPa (30 ksf).

The load cases will be:

Working Loads - triangular stress distribution.

a). Working stress, no 1.33 factor.
b). Working stress - with a 1.33 increase on the allowable soil stress.
c). Working stress - with a doubled eccentricity, no 1.33 factor.
d). Working stress - with a doubled eccentricity and a 1.33 increase on the allowable soils stress.
e). Working stress - doubled eccentricity, infinite soils stress at toe.

Factored Loads - using a constant soils factored resistance of (say) 1.5 times the allowable soils stress, with no 1.33 increase applied. The load factors are N.B.C.C. 1985 (Ref 2).

f). Triangular soils distribution, DL = 1.25, LL = 1.5

g). Triangular soils distribution, DL = .85, LL = 1.5

h). Rectangular soils distribution, DL = 1.25, LL = 1.5

i). Rectangular soils distribution, DL = .85, LL = 1.5

The widths are:

	Width	
WORKING STRESS	Metres	Feet
A (not allowed by code)	18.3	(60')
B (not allowed by code)	16.8	(55')
C	30.5	(100')
D	28.9	(95')
E	24.4	(80')
FACTORED LOADS		
F	19.6	(64.6')
G	24.9	(81.9')
H	18.4	(60.5')
I	24.1	(79.1')

A few notes on the above:

i) The working stress method, with its arbitrary increase of 2 on the eccentricity gives big footings and is ambiguous about what soils stress to use at the "doubled eccentricity" condition.

ii) The factored load condition is more consistent, but currently there appears to be no common practice regarding "factored" soils resistance. The 1.5 factor used above was completely arbitrary.

iii) Footing sizes are functions of load eccentricities and allowable soils stresses, which are independent parameters. The difference in footing sizes in the example above are relatively insensitive to the "soils factor" used and the type of stress distribution, i.e. triangular or rectangular. These two parameters would have a larger effect at lower allowable soil stress values.

iv) The toe projection of footings past supporting walls have a direct affect on the footing depth, and thus on the volume, and costs. The smallest footing, consistent with the factors of safety used in the structure above, should be the goal of footing design.

SUMMARY

i) Buildings are designed for seismic forces that are lower than the expected "elastic response" forces. This is based on the building structure being able to yield or sustain inelastic deformation.

ii) Seismic loads in structures are resisted by a variety of systems - the most common being shear walls, braced frames, moment frames or combinations of these.

iii) The structural system passes the seismic loads down to the footings and, hence, to the soil. In general, it is considered best to keep yielding behaviour out of the soil and footings, make them stronger than the structure, and confine yielding to the structure.

From a structural engineers view, it would be desireable to:

a). Automatically get "seismic" recommendations for retaining walls.

b). Get "factored" soil resistances to be used with the current N.B.C.C. 1985 (Ref 2) factored load requirements. This would simplify and make more consistent most footing designs.

REFERENCES

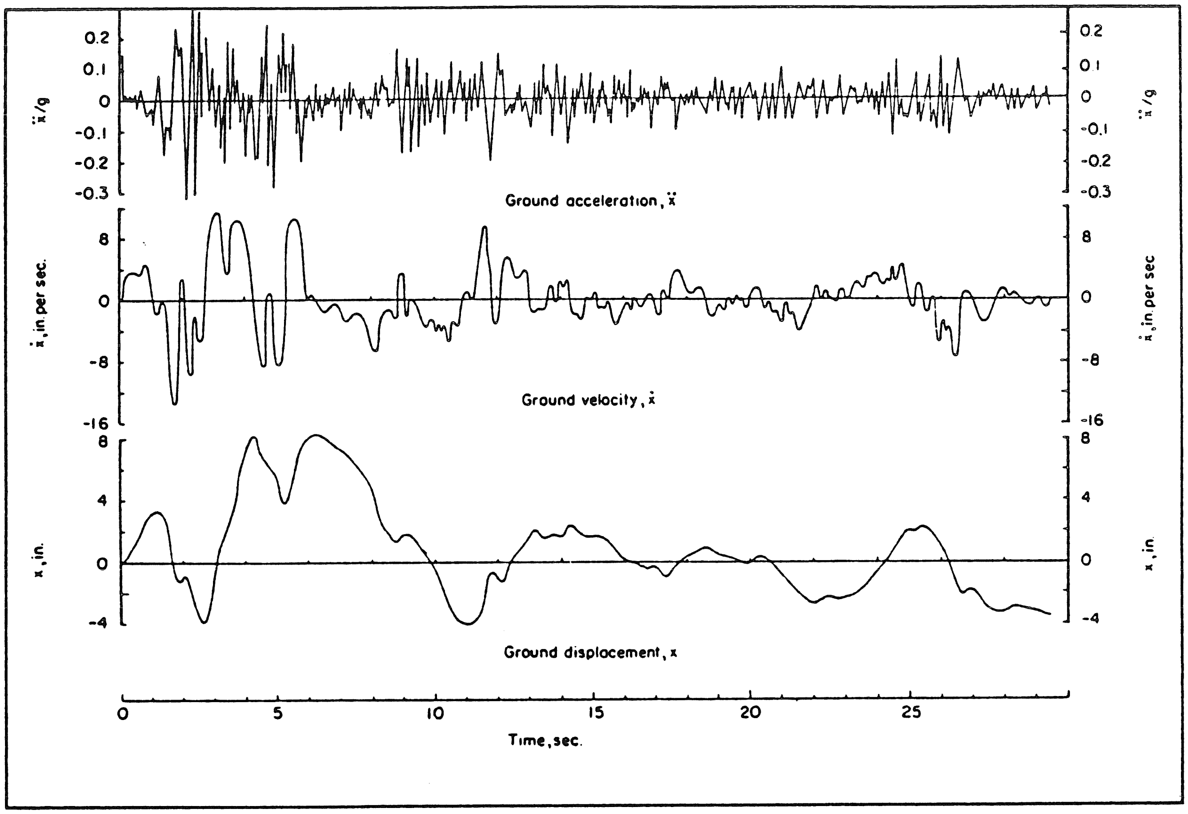
1. Blume, J.A., Newmark, N.M., Corning, L.H., 1961. "Design of Multistorey Reinforced Concrete Buildings for Earthquake Motions" - Portland Cement Association.

2. National Building Code of Canada - 1985.

3. Supplement to the National Building Code of Canada - 1985.

4. CAN3-A23.3-M84 - "Design of Concrete Structures for Buildings" - Canadian Standards Association.

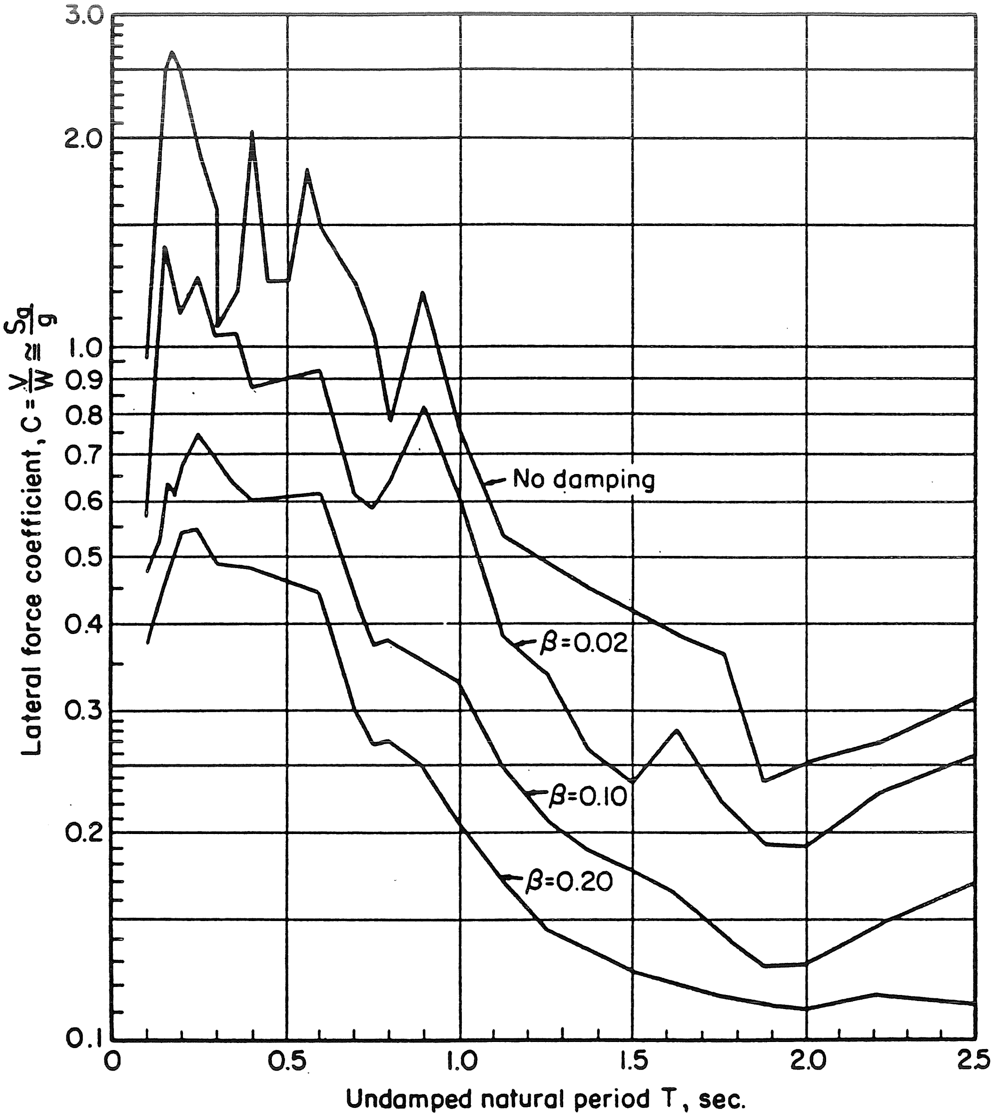
5. NZS 4203:1976 - "Code of Practice for General Structural Design and Design Loadings for Buildings" - Standards Association of New Zealand.



Ground acceleration, velocity, and displacement, El Centro, Calif., earthquake of May 18, 1940, N-S component.

REF 1.

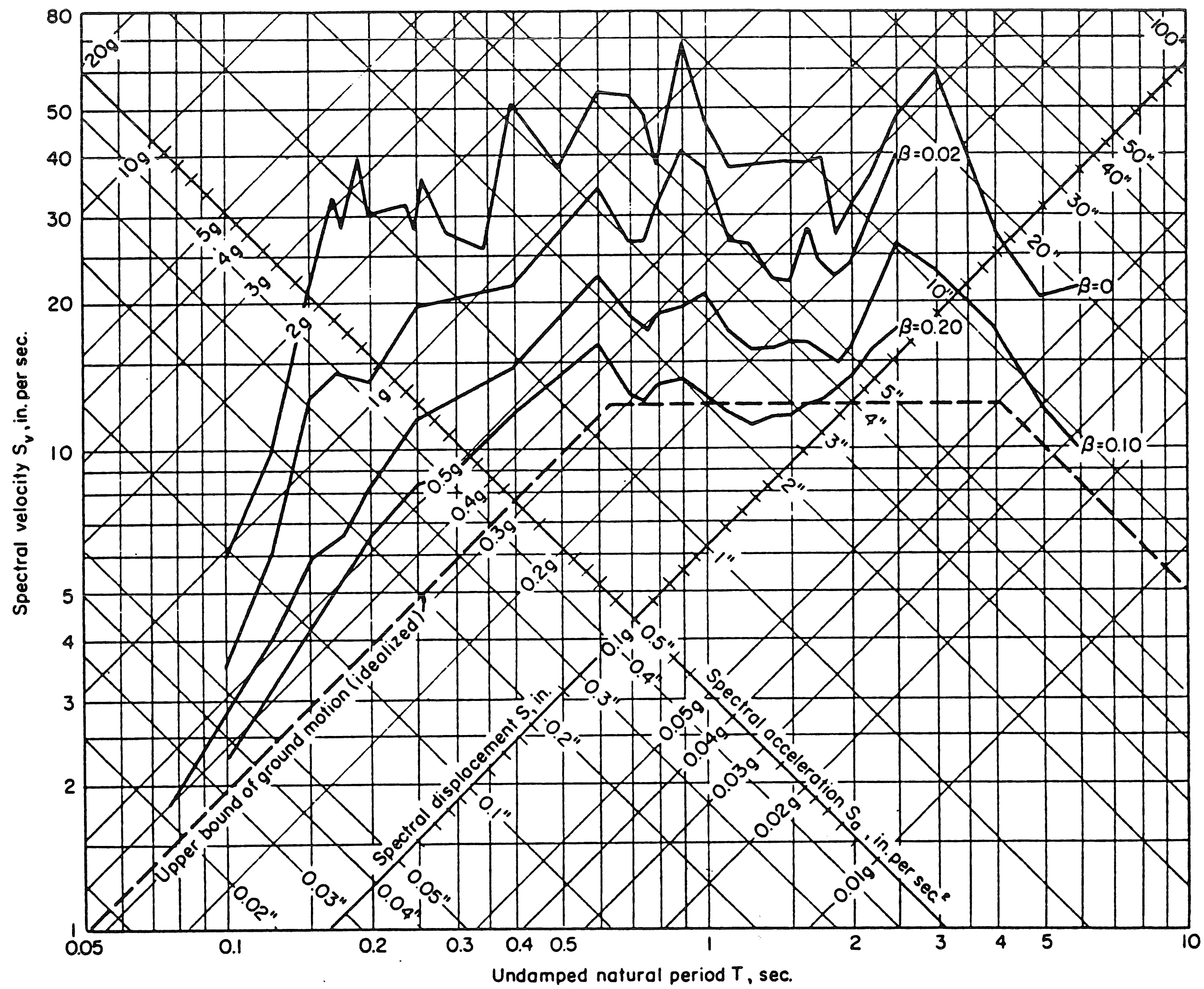
FIG. 1



Acceleration spectra for elastic systems, 1940 El Centro earthquake.

REF 1.

FIG. 2.



Response spectra for elastic systems, 1940 El Centro earthquake.

REF 1.

FIG. 3

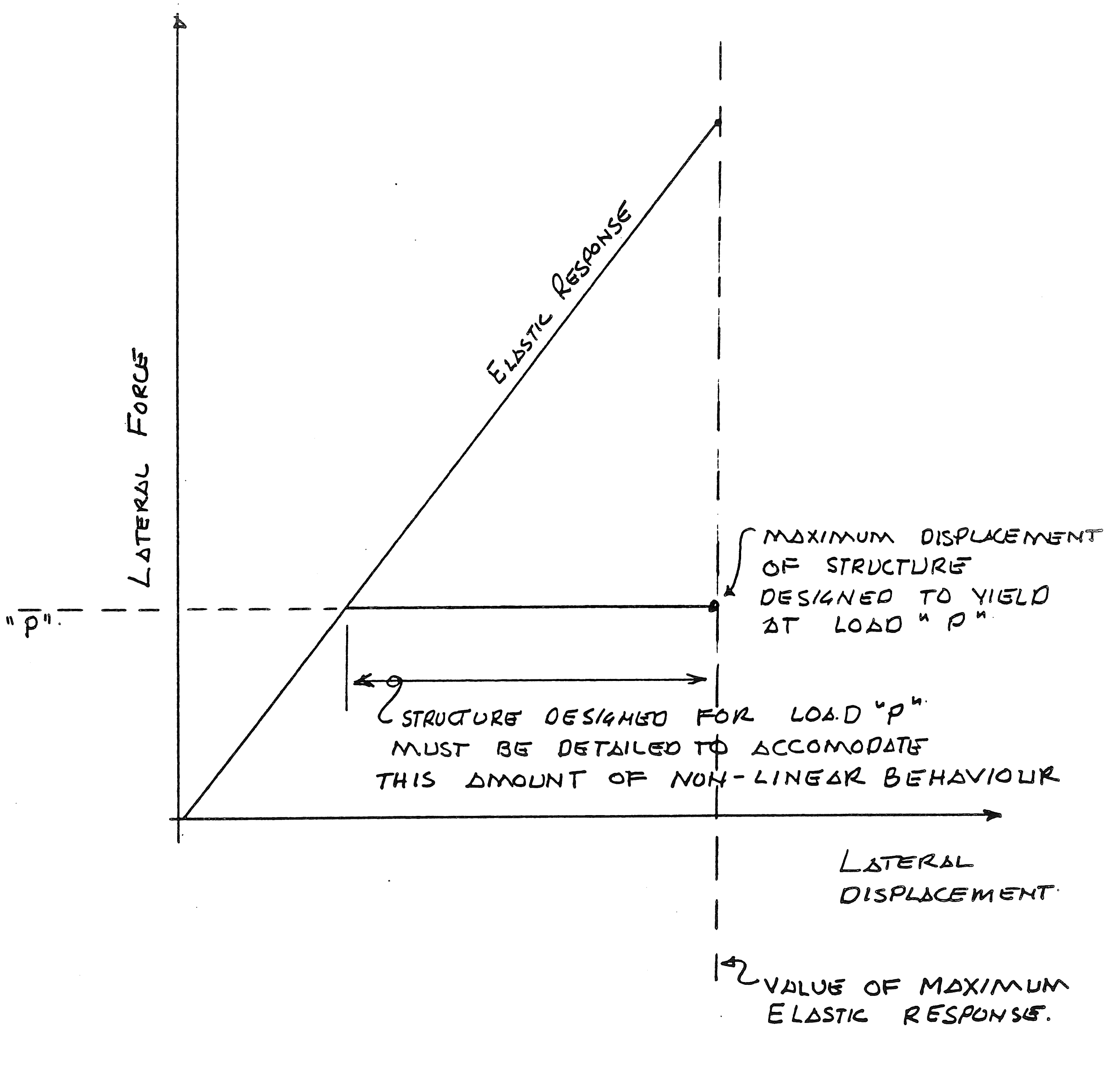
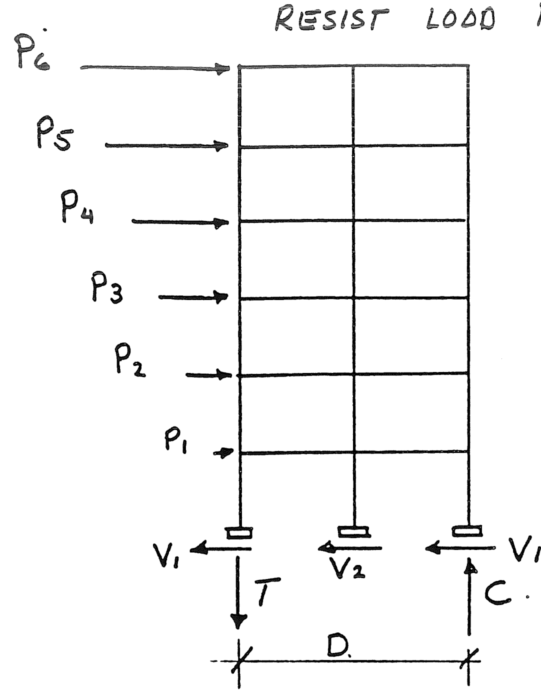

LATERAL FORCE
ELASTIC RESPONSE
"P".
MAXIMUM DISPLACEMENT OF STRUCTURE DESIGNED TO YIELD AT LOAD "P"
STRUCTURE DESIGNED FOR LOAD "P" MUST BE DETAILED TO ACCOMODATE THIS AMOUNT OF NON-LINEAR BEHAVIOUR
LATERAL DISPLACEMENT
VALUE OF MAXIMUM ELASTIC RESPONSE.

FIG. 4.

– SHEARS AT COLUMNS/FOOTINGS.
RESIST LOAD P.

$\sum P_i = \sum V_i$



– OVERTURNING MOMENT DUE TO LOADS P RESISTED BY COLUMN LOADS T, C.

$M_{EQ} = T{*}D = C{*}D$

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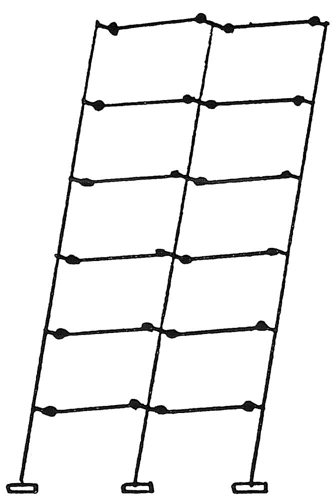

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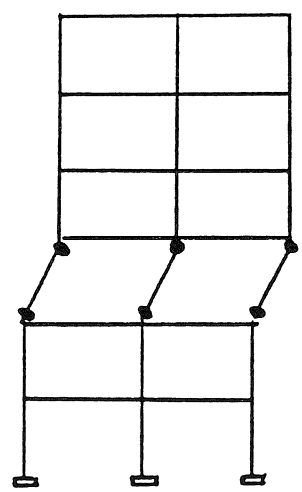

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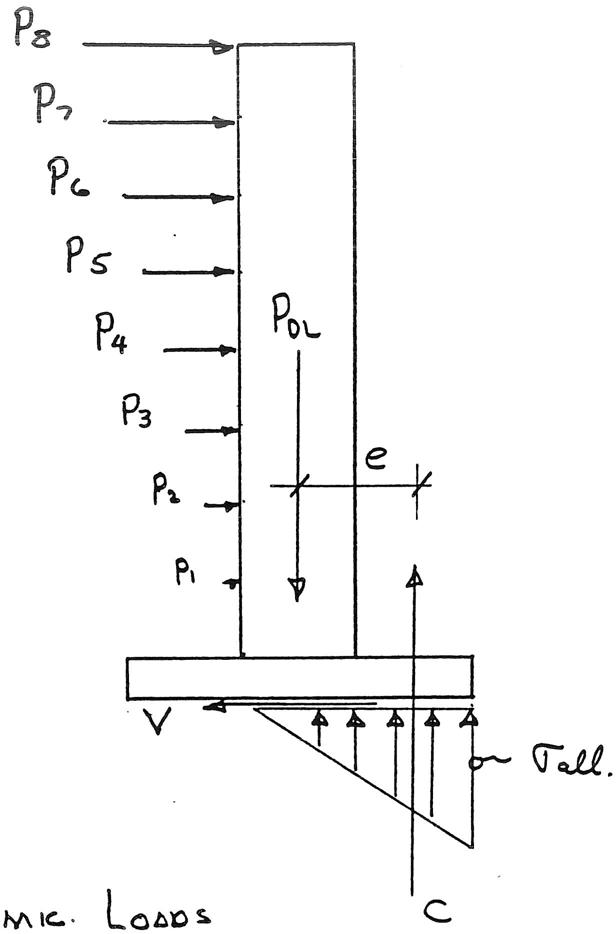
● = PLASTIC HINGE
REGION OF NON-LINEAR BEHAVIOUR.

FIG. 5.



P_i = LATERAL SEISMIC LOADS

P_{DL} = BUILDING DEAD LOAD
ON CORE WALL.

$V = \sum P_i$

$C = P_{DL}$ = RESULTANT OF SOILS STRESS.

$C * e = P_{DL} * e \Rightarrow$ COUPLE RESISTING SEISMIC OVERTURNING MOMENT

σ_{all} = ALLOWABLE SOIL STRESS.

FIG. 6