USE OF YIELD RATIO RESPONSE SPECTRA TO DESIGN YIELDING MEMBERS FOR IMPROVING EARTHQUAKE RESISTANCE OF BRITTLE STRUCTURE

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Synopsis

This paper describes the design of yielding members to be added to the porcelain columns which support 80 ton smoothing reactors of the N.Z. Electricity Department's High Voltage Direct Current transmission system, in order to improve their earthquake resistance.

It shows how the extent of the yield was estimated from the response spectra given by P.C. Jennings in "Earthquake Response of a Yielding Structure" and how the margin against collapse was estimated from Jennings and Husid's "Collapse of Yielding Structures during Earthquakes".

1. Introduction

In the N.Z. Electricity Department's High Voltage Direct Current (H.V.D.C.) power transmission link across Cook Strait, the currents in the line conductors are smoothed by 80 ton iron cored smoothing reactors, containing copper windings which carry the line currents at the line voltage of 250,000 volts with respect to earth. Sufficient insulation could not be built into these reactors to provide full insulation between the windings and the reactor tanks because of transport weight and size restrictions. It was therefore necessary to provide some of the required insulation to earth externally, so the complete reactors were each mounted on four porcelain insulator columns - each approximately 8'7" high by 3'0" in diameter.

As all this equipment is installed in an active earthquake area, the original design included a seismic factor of 0.25 g, but investigation some time after the installation of the H.V.D.C. link indicated that the natural period and the inherent damping of the reactors and their supports were such that they could hardly be expected to survive earthquakes 1/3rd as strong as say El Centro 1940 (NS).

Because of the size and cost of the smoothing reactors and because of their importance in the H.V.D.C. link it was considered desirable to improve the earthquake strength of their supports.

2. Plastically Yielding Beam Concept

Originally each of the reactors, along with its associated surge diverters, was mounted on a platform which was supported at the reactor end by four large porcelain insulators

and at the diverter end by two stacks of normal solid core cylindrical post insulators (Figs. 1 and 2).

In the aseismic design, fabricated steel bosses are to be solidly mounted on the top and bottom of each of the large insulator columns and pin jointed to the foundation slab at the bottom and to the platform at the top. "I" section steel beams welded between the bosses were dimensioned so that they would yield plastically at a selected acceleration of the reactor (see Fig. 3). "I" beam sections were used so that as much as possible of the beam material across the yielding section would become plastic at the same time. In this way it was endeavoured to approach idealised load - deflection curve behaviour of the beams and so use the yield point "cut-off" to limit the bending stresses induced in the insulator columns during an earthquake.

3. Design Figures

3.1 Design Acceleration

When the beams bend under the action of an earthquake, they form double cantilever systems, as do the insulator columns. Because of the general symmetry of the structures it was permissible to assume that the points of inflexion would be at the mid-points of the beams and of the insulator columns, and so all members were reduced to cantilevers of length half span for calculation purposes.

The combined effect of the bending and direct stresses in the porcelain insulator columns was estimated using the empirical relation

$$\frac{1}{S_c} = \frac{1}{S_b} = \frac{1}{S_t}$$

where S_c = combined factor of safety

S_b = factor of safety in bending alone

$$= \frac{\text{breaking stress in bending}}{\text{actual stress in bending}}$$

and S_t = factor of safety in tension alone

$$= \frac{\text{breaking stress in tension}}{\text{actual stress in tension}}$$

If the reciprocals of the "Factors of Safety" are regarded as "Loading Factors", then the empirical relation can be restated

"The combined loading factor is equal to the sum of the component loading factors".

The porcelain columns are custom built and

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are so large that the stresses which occur under load are greatly influenced by flexibility of the end plates and fastenings and by the degree of tightening of holding down bolts. Tests to establish the statistics of porcelain strength would have been prohibitively costly so the manufacturers gave the "ultimate strength" of the porcelain as 115 kg/cm² on the basis of their experience and engineering judgement.

When questioned they agreed that this figure could be regarded as strength in pure tension and that it was not unreasonable to take 230 kg/cm² as the corresponding strength in bending.

Works tests with strain gauges showed that tension stresses from bending exceeded those calculated from simple beam theory by up to 20 or 30% and compression stresses were appreciably larger still.

These figures together with a factor of safety of 2.5 (stress at failure/maximum dynamic stress) indicated a figure of 0.28 g as a desirable design acceleration. A further factor of 1.3 was allowed for increased beam resistance beyond the yield point and so the design acceleration for the yield point of the beams was taken as $\frac{0.28}{1.3} = 0.22$ g.

3.2 Design Natural Period of Oscillation

Long natural periods of oscillation of the aseismic structures were required to keep away from the natural period of the ground (0.25 seconds as measured by our strong motion recorder on the site of one pair of the two pairs of reactors - fig. 4) and to keep away from the "hump" of the Design Earthquake Response Spectra provisionally adopted for plant design purposes by the Department(1, 2). On the other hand, it was desired that the period should be short enough to keep the maximum reactor displacement low and so maintain stability and avoid requiring large flexibility in the electrical connections. A compromise design period of 0.5 seconds was chosen.

4. Design Development

4.1 Beam Design

Measured characteristics of the reactors and their original structures were:-

Direction of shaking	Natural period of vibration (seconds)	Damping (% of critical)
N - S (short beams)	0.316	1.4
E - W (long beams)	0.212	1.3

In designing the yielding beams, the beam section moduli, Z, were chosen to give a yield acceleration of the reactors of 0.22 g, and the beam moments of inertia, I, were chosen to give the aseismic structures a natural period of vibration of 0.5 seconds. The beam dimensions were fixed by these chosen values of I and Z. In calculating the stiffness of the yielding beams, allowance was made for the flexibilities of other members of the structures.

4.2 Extent of Yield in Beam Material

Jennings (3) presents "displacement spectra", one of which is reproduced in Fig. 5, which give the average values of the ratios

$$\frac{\text{maximum displacement}}{\text{yield displacement}} = \left| \frac{X}{X_y} \right|_{\text{max}}$$

obtained by applying his eight pseudo-earthquakes, to the structure model shown, for ranges of acceleration ratio, damping, and natural period.

He defines "acceleration ratio" as

$$\frac{\text{r.m.s. value of the earthquake acceleration record}}{\text{acceleration corresponding to yield stress in structure}}$$

and gives the r.m.s. acceleration for a number of earthquake records including El Centro 1940 (NS) which is 2.2 ft/sec².

The Department's Design Earthquake was derived by Skinner (1) from eight earthquake records scaled up to the level of El Centro 1940 (NS) and it was therefore considered appropriate to take 2.2 ft/sec² as the "design earthquake r.m.s. acceleration". Thus, for the structure with a yield acceleration of 0.22 g, the "design earthquake acceleration ratio" is

$$= \frac{2.2}{0.22 \times 32.2} = 0.31$$

It is believed that this design of structure will have inherent damping of the order of 2% of critical.

Taking : natural period 0.5 sec.
damping 2% of critical
acceleration ratio 0.31

Jennings curves, reproduced as Fig. 4, indicate an average value of $\left| \frac{X}{X_y} \right|_{\text{max}}$ of 2.5.

That is, in an earthquake of this size the maximum platform displacement can be expected to be of the order of 2.5 times the yield displacement.

The yield deflection of the reactor platform at 0.22 g was calculated to be 0.535 inches, of which 0.32 inches was provided in one direction by elastic deflection of the shorter beams. Design earthquake maximum deflection was taken as 2.5 x 0.535 i.e. 1.34 inches of which 1.34 - 0.535 i.e. 0.81 inches was assumed provided by plastic yielding of the short beams. The lengths of the plastic hinges in the yielding beams were taken as $\frac{1}{4}$ x the half span (4), plus 1" to allow for the tapering boss stub beams. This gave a plastic hinge length of 4" for the shorter beams. Geometrical treatment of the elastic and plastic beam deflections indicated a maximum total strain to be expected of the order of 0.85%.

A total strain of the order of 0.7% was estimated for the longer beams.

Bertero and Popov's experiments (6) on low cycle fatigue of beams of approximately the size used in the present structure showed

that cyclic loading beyond the yield point can be withstood a 100 or more times if the maximum strain is less than about 1%.

Thus it can be reasonably expected that the structure could withstand a number of earthquakes of the "design" size without suffering fatigue failure.

4.3 Margin of Stability Against Collapse

In reference (5) Jennings and Husid study the factors which influence the collapse of single storey yielding structures in strong earthquakes using a model to which our structure closely approximates.

They establish that a yielding structure of this nature will collapse eventually if shaken for a length of time which can be estimated from the height of the structure, its yield acceleration, the intensity of the shaking, and the slope of the force deflection characteristic beyond the yield point. For an elastoplastic structure the mean time to collapse is given by

$$\bar{T}_0 = 2000 \frac{L}{g^2}$$

where L = height of structure

$$\theta = \frac{E}{a_y/g}$$

$$E = \frac{\text{earthquake intensity ratio}}{\text{r.m.s. of accelgration record (ft/sec}^2\text{)}} \\ = \frac{0.697 \text{ (ft/sec}^2\text{)}}{0.697 \text{ (ft/sec}^2\text{)}}$$

0.697 = r.m.s. acceleration of the ensemble of artificial earthquakes used in the study.

a_y = yield acceleration of the structure (i.e. steady acceleration of the ground which would just cause yielding).

The averaged spectra used by the Department for design purposes generally represent earth shaking about the intensity of El Centro 1940 NS for which the R.M.S. acceleration is 2.2 ft/sec² and the "Jennings and Husid" E ratio is $\frac{2.2}{0.697} = 3.15$. These values therefore approx-

imately represent our design earthquake so that the time it would have to continue to cause collapse of our structure which has a yield acceleration of 0.22 g and height 10.3 feet can be estimated thus:

$$\theta = \frac{E}{a_y/g} = \frac{3.15}{0.22} = 14.3$$

$$\bar{T}_0 = 2000 \frac{L}{g^2} \\ = 2000 \frac{10.3}{(14.3)^2} \\ = 100 \text{ seconds.}$$

Jennings and Husid point out that their equation implies that for a given structure the time to collapse is inversely proportional to the square of the earthquake intensity, and from this two interesting points can be deduced.

(a) In an earthquake 50% stronger than the design earthquake, the time to collapse

would be of the order of $100 \times \left(\frac{1}{1.5}\right)^2 = 45$ seconds.

(b) In a very short jolting type of earthquake such as Parkfield 1966 (max. ground acceleration 0.52 ft/sec²) the intensity would have to be many times that of the design earthquake to produce collapse in the short duration time.

The above estimates are based on an idealised elastoplastic yield characteristic and Jennings and Husid show that practical structures which give increasing resistance beyond yield point have greater margins against collapse.

4.4 Surge Diverters

The surge diverters are mounted on an extension to each platform (Fig.2) and present two design problems.

(1) In the longitudinal direction they form cantilever appendages of period 0.2 seconds and weight about 0.015 that of the main structure (including the reactor) whose period is 0.5 seconds.

The two body response spectra derived by Penzien and Chopra (7) indicate that the response of the diverters in an El Centro 1940 NS type earthquake would be about 1.3g if the main structure remained elastic. However, the yielding characteristic of the support structure will reduce the maximum acceleration applied to the diverters and further work is required to determine the amount of the reduction and what additional protective measures if any are required.

(2) The platform extension and diverters form an asymmetric load which may produce torsional effects. Provision of yielding attachments for the lighter porcelain columns which support the platform extensions was considered but it seems likely that an arrangement of insulated diagonal bracings fitted with springs and hydraulic dampers may be necessary to protect the diverters in the transverse direction and to provide torsional damping for the whole structure.

5. Structural Design Considerations

The translation of the design figures into a practical design presented several problems, the main one being the design of a suitable beam-to-boss joint.

The bosses fitted to the tops and bottoms of the insulator columns had to each contain a conical component to transmit the large down-thrust loading (60 tons max/insulator column) from the insulator column walls to the "pin joints".

Because the point of yielding in the beams was required to be accurately controlled in order to control the yield bending moment, the moments of inertia of the beams had to be increased from their points of yielding toward the bosses in order to "beat" the increasing bending moments. This necessarily involved a discontinuity right at the point of yielding, and so this part of the design was carefully investigated.

Material with good fracture toughness was required for the yielding beams because of the

severe loading duty. A convenient method of construction would have been to cast beam stubs containing the yielding sections integrally with the bosses. However, this would have required special casting materials, and supply difficulties ruled these out.

In the design finally adopted, notch ductile steel discs were welded to the conical parts of the bosses (fig. 3). These discs were profile cut with stub beam flanges projecting from them, and these flanges were tapered at their outer ends down to the beam flange thickness. With this change in flange thickness along with the changing flange width (to give a smooth transition from beam to boss) satisfactory stress gradients in the stub beams were obtained.

The aseismic supports for the reactor ends of the platforms are at present in the course of manufacture. It is planned to test load a spare yielding beam and boss assembly by taking it through a number of cycles into the plastic range.

6. Conclusions

Yielding components have been used to convert a brittle structure into a yielding structure with substantial improvement in earthquake resistance.

The design was greatly facilitated by the fortunate similarity of the support structure to the models chosen for computer analysis in references (3) and (5) and by the experiments reported in reference (6).

The same detailed care in the design and manufacture of the components was required for these relatively small structures as is given to large steel buildings to minimise the risk of failure under earthquake loading.

The work is expected to cost approximately \$9,000 for each of the four reactors, which are essential to the operation of the \$38 million High Voltage Direct Current Transmission Link.

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8. References

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The tests showed the design to be unsatisfactory for the present purposes primarily because of the use of rectangular sections with a high ratio of plastic to elastic modulus. They are fully described in Ministry of Works Central Laboratories Report No. 342 "Benmore-Haywards H.V. D.C. Reactors -- Static and Dynamic Testing of Ductile Springs".

Fig. 1. The Original H.V.D.C. Smoothing
Reactor Installation

Fig.2. Plan of Reactor and Diverter Platform

SECTION THROUGH SUPPORT COLUMN.

PART No.	DESCRIPTION
1	Conical casting
2	Boss upper stub beam disc
3	Spacer ring
4	Boss support plate
5	Support plate stiffener
6	Boss cylinder
7	Boss lower stub beam disc
8	Bolt (with head)
8A	Bolt (without head)
9	Washer - flat, special
10	Nut - special
11	Keeper
12	Rocker plate - bottom frame
12A	Rocker plate - top frame
13	Boss cylinder stiffener
14	Sole plate
15	Short beam flange
16	Short beam web
17	Long beam flange
18	Long beam web

GENERAL ARRANGEMENT OF SUPPORT STRUCTURE.
Scale - N.T.S.

Fig.3. Details of the Aseismic Support Structures.

A. 225° B. 135° C. Vertical

Time Base: 0.2 sec/dot

Fig.4. Ground Acceleration Record at Haywards
Substation

Jennings' Fig.10 in Ref.3.

Fig.5. Average of Displacement Spectra for Yielding Structures.