INANGAHUA EARTHQUAKE DAMAGE ON RAILWAYS

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1. Introduction

The Inangahua earthquake of the 24th May 1968 disrupted rail communications over a wide area of the West Coast. At that time the writer's main efforts were aimed at restoring the severed communications - engineering observations were incidental to the main task. This paper reflects some of these observations. It is not intended as a serious dialogue but rather a vehicle to encourage thinking on the question of design standards and construction methods for earthworks and culverts in country where earthquakes are likely to occur.

1.(1) Geography of area

Inangahua (fig. 1) on the Stillwater-Westport Line is an important railhead for timber and lime from the Nelson Province destined for the West Coast. The township is at the junction of the Inangahua and Buller Rivers.

South of Inangahua the line runs mainly on flat land or on raised embankments - this portion of the line was opened in 1914. In 1943 the line to the north was opened. Here the line was constructed on sidlings hugging the lower reaches of the Buller River.

1.(2) Damage—the type and extent

In general where the line was laid in firm flat ground, little foundation damage was encountered. Only the track was distorted and the ballast section wasted. But where the track was laid on embankments these settled. And where there were sidlings, slips, rockslides and rockfalls occurred.

The bulk of the damage was concentrated in a 20 mile radius of Inangahua. By and large the damage was greater nearer to the epicentre. But this was not consistently so, because of the engineering criteria referred to elsewhere in the paper.

In order to restore the track to normal operations, the following earthworks were necessary:

Filling	19,000 c. yds.
Slip clearance	30,000 c. yds.
Ballast & sub ballast	42,000 c. yds.

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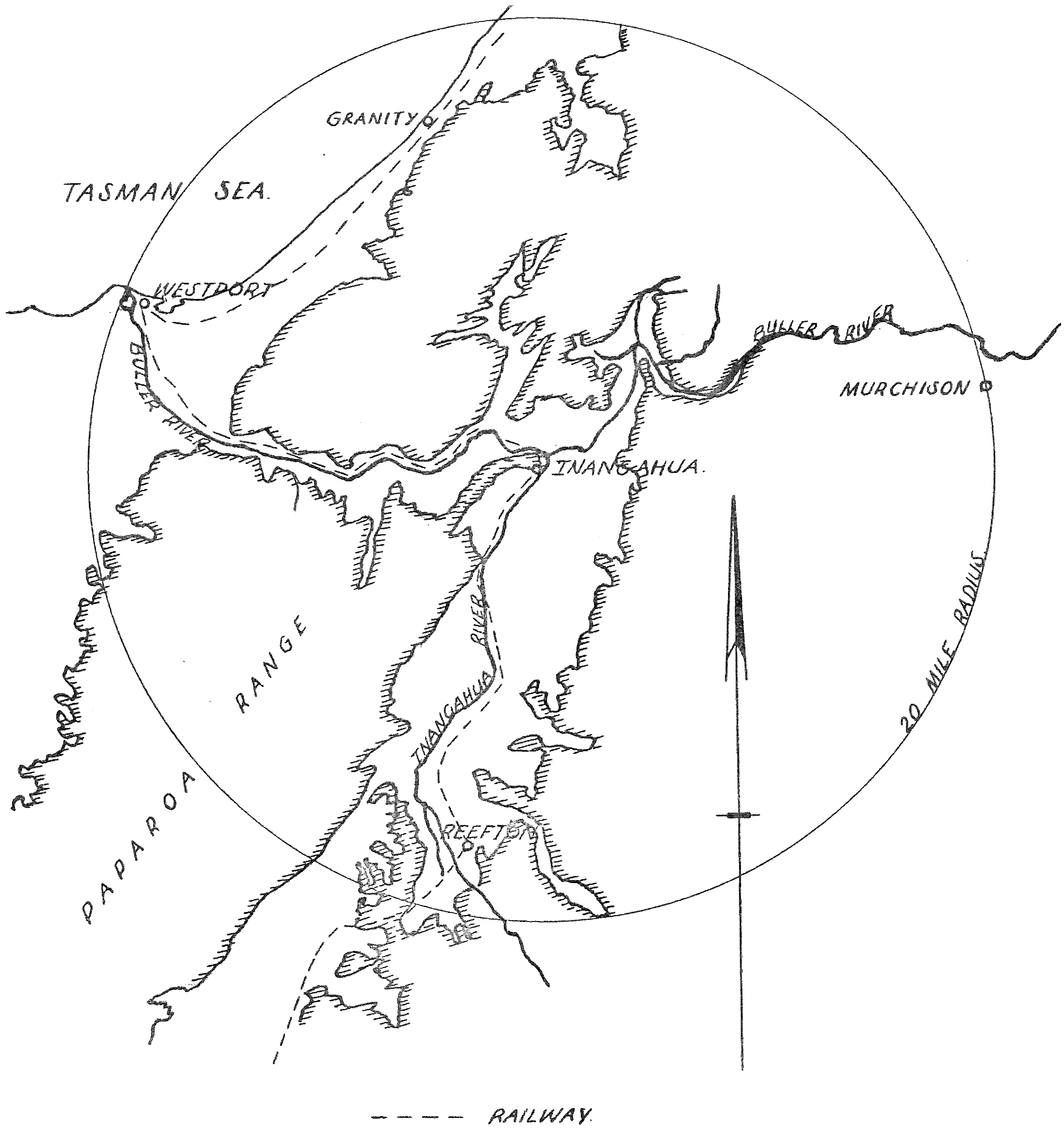


LOCALITY PLAN

FIG 1.

2. Part A—Sidlings and cuttings behaviour R

2.(1) Equilibrium before the earthquake

The question of providing a rail in an area free from natural disturbances depends eventually on economic reality. In this case the decision was to locate the facility in a narrow gorge. Grade requirements ruled out the prospect of going over the top, tunnelling costs prohibited the choice of going through the hills - instead a compromise was adopted. Sharp curves were constructed on sidlings carved out of the hills and 6 tunnels of total length of 2445 feet were provided.

In most cases the natural slopes were steep; the construction batters merely accentuated the steepness. Some of these batters were almost vertical depending on the nature of the country. Benching was not done - the sheer magnitude of the earthworks probably being the deciding factor.

Also aggravating these conditions was the high rainfall factor - 164 inches a year. This adding to the steep runoff caused leaching away of spoil supporting loose boulders up to 10 tons in size hundreds of feet above the line.

This delicate equilibrium which existed before the earthquake was upset by the rapid acceleration forces at the time of the quake and the marginally stable components fell on to the rail below.

The slips and rockslides encountered were typified by the following:

2.(2) Hawks crag breccia (73^{m}peg)

The material in this area was largely a massive hard conglomeratic rock composed of granite and greywacke set in a matrix of sand and silt. The boulder size was up to 50 tons. The site was an ancient slip and the earthquake merely reactivated it. The batter slopes were approximately 1 to 1.

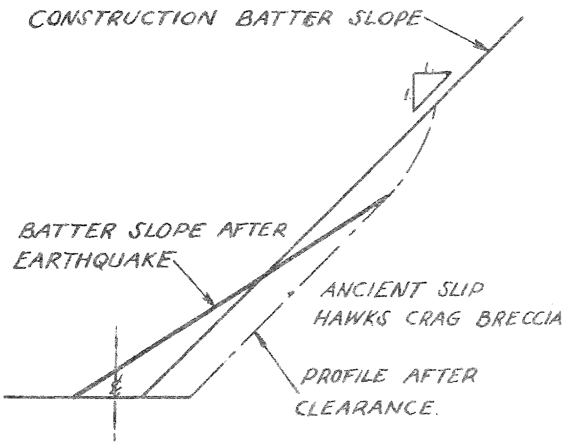


FIG. 2.
SLIP AT 73M.

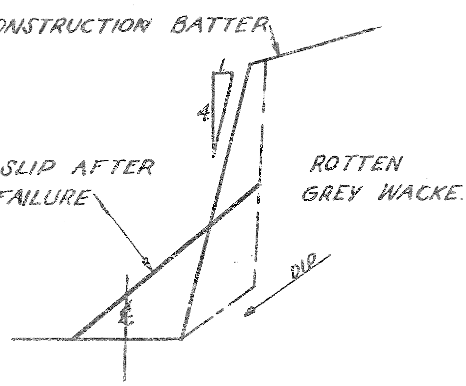


FIG. 3.
SLIP AT 67M 40ch.

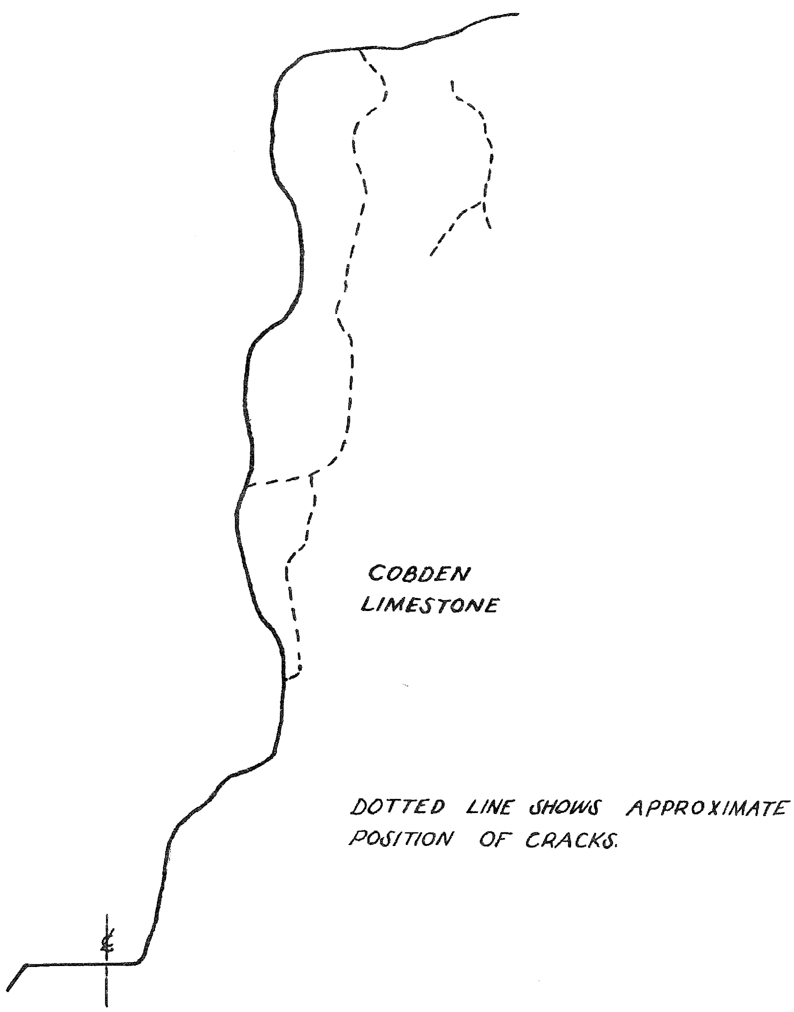
Clearing and stabilization of this slip (Fig. 2) presented a problem. Removal of the spoil from the toe merely triggered off further slips, and gradually the exposed face got wider and higher. The solution eventually was to crib log the toe.

2.(3) Rock slides (67^{m}40^c)

In a number of instances the sidlings were composed of large shattered rock mainly of mudstone origin. The batters were steep, some nearly vertical. The earthquake merely destroyed (Fig. 3) the bonding between the rock and slides developed.

2.(4) Limestone bluffs (Whitecliffs)—60^{m}58^c

Prior to the earthquake this bluff (Fig. 4) had almost a vertical face. It stood 275 feet high and the track was within 15 feet of the base. The earthquake caused deep seated vertical cracks in the face. Besides this permanent damage, large sections of the face in block form 100 tons or larger in size were dislodged onto the ground below.



CROSS SECTION OF LIMESTONE BLUFFS

FIG. 4

2.(5) Muddy sandstone sidlings (Oweka)—56^m10^c

At this locality the line was at the base of a hill and Provincial Highway No. 69 was halfway up. In this instance the top layer of the muddy sandstone had become fragmented on a low angle bedding plane dipping towards the road. Slip material including boulders 20 tons or more covered the road or fell onto the rail below.

The stabilization action was to provide a wide bench above the road. The rail was subsequently pulled away from the hill.

Not far from the Oweka Bluff is an area known as Hard Creek, which was composed of muddy sandstone materials. The action of the earthquake was to start a slide on a 5^o bedding plane. The material in block form up to 500 tons in size moved forward across a swamp. Some blocks nearly 10 tons in size moved 5 chains from the base of the hill.

2.(6) Siltstone and mudstone cutting (landing)—52^m40^c

This cutting was formed out of weathered siltstone and mudstone overbedding a sandstone layer. Prior to the earthquake the cutting had batters of 1 to 2.

The earthquake caused deep seated cracks above the crest of the cutting and the material fell into the cutting. The stabilization action has been to provide a wide bench on top of the sandstone layer.

2.(7) Lessons for the future

To overcome or at least to minimise the effects of material falling on the track depends ultimately on the cost-benefit relationship. In this instance the cost-benefit ratio would have precluded a comprehensive programme of batter flattening, benching and removal of semi-loose boulders high up the hillside.

There were however two aspects which contributed to the general instability of the area and which could have been dealt with without adding greatly to the construction and maintenance costs.

Firstly this concerned the care given to batters. In a number of instances the batters were left with trees or scrub growing very close to the edge. And prior to the earthquake some were even growing over the edge. These trees eventually fell onto the line below.

The suggestion is that if the trees and large growth for a distance of say 10' above the top of the batter were removed at the time of construction and kept down as part of normal maintenance the common place slumping of the top of the batters may not have been so pronounced.

Batter care could have been further improved if the cut off drains had been given more attention.

Secondly this concerned the steepness of some of the batters. Some of these batters were left with steep slopes when engineeringly flatter slopes were possible without significantly adding to the construction cost. Possibly a stability analysis beforehand may have revealed the need for flatter slopes.

3. Part B—Fill behaviour

3.(1) Failure pattern

Within a 20 mile radius of Inangahua most fillings experienced settlement. By reference to fixed structures such as bridge ends it was established that settlements up to 60" had taken place. The general settlement was between 12" - 24". But even those fills which suffered minor settlements of from 3" to 6" had to be built up, especially at the bridge ends, before trains could be operated at reduced speeds.

The characteristic failure pattern was a downward movement to the lowest unconfined level. In some cases the failure was of the slip circle type (Fig. 5).

3.(2) Engineering factors

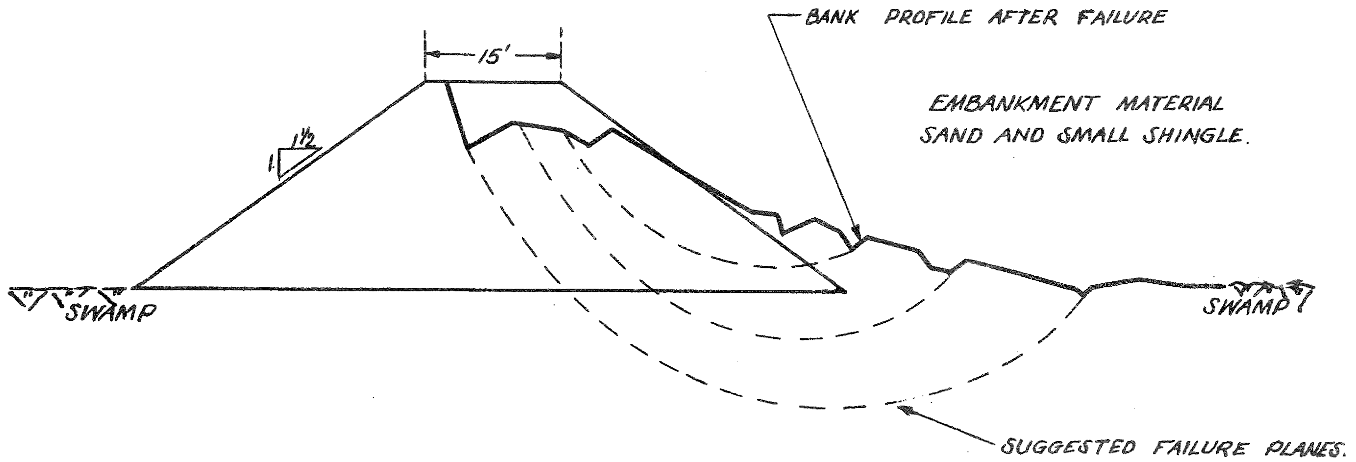
The amount of settlement did not depend wholly on the seismic factors - engineering considerations also effected the stability of the fills.

Firstly the nature of the country on which the fills were formed played an important part in fill stability. Firm ground containing river gravels offered greater resistance to downwards movements and therefore settlement of material tended to be less significant. On the other hand the less confined the country e.g. swamps, the greater was the amount of settlement.

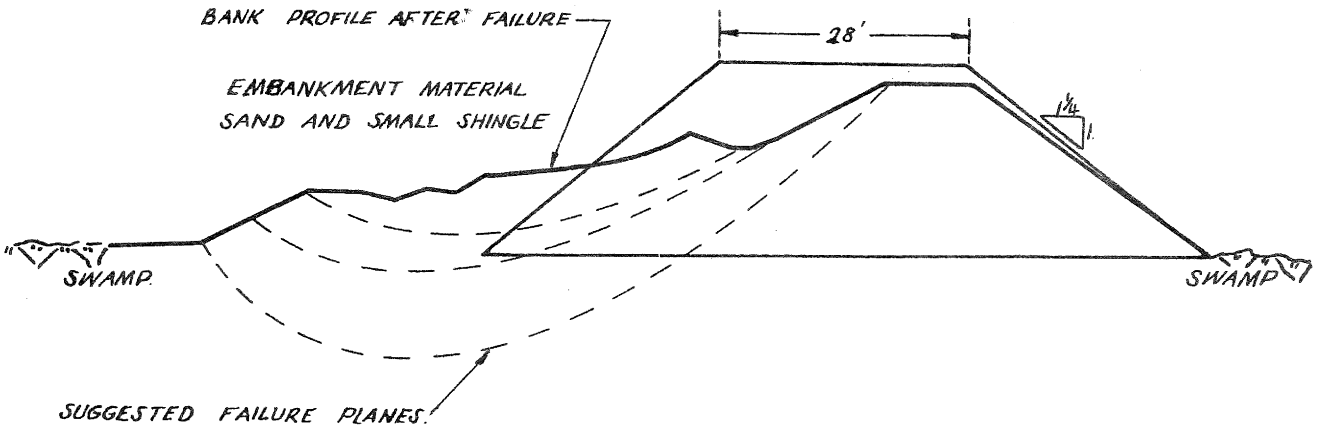
Secondly the type of fill material had a bearing on the fill behaviour. Clay type fills tended to be more unstable than the sandy shingly fills.

Thirdly the method of compaction played an important part. In this case the embankments did not receive modern compaction techniques because they were constructed before their advent.

In the main the banks were formed by the "end tipping" method and any compaction was limited to the weight of fill itself, plus the live loads that followed later. Besides the end tipping at the time of construction "side tipping" was used for bank widening at the time of and subsequent to construction. This unconsolidated feature of the fills encouraged slumping especially of the sides.



64 MILE PEG



64 m. 10ch.

SLIP CIRCLE EMBANKMENT FAILURES NEAR RAHUI.

FIG 5.

Fourthly and allied to the compaction standard, was the amount of water content in the fill. The higher the water content, the greater the susceptibility to settlement. This was borne out by the failure of a waterlogged embankment 70 miles from the epicentre, when closer embankments stood up reasonably well.

A further possible factor was the slope of the original ground surface before the filling was placed. Some differential settlement was noted in side fills and this may be attributed to this assumption. However equally valid is the view that the differential settlements were caused by the different fill heights on side fills.

3.(3) Lessons for the future

The Inangahua earthquake showed up the weakness in the old construction methods. The choice of fill materials and the lack of attention given to bedding conditions were also brought into focus. Present standards can therefore be evaluated in the light of this experience.

With bank widening becoming a regular feature these considerations are also apposite. This is because some old banks are being widened still using old methods and there exists the possibility that these are vulnerable to earthquake forces.

It is a common method of bank widening to use the side tip system. Often, because of the need to keep the utility open during the widening programme, benching into the old bank and compaction of the new fill is not done. The widened bank therefore ends up with a batter dependant on the repose angle of the fill rather than the desirable batter slope - furthermore it is not compacted. A bank of this type is therefore vulnerable.

It is suggested that if it is not economically possible to widen using approved means, then a different standard of cross section for the bank should be adopted. A greater width at the top should be adopted to compensate for the steeper uncompacted fill material.

One other suggestion in connection with bank widening is the removal of vegetation from the original batters. Frequently this is not done. Eventually this vegetation decays and in time probably acts as a lubricant for the new fill. To avoid this happening it is desirable to clean off the old face completely or at the very last burn off the growth before new filling is tipped.

Mention has been made about the settlement at bridge ends. It seems that this is one area of an embankment where settlement should be guarded against at all possible cost. In the case of railways there is the difficulty in rebuilding quickly the formation at bridge ends and in the case of roads to remove the danger created by an exposed bridge end.

(Note: A motorist died after collision with a bridge end 70 miles from Inangahua.)

4. Part C—Culvert behaviour

4.(1) Damage characteristics

The culverts in this area were concrete pipes, arches and box culverts. Damage to them was not widespread. Of the 72 culverts with openings greater than 6 square feet between 45m. 53c. and 71m. 79c. (Inangahua 56m. 54c.) only 8 suffered damage to the extent that repairs were necessary.

The damage was as follows:

1. Settlement of the culverts.
2. Opening up of joints in pipe culverts.
3. Vertical and horizontal cracking.
4. Cracking of head walls.

5. Separation of apron from culvert.

In no case was the damage such that the culverts failed to operate.

4.(2) Engineering factors

The reasons why only some culverts were damaged and the majority were not, was dependant on the following:

1. Bedding conditions.
2. Culvert design.
3. Type of fill above the culverts.
4. The amount of compaction given to the backfill.

4.(2)1 Bedding conditions

Culverts laid in firm dry natural ground stood up best. Those that were laid in soft and wet ground displayed the tendency to settle. In some instances the settlement was uneven - the culverts bowed in the middle. The largest recorded uniform settlement was 9" (Fig. 6).

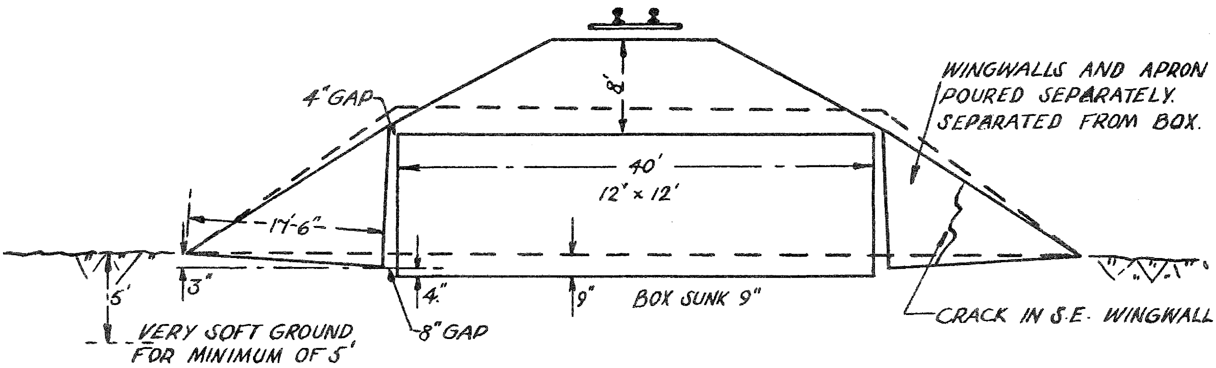


FIG 6: CULVERT SETTLEMENT
BRIDGE 88 57m. 27ch.

4.(2)2 Culvert design

There was no clear indication to suggest that the properly designed concrete pipe system incorporating reinforced concrete bedding and haunching was in any way superior to the arch or box system. This was despite the fact that none of the 8 concrete pipe culverts had been damaged, whereas the other types had. The weakest system of the lot was the concrete pipe culvert which did not have proper bedding or haunching. In one instance the culvert bowed 6" in the middle and the joints opened up from 1" to 3".

In only a few instances were the culverts provided with head and wing walls. These were of the mass concrete type with little reinforcing. These tended to crack. The damage however was not as severe as occurred in the case of highway culverts (S.H.6).

The junction between the culvert and the exit aprons appeared to be another area of weakness. In a few cases separation of the aprons from the rest of the culvert took place.

4.(2)3 Type of fill above the culvert

Damage to the culverts occurred when the fill above subsided by large amounts. The fills which were more susceptible to downwards movement e.g. the clay-mudstone type tended to cause greater damage to the culverts.

It is considered that a likely explanation for the failure of the headwalls was the outwards movement from the fill above. In the case of culverts on S.H.6 the headwall damage was even more pronounced. In these cases the fills were the clay-mudstone type.

4.(2)4 Compaction of backfill

It has already been mentioned that the amount of compaction a fill received determined it's stability. The same also applies to the culverts. The culverts nearer the surface sustained more damage than those constructed at greater depth. It is possible that the seismic forces were greater at the surface thereby accounting for the damage but just as likely is the view that the culverts at the lower levels were more firmly held in place by the weight of fill above. If this were so, with shallower culverts greater emphasis needs to be placed on compaction.

4.(3) Lessons for the future

The pronounced failure of pipe culverts which had no concrete bedding or haunching is an aspect which cannot be ignored. Likewise the attention given to bedding conditions and choice of back fill needs better consideration in the future. Also the question of head wall and wing wall design is one which needs re-examination.

The aim is for the culvert to act as a unit. It should not pull apart nor should it settle. If settlement is to take place it should be confined to uniform settlement and not the differential type.

Conclusions

The overriding conclusion is that the design standard and construction practices of the past are inadequate in the face of an earthquake with a magnitude of 7 on the Richter scale. It is even probable that some present methods are also suspect. The Inangahua earthquake has pointed out some of the deficiencies; the more obvious ones have been mentioned here, but it is abundantly clear that the information is not complete. More observations of earthquake behaviour need to be made. This should be done before the restoration of a utility has begun. Furthermore engineering personel from outside should enter the area to make these observations, thereby making it unnecessary to rely on those whose prime aim is to repair the damage.

This earthquake besides causing considerable damage to public utilities necessitating large outlays of public money before they were in service again also denied the citizens the vital means of communication. The social and economic life of the community therefore suffered. It therefore behoves the engineering profession to make sure that these setbacks are not repeated.

Most of the improvements are likely to lead to an increase in capital cost. Some may involve slight increase in maintenance costs. But if these choices are not consciously considered the lessons from the Inangahua Earthquake would have been wasted.

A LETTER TO THE EDITOR OCTOBER 1968

Re: Mr A.L. Andrews Article in Bulletin of July, 1968

I was interested in this article because:-

(a) It had direct reference to Fleming's portal and cantilever methods.

(b) The paper provided a good instance of the difficulties arising from assumptions and approximations in aseismic design.

As regards Fleming and his simplified methods; he developed them about 50 years ago. This was not long after the monumental studies of building frames by Professor Wilson of Illinois. These covered both gravity and lateral loading and were on a slope - deflection basis. Fleming was acelebrated engineer of his day whose work in practice was highly regarded. His views were often sought and his recommendations quoted by reliable authorities. Then, about 40 years ago, came the notable studies of deflected structures by Professor Hardy Cross, whose work particularly as to mement and shear distribution has been applied and extended for more than a generation. Yet in spite of all this no practicable analytical methods seem to be available to give anything more than a very dubious estimate of deflections due to seismic effects on multi-storey buildings. There are far too many imponderables, both within the structure and externally. But we have to try to cope with these as best we can - hence many approximations and assumptions. It is here that the paper is of particular interest; it demonstrates, by one fundamental instance, the need for continuing appraisal of the effects of any initial assumption or approximations.

As to structural approximations in general I have found "Statically Indeterminate Structures" by Benjamin of Stanford (1959) particularly good. Although I do not care for his treatment of shear walls that have openings in them, the letterpress of the book as a whole shows a firm grasp of the relationships between theory and reality.

Reverting once more to estimating deflections in buildings - I think that for anything in which we could have much confidence we are forced right back to more full scale local test data. In other words we need sufficient instrumental data from modern local structures plus the relevant seismological records. Such date can't be got or interpreted easily or cheaply but they are urgently needed. There is no worthwhile substitute: the complexities of transient response and damping, apart from many other factors, determine this. Who can or will, or whose business is it, to get and handle the information?

P.S. Structural Engineers Association of California a few years back made some suggestions about allowances for storey drift floor to floor and when to allow for same. But in the end recommendations were left merely as guidance and nothing, unless quite recently, has been made mandatory.

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AUCKLAND.