Ferry Buildings

Report on Proposed Structural Strengthening for Earthquake Resistance

Gurley & Nicolls

MACDONALD BARNETT PARTNERS

REGISTERED PROFESSIONAL ENGINEERS
CONSULTING CIVIL & STRUCTURAL ENGINEERS
A. J. MACDONALD, B.E., F.N.Z.I.E., M.I.C.E.
C. R. BARNETT, B.E., F.N.Z.I.E., M.I.C.E.
M. B. SPICER, B.E., M.N.Z.I.E.

P.O. BOX 37-077
429 PARNELL ROAD,
PARNELL, 1,
AUCKLAND, N.Z.
TELEPHONE 795-670
TELEGRAMS: "MACBAR"

Wargon Chapman & Gurley,
Registered Engineers,
P.O. Box 47-215,
AUCKLAND

YOUR REFERENCE:

OUR REFERENCE: 2989 MBS
3 Sep. 81

Dear Sirs,

RE: THE AUCKLAND HARBOUR BOARD FERRY BUILDING
FIRE RESISTANT CONSTRUCTION AND EGRESS ASPECTS

1. On 21st August you visited our office to discuss the redevelopment work that is being considered for the Auckland Harbour Board Ferry Building. Subsequently you requested that we advise you on "Fire Resistant Construction and Egress" aspects of the existing building, also to report on these aspects in relation to the possible redevelopment alternatives that you have been considering. Our Mr. Barnett and Spicer visited the site in company with the chief technical officer of the Auckland Fire Brigade. The purpose of this meeting was to appraise the current situation and to consult with the fire brigade controlling authority on various redevelopment proposals that were being considered. A verbal undertaking was obtained from them as to what would be required in terms of fire and egress upgrading to ensure compliance with NZS 1900 Chapter 5 and the fire brigade authority.

2. The building is located in the central fire risk area. This implies that it would need to meet the conditions of a "Type 2 construction" to fully comply with the requirements of Chapter 5. The floor plan area is approximately 935 m^2 (10,065 ft^2) and the base building comprises four storeys, above which rises the clock tower superstructure.

3. Commercial and industrial buildings such as office buildings, showrooms, shops for non-combustible and non-explosive materials are classified in the low risk division Group D1. A maximum floor plan area 1859 m^2 (20,000 ft^2) with unlimited storeys is allowed for this classification. General shops, restaurants, sale rooms, department stores, market buildings and workshops and workrooms manufacturing or storing materials for semi-hazardous processes are classified in the moderate risk division Group D2. In this classification a maximum floor area of 1394 m^2 (15,000 ft^2) with unlimited numbers of storeys is allowed.

Contd...over

MACDONALD BARNETT PARTNERS

REGISTERED PROFESSIONAL ENGINEERS
CONSULTING CIVIL & STRUCTURAL ENGINEERS
A. J. MACDONALD, B.E., F.N.Z.I.E., M.I.C.E.
C. R. BARNETT, B.E., F.N.Z.I.E., M.I.C.E.
M. B. SPICER, B.E., M.N.Z.I.E.

P.O. BOX 37-077
429 PARNELL ROAD,
PARNELL, 1,
AUCKLAND, N.Z.
TELEPHONE 795-670
TELEGRAMS: "MACBAR"

Wargon Chapman & Gurley,
Registered Engineers,
P.O. Box 47-215,
AUCKLAND

YOUR REFERENCE: .

OUR REFERENCE: 2989 MBS
3 Sep. 81

Dear Sirs,

RE: THE AUCKLAND HARBOUR BOARD FERRY BUILDING
FIRE RESISTANT CONSTRUCTION AND EGRESS ASPECTS

1. On 21st August you visited our office to discuss the redevelopment work that is being considered for the Auckland Harbour Board Ferry Building. Subsequently you requested that we advise you on "Fire Resistant Construction and Egress" aspects of the existing building, also to report on these aspects in relation to the possible redevelopment alternatives that you have been considering. Our Mr. Barnett and Spicer visited the site in company with the chief technical officer of the Auckland Fire Brigade. The purpose of this meeting was to appraise the current situation and to consult with the fire brigade controlling authority on various redevelopment proposals that were being considered. A verbal undertaking was obtained from them as to what would be required in terms of fire and egress upgrading to ensure compliance with NZS 1900 Chapter 5 and the fire brigade authority.

2. The building is located in the central fire risk area. This implies that it would need to meet the conditions of a "Type 2 construction" to fully comply with the requirements of Chapter 5. The floor plan area is approximately 935 m^2 (10,065 ft^2) and the base building comprises four storeys, above which rises the clock tower superstructure.

3. Commercial and industrial buildings such as office buildings, showrooms, shops for non-combustible and non-explosive materials are classified in the low risk division Group D1. A maximum floor plan area 1859 m^2 (20,000 ft^2) with unlimited storeys is allowed for this classification. General shops, restaurants, sale rooms, department stores, market buildings and workshops and workrooms manufacturing or storing materials for semi-hazardous processes are classified in the moderate risk division Group D2. In this classification a maximum floor area of 1394 m^2 (15,000 ft^2) with unlimited numbers of storeys is allowed.

Contd...over

4. We understand that all of the main floors of the building consist of a 150 mm (6") reinforced concrete slab, which is overlaid in most instances by timber joists and decking to form the walking surfaces. A 150 mm thick concrete floor has a 3 hour fire resistance rating, assuming a minimum cover of 25 mm to the reinforcing steel. The maximum fire rating of a floor in a building classified as low risk Group D1 is 1½ hours and in a moderate risk Group D2 building is 2 hours. This 2 hour rating, however, may be reduced to 1½ hours where all walls, excepting fire walls are more than 15 feet from the legal boundary. We are not sure whether this situation applies in this instance, but nevertheless, the existing floors appear to have a healthy reserve of rating over and above the minimum requirement, notwithstanding the possibility that the cover to the reinforcing steel may in some instances be somewhat less than the minimum 25 mm requirement. No doubt there will be penetrations through the floors to accommodate the passage of services, ducts, etc. Some of these openings will need to be attended to, by way of provision of fire dampers for ducts, etc. and to ensure that only metal pipework or other suitable fire resistant materials pass through the floors to prevent fire bridging.

5. The main vertical supporting members primarily consist of brick and block masonry walls, together with masonry columns and other ornamentation. These members are all of massive section and of themselves quite readily provide more than the required minimum fire rating of 2 hours.

6. The main horizontal spanning elements other than the floors comprise massive steel beams which in the most instances are plaster or concrete encased. It has not been possible to ascertain the overall thickness of the encasing, but due to the general massive nature of the construction, one can assume that in general the horizontal elements will meet the 2 hour fire rating that is required for code compliance, although there may be odd isolated situations that will need upgrading.

7. The external perimeter walls have a relatively small proportion of openings in them and as the separation distances between other existing adjacent buildings are generous, the facade structure, infill panels, and window openings readily comply with the fire resistance ratings laid down in the code.

8. In summary thus, the floor areas and basic supporting fabric of the existing building comply with the current code ordinances. This being the case, any anticipated redevelopment automatically meets the main fundamental requirements for the fire risk area.

9. The existing building is currently occupied by a variety of tenants. Normally a 1 hour fire rated partition is required to separate adjacent tenancies and to separate any tenancy space and the routes and means of egress. It is noted that some of the existing corridors have plain glazing at high level, which is unacceptable and will need to be replaced by Georgian Wired Glass. Alternatively, the existing partitions could be extended in fire rated construction up to the underside of the floor above. It is also probable that a large proportion of the inter-tenancy partitions do not meet the minimum 1 hour rating and will need to be upgraded by the addition of further layers of Gibralter Board to ensure a full compliance. All doors giving access from individual tenancies in to the corridors would also need to be checked to ensure compliance with smoke stop standards.

Contd...over

10. In most instances there were no suspended ceilings in the building and thus no created ceiling spaces. However, in the roof space, ½ hour fire stops would need to be installed to provide areas less than 180 m^2, and with each stop no further apart than 15 m in any one direction. This could easily be effected by installing conventional 100 x 50 stud walls lined on each side with Gibralter Board. Alternatively, the third floor ceiling could be removed to expose the existing timber roof structure. This alternative has considerable merit for certain types of development such as restaurant or museum facilities.

11. The existing means of egress and widths of exit ways from the building are substandard in several aspects. The primary egress is by way of the corridors on each floor leading into the smoke protected lobbies which form part of the main central staircase. It was noted that the smoke stop doors, which separate the central stair lobby from the corridors either side, need to be upgraded to minimise the gap between the door itself and the enclosing frame. It is noted that these doors are currently swinging in both directions and upgrading could readily be effected by incorporating a planted timber stop restricting the door to one way opening in the direction of exit travel. The stairs themselves appear to conform in geometry and form of construction.

12. Primary egress is augumented on the west end of the building by a secondary egress stair which also complies as to geometry and form of construction, excepting that the bottom flight of the stair incorporates a small winder which is not allowed. This would need to be removed by installing a landing at the head of the winder and then returning the stair back down towards the western end. These alterations are of a minor nature, however, and would readily be incorporated into any general upgrading of the building. At the eastern end of the building, a strictly non-compling "ships ladder" form of egress exists and this would need to be upgraded generally in a manner to match the west end of the building. This upgrading is necessary so as to avoid forming what is known as the "cul de sac" situation, where the distance of travel from the eastern end of the building to the central stair exceeds the allowable. Furthermore, the maximum allowable area of a "cul de sac" situation in Type 2 construction Group D1 or D2 occupancies is only 2500 ft^2, whereas the existing cul de sac area of the eastern end of the building approximates to 4,000 ft^2.

13. It was noted that in some instances the secondary egress stairs could only be reached by passing through a tenancy. In these cases, tenancy partitions would need to be repositioned to exclude the exit-way itself.

14. Turning now to the capacity of exit ways, assuming that the eastern stair-way is upgraded to match the west, the widths of the stairways and corresponding units of egress are as follows:-

		Width	Units of Egress
	Central Stair 1.45 m	57"	2.83 = 3
	Western Stair 0.9	36"	1.67 = 1.5
(Upgraded)	Eastern Stair 0.9	36"	1.67 = 1.5
			6.0.

We understand that some thickening of the western and eastern stair walls may be be required for structural upgrading. As the stair widths are slightly generous in terms of egress this additional thickening will not reduce the theoretical capacities.

Contd...over

15. The Fire Code allows 60 people to be accommodated on each floor for every unit of egress. This gives a maximum theoretical floor population of 360. Assuming a nett area of approximately (9,000 ft^2), the maximum allowable floor density would be 25 per sq.ft. This density would enable the following types of occupation to be adopted without additional egress being required.

Factory manufacturing
Workrooms such as for clothing manufacture
Offices
Shops
Schools.

16. Should however, it be desired to convert the upper storey into a restaurant, then some additional width of egress stair or another stair might be required. Without this additional egress the maximum nett usable floor area would be 7,200 ft^2. In certain situations, however, dependant on catering, the whole of the upper storey could be converted into a restaurant since a considerable amount of space would be required for catering services, etc. and this space is not brought into the calculations. On the other hand, if a night club, cabaret or dance hall type of activity should be envisaged, then the Liquor Licensing Act has quite restrictive egress requirements and depending on each situation, a maximum nett usable space of between 3,000 ft^2 and 4,320 ft could be expected. Once again the facility space would be additional to this area.

17. A further possibility for development could be the formation of a museum type of facility. Chapter 5 does not consider this usage and thus egress requirements would be subject to negotiation and discussion with the controlling authorities. However, if the Auckland Art Gallery situation is taken as a guide, then it appears that the existing egress would be satisfactory to serve one floor converted to museum facilities.

18. As an alterantive and indeed, completely different approach to upgrading the east end egress stair, there may be some merit in considering two new stair towers situated at roughly quarter points along the length of the building. Situated in this location the cul de sac maximum areas and travel distances could be complied with and the location would also be favourable in providing some of the additional earthquake resistance needed for the building. Dependant on the building usage, it could be desirable to incorporate new lift facilities at these locations. If this approach was adopted and depending on the general arrangements of tenancies, it might be possible to do away with a considerable amount of corridor space at each end of the building, close off the existing stairs and lift shaft, so as to establish a greater usable floor area and thus higher rental return.

19. In conclusion, it is evident that there are no major problems to be overcome in upgrading the building to meet acceptable fire and egress standards. Whichever approach is adopted will to a greater extent depend on the type of occupancies that are adopted and the overall economics of the total refurbishment and structural upgrading.

Contd...over

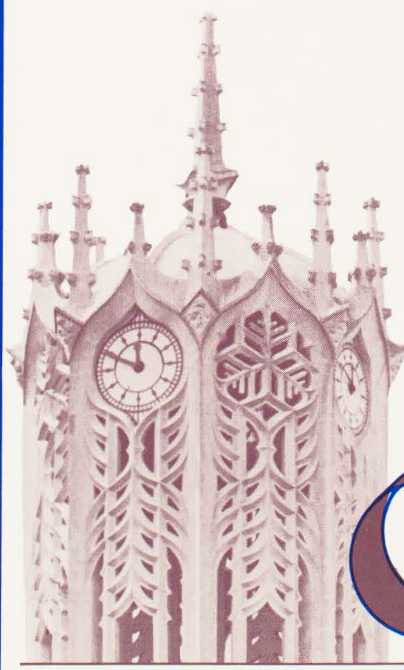
20. We trust that this is satisfactory and assists you in your consideration of the ferry building.

Yours faithfully,
MACDONALD BARNETT PARTNERS

c.c. File 2989

AUCKLAND FERRY BUILDING

ASSESSMENT OF SEISMIC WALL PRESSURES



Applied Research Office

The University of Auckland

The University of Auckland, Private Bag, Auckland. Telephone 792-300 Ext. 328 or 330.

AUCKLAND FERRY BUILDING

ASSESSMENT OF SEISMIC WALL PRESSURES

Report prepared by: Dr T.M. Larkin,
Lecturer,
Civil Engineering Department,
University of Auckland,
Private Bag,
AUCKLAND

Report prepared for: Wargon Chapman & Gurley,
Consulting Engineers,
AUCKLAND

APPLIED RESEARCH OFFICE
THE UNIVERSITY OF AUCKLAND

November 1981

AUCKLAND FERRY BUILDING

Assessment of Seismic Wall Pressures

1. Introduction

This report presents the results of a study of the seismically induced soil pressures on the sea wall of the Auckland Ferry Building. The aim was to evaluate the seismic loading using a relatively simple yet reasonable method of analysis. More sophisticated methods, such as finite element solutions, are possible but with the present knowledge of the site there is little advantage in this technique.

The sea wall of the Ferry Building is 8.5 metres high and 4 metres thick at its base. It supports one side of the Ferry Building (the other side being piled) and rests on the Waitemata series bedrock. The area is one of reclaimed land, and as such the wall retains loose normally consolidated soils which are heterogeneous in nature with some organic matter and debris.

There is very little recorded data on seismically induced wall pressures. In many circumstances seismic wall pressures are not considered to pose a major threat to the structural integrity of walls or buildings. There are few situations where earthquakes have severely damaged walls. This has led engineers to regard the seismic design of walls as an unimportant topic. The available data and analyses support this conclusion. However possible exceptions are walls in soft soil or granular media where significant strength loss (liquefaction) may result from oscillatory earthquake loading.

The site under consideration should be regarded as a site where seismic liquefaction has a relatively high probability. The area is one of heterogeneous filling but containing significant quantities of very loose silty sands with S.P.T. values of approximately 5.

This study was initiated to evaluate the seismic loading on the wall in the event of both significant soil strength loss during shaking and also in the event of the ground maintaining its integrity throughout the seismic event. The latter is by far the more difficult task to accomplish and the analyses performed and presented here were carried out to answer this question.

The data needed for the analyses was evaluated from the information contained in references 1 and 2.

The seismic design level adopted in the study was a peak bedrock acceleration of 0.15 g. This value was used for consistency with other work in references 1 and 2. This level of seismic activity represents approximately a 65% probability of being exceeded in 100 years.

2. Method of Analysis

The method of analysis was based on calculation of free field seismic deformations and the requirement of satisfying the boundary conditions relating to the soil/wall interface (reference 3). No relative deformations may occur between the wall and the soil during a seismic event.

The soil medium was treated as a very loose fine sand. An estimate of the dynamic properties of shear modulus and viscous damping factor was obtained by published results of dynamic soil tests. A one dimensional seismic response analysis was carried out to evaluate the magnitude of the free field shear stresses and strains. This analysis employed the recorded motions of the Eureka 1954 earthquake scaled to produce a peak acceleration of 0.15 g.

Dynamic wall pressures involve the mobilisation of a degree of the passive resistance of the soil. The loads on the wall may be found by relating the free field deformations to the deformations required to produce full passive wedge action.

The limited blow count data available for the soils (reference 1) indicate a relative density, of D_r, of approximately 30%. This value of density may be used to evaluate the friction angle of the silty sand, ϕ', as

$$\phi' = 25 + 0.15\, D_r = 32^o$$

In this case the flow value, N_ϕ, used to compute passive pressures is N_ϕ

$$N_\phi = 3.26$$

and a value of the lateral stress coefficient , K_o, was choosen as 0.5. Thus the passive pressure at any depth is 3.26 times the effective overburden pressure.

It is also necessary to estimate the deformations required to mobilise full passive pressure. A survey of the literature for loose sands (reference 4) reveals that the displacement required for passive pressure, δ_c, varies between 0.07 to 0.43 of the wall height for loose sands. In this study a value of 0.05 of the wall height was used as being a reasonable yet conservative value. Higher values of δ_c lead to lower wall pressures. The degree of dynamic passive pressure mobilised is calculated by using the computed free field displacement, δ, in conjunction with δ_c.

$$\text{Dynamic Pressure} = \frac{\delta}{\delta_c}\,(N_\phi\ \sigma_v' - K_o\ \sigma_v')$$

where σ_v' is the vertical effective stress.

These analyses were carried out assuming no strength loss occurred in the silty sand. Thus these represent the non-liquefaction design case.

The effect of dynamic pore pressure rise (leading to liquefaction) is two fold. The dynamic properties of the soil are dependent on the mean effective stress state. As the pore pressure rises the mean effective stress reduces, since the total stress remains constant, leading to a reduced value of the shear modulus. This effect increases the free field deformations. However the reduction in vertical effective stress leads to lower values of the passive pressure. These two effects have opposing influences on the dynamic pressures on the wall.

An analysis was carried out to simulate partial strength loss in the silty sands by a build up of dynamic pore pressure. The vertical effective stress was reduced to half its static value. This led to higher seismic free field displacements but when coupled with the reduction in passive pressure (at δ_c) the result was only a small increase in dynamic soil pressure.

The upper bound to the effect of dynamic pore pressure rise is liquefaction of the soil. In this case the silty sands will sustain very little shear stress and the behaviour may be approximated to that of a liquid. In this case for design, the lateral stress may be calculated using a liquid density of 1950 kg/m^3 and a lateral stress coefficient, K, of 1.

3. Results of Analyses

The computation of the free field seismic deformations was carried out using a computer program. The results of the analysis are shown in Figure 1. Both the shear strain and stress increase with depth. The maximum strain levels reached are relatively large, 0.35% near the base of the sand. This reflects the low rigidity of the sands more than the level of seismic motion. These results were obtained using strain compatible values of shear modulus and damping factor.

Using the results of Figure 1 the dynamic wall pressures were computed for the non-liquefaction case. The results for LWST are shown in Figure 2. Both the dynamic and the total (static + dynamic) soil pressure is shown. The results indicate low values of dynamic soil pressure. This result is in general agreement with other analytical studies in that these studies have also shown low values of dynamic wall pressures. The maximum dynamic pressure is approximately 6 kPa and occurs at mid height of the wall. The distribution of dynamic pressure is approximately parabolic, with zero values at the top and bottom of the wall.

An analysis was also carried out for the HWST case and the computed dynamic soil pressure is shown in Figure 3. It is seen that these values are considerably lower than the LWST values. This is due to the higher vertical effective stress under LWST conditions.

4. Conclusions and Recommendations

The work carried out establishes the magnitude of the dynamic wall pressures in the case of an unliquefied soil deposits. These pressures are small, about 6 kPa maximum. This data is in general agreement with observed wall pressures and the few analytical studies to date. The results clearly illustrate the large conservatism in using full passive pressures in the seismic design of walls.

The critical design case is clearly the soil pressures in the liquefied state. It is reasonable, and conservative in most cases, to use a lateral stress coefficient of 1 to represent the wall pressures in this case.

The appropriate design case is a HWST situation where the lateral pressure may be calculated from a heavy liquid with a mass density of 1950 kg m^{-3}.

In a very heterogeneous fill, such as this material, it is likely that liquefaction if it occurs will be limited to zones where instability and very high pore pressures will occur. Complete strength loss by the soil prevents shear waves travelling through the liquefied soil, which thus acts as a wave barrier to isolate the unliquefied material above. In the case of this fill, interaction between the wall and building will continue to create deformations in the liquefied soil. Should liquefaction develop it may well occur some time after the peak acceleration has passed. The process of pore pressure build up is not an instant phenomenon but is dependent on the dynamic stress history of the soil media, and also the capabilities of the deposit to dissipate the excess water pressure created. The pockets of coarse basalt cobbles will provide a rapid drainage path for excess water pressures to drain. This will retard the growth of dynamic pore pressures and hence decrease the probability of liquefaction.

The analyses described here are not a direct analytical assessment of the response of a zone of unliquefied material overlying a liquefied deposit. Such an analysis would be very difficult and costly to perform and would require detailed site information. More sophisticated analyses are feasible, but should be carefully considered in the light of this report. Confidence needs to be held in the applicability of the results obtained from costly analyses.

The analyses performed support the conclusion that the seismic wall pressures will be significantly less than passive. This leads to the opinion that the design wall pressures should be based upon the liquefied soil case and a lateral stress coefficient of 1. The recommended design wall pressures are shown in Figure 4.

The design case considered is a soil deposit which has liquefied below the water table (HWST) and which has a small dynamic component on the 1.5 m above the water table. The pressures below the water table are those created by a liquid of density 1950 kg/m^3.

The use of the lateral stress coefficient of 1 in conjunction with a liquid of density 1950 kg m^{-3} is quite conservative enough when viewed in the framework of the level of seismicity being contemplated.

T.J. Larkin

T.J. Larkin,
Lecturer,
Civil Engineering Department

20th November, 1981

References

1. "Preliminary Soils Investigation, Ferry Building Strengthening, 85 Quay Street, Auckland", Brickell, Moss and Partners, September 1981, Preliminary Soils Investigation.

2. Section 11 of Wargon Chapman and Gurley Report on the Auckland Ferry Building to the Auckland Harbour Board, September 1981.

3. Taylor, P.W., 1980. "Seismic Pressures from Cohesive Soils on Basement Walls", Seminar on Geotechnical Aspects of Retaining Wall Design, Construction and Performance, Centre for Continuing Education, University of Auckland.

4. Indrawan, Z., 1981. "The Seismic Earth Pressure on Basement Walls Exerted by Cohesive Soil", M.E. Thesis, University of Auckland, February 1981.

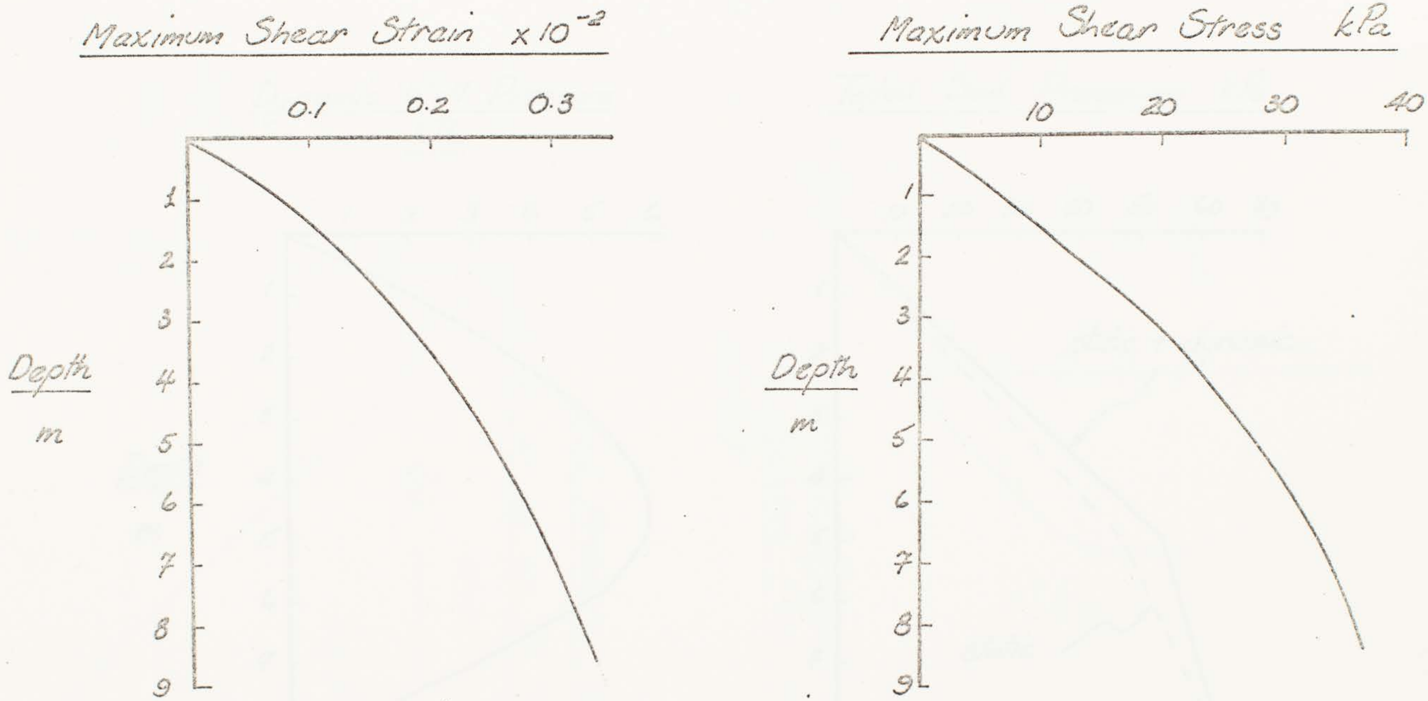


Figure 1 : Seismic Free Field Response

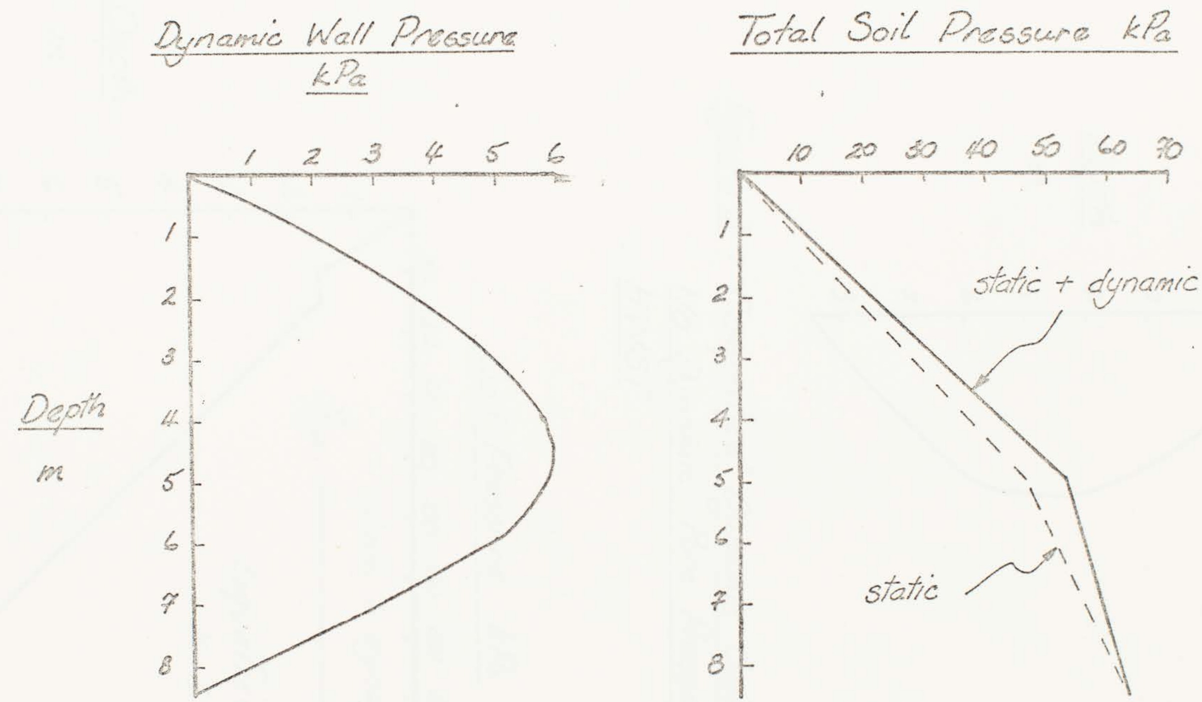


Figure 2 : Computed Wall Pressures
No Dynamic Pore Pressures
LWST

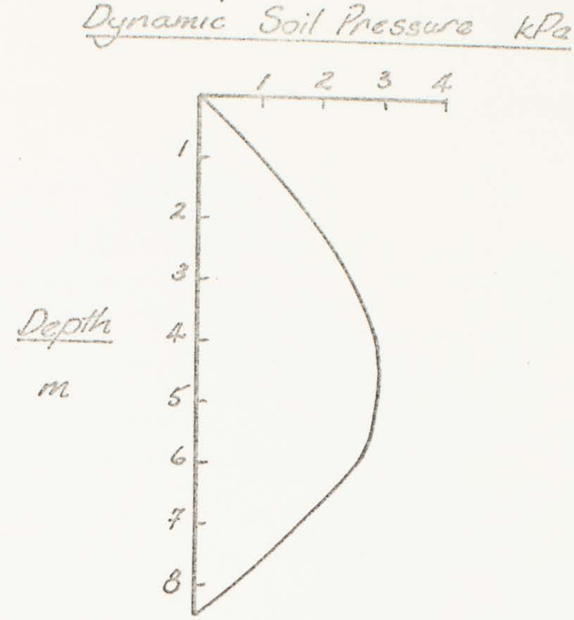


Figure 3 : Computed Dynamic Soil Pressure
No Dynamic Pore Pressure
HWST

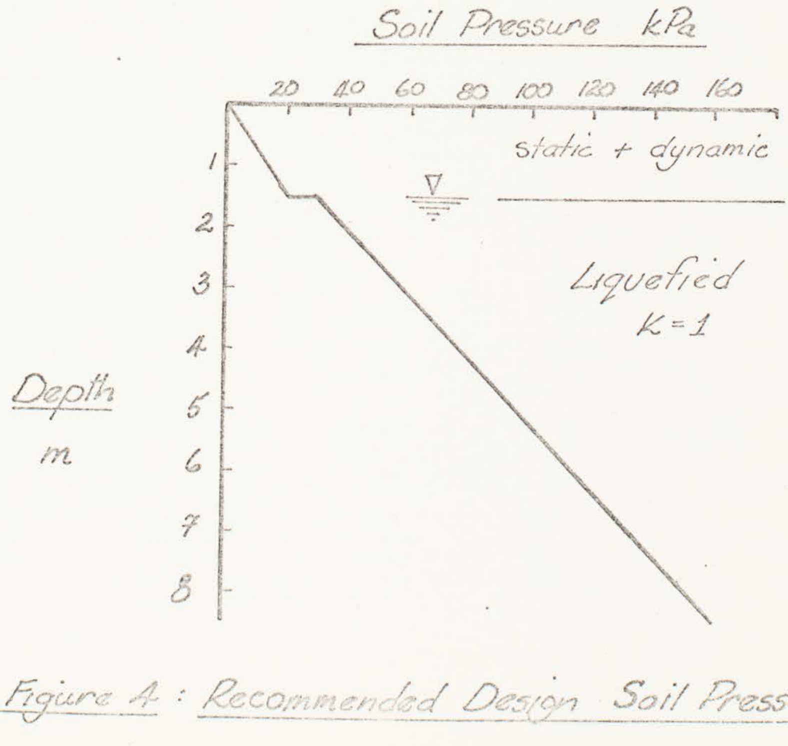


Figure 4 : Recommended Design Soil Pressure