CITY OF PORT ALBERNI

SEISMIC SURVEY OF CITY-OWNED BUILDINGS

Prepared by

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EXECUTIVE SUMMARY

The City of Port Alberni has provided funding to carry out a seismic evaluation of City owned buildings. It is expected that most of the facilities either would be required for post disaster functions, or their destruction would affect the ability of the City to operate effectively after the disaster.

The City is located in one of Canada's most severe seismic regions, that is, the probable forces are very high, approximately 50% greater than those anticipated in the City of Vancouver.

It has been determined that the majority of the structures do not meet the requirements of the 1990 National Building Code and much damage including some collapses can be expected even during a moderate earthquake. The results are not unique in that the work carried out by CWMM for other groups of buildings in Canada, as well as similar work carried out by others in the United States, has resulted in conclusions that show that most buildings are deficient, often substantially.

1.0 INTRODUCTION

The firm of Choukalos Woodburn McKenzie Maranda Ltd. (CWMM) was invited on May 7, 1990 to submit a proposal for conducting a seismic evaluation study of City-owned buildings in Port Alberni. The evaluation was to include eight major buildings and six pump stations.

Subsequently, on June 1, 1990, CWMM was advised that their proposal was successful and that they would be awarded the contract to carry out the work. Purchase order #25141 was referenced.

It was required that a preliminary assessment must be completed on the Public Works Building by the end of June and that the remainder of the work was to be completed within three months of award of contract, that is, by September 1, 1990.

2.0 PURPOSE

The purpose of the study is to carry out a seismic evaluation of fourteen City-owned buildings and to provide general recommendations regarding possible structural retrofitting if buildings did not meet Code requirements. Details of the requirements were outlined in the Request for Proposal dated May 7, 1990. The main items were:

(1) Review of construction drawings and inspection of buildings to ensure that all renovations have been addressed.

(2) Carry out a structural assessment and compare capacity with that required by the present Codes. (The proposed 1990 Code was chosen as the basis since upgrading would most likely be carried out to this Code.)

(3) Consider non-structural items.

(4) Provide general recommendations for upgrading, including options where applicable.

The following structures are to be analysed. Each is intended to have a post disaster function and as such must remain safe to life, have minimal structural damage, and be capable of functioning for a short period after an earthquake.

(1) PA1 JOHNSTON ROAD PUMPSTATION

(2) PA2 SOMASS PUMPSTATION

(3) PA3 WALLACE STREET PUMPHOUSE

(4) PA4 4TH AVENUE PUMPHOUSE

(5) PA5 BAINBRIDGE PUMPSTATION

(6) PA6 COWICHAN PUMPSTATION

(7) PA7 PUBLIC WORKS OFFICE AND MAINTENANCE CENTRE

(8) PA8 EQUIPMENT AND STORAGE BUILDING

(9) PA9 CITY HALL

(10) PA10 FIRE HALL

(11) PA11 ALBERNI HEALTH CENTRE

(12) PA12 CURLING RINK AND SKATING ARENA

(13) PA13 ECHO CENTRE

(14) PA14 PUBLIC SAFETY BUILDING

3.0 CODE CONFORMITY

Major building codes such as the National Building Code in Canada and the Uniform Building Code in the United States, have been developed for the design of new buildings and do not address the special requirement of existing structures. Methods of assessing the seismic behaviour of existing buildings usually involve comparing the period, the force distribution, the displacement, etc. to the criteria specified in the latest code. It was assumed that if the criteria was not met then the building was an unacceptable seismic hazard. Engineers and researchers are now questioning the effectiveness of upgrading to meet the precise requirements of the latest codes.

Major codes vary in their approach to seismic design with a design meeting one code being deficient when compared to that of another. A report entitled "Comparison Among Earthquake Codes", published in Earthquake Spectra (Vol. 5, No. 4, 1989) summarizes the variation amongst several of the important American codes. A basic philosophical disagreement still exists relating to the design of buildings. One side feels that performance would be adequately met by increasing the level of the specified forces, while others feel that this is inadequate and that the necessary performance would only be met by stricter design and construction requirements, since, in fact, the actual force that one designs for is between 20% and 30% of the actual force. The design for seismic performance of new buildings is still controversial.

Structures can be designed to resist major earthquakes without substantial damage. This would, however, in most situations be prohibitively expensive and unwarranted because of the extremely low probability of the code quake occurring. A small number of very important buildings are being designed in the United States to meet such requirements but they are being designed for values far in excess of any code. The normal code provisions are to reduce the probability of fatalities to an acceptably small value and to accept structural damage to the building in a major earthquake.

The philosophy associated with the upgrading of existing structures is even more controversial. Most engineers feel that rigidly upgrading to strict compliance with present codes is generally not technically possible. Often, even where it may be possible, the upgrading costs are prohibitive, suggesting demolition and replacement. Engineers note that strict compliance cannot occur without a complete inspection of all structural components and connections. For many structures this would necessitate extensive dismantling. Many of these engineers feel that such strict compliance is not essential, and that an experienced engineer, using his professional judgement in meeting the intent of the Code, i.e., to prevent loss of life, is an appropriate and satisfactory approach.

An important Code concept, that is often not realized by the general public, is that the intent of the Code is to protect lives. The building may meet the Code, yet suffer severe damage and may in fact require demolition. Buildings that are to remain functional for post disaster functions must be designed for a slightly higher force by the inclusion of an importance factor. However, this force is still substantially below the actual forces that will occur. Seismic design of new or existing structures cannot at present confer any guarantees, and the full effectiveness of rehabilitation remains somewhat uncertain since historic upgrading data is minimal. In general, however, an upgrade for the type of structures in this inventory, will substantially improve their performance and minimize risk of collapse to that approaching new structures.

4.0 MANDATORY UPGRADING

With certain exceptions, today's Codes do not require mandatory abatement.

The N.B.C. does not require that an existing building be upgraded to meet today's more stringent codes, with the exception of buildings that are to undergo alterations. This may require that the entire structure be brought up to date. There is no specific statement in the Code and it can be inferred from the lack of statement to the contrary that upgrading, where required, must be to the full levels of the latest Code.

The City of Vancouver requires seismic conformity when there is a change in occupancy, loading, or when the proposed upgrading exceeds 25% of the assessed value of the building. Vancouver tends to require that the upgrade force level approach the Code requirements for new structures.

Los Angeles has partially addressed the problem of existing buildings by including in their Code, Division 88, "Earthquake Hazard Reduction in Existing Buildings". This establishes mandatory minimum standards for existing buildings. However, it applies only to unreinforced masonry brick structures, constructed prior to 1934, a building type that has historically behaved poorly. This Code requires that owners of all such structures retain engineers to assess their buildings and then upgrade them within a specified period of time. The design requirement is to meet the force levels of between 50% and 70% of their 1980 Code. The Code limits upgrades to pre 1934 structures because after that time such construction was banned. Unreinforced masonry, which is a material that behaves similarly to unreinforced brick, and as such is a bad performer, is a major structural component of most of the Port Alberni structures listed in the inventory.

Subsequent to the Loma Prieta earthquake in California, the City of Oakland has required that buildings that have suffered 10% damage to their structural systems must be upgraded up to meet the present Codes.

Most Codes have "unsafe building provisions". It is probable that compulsory mitigation for seismic hazards can be enforced through this provision. However, there is no record of authorities using this criteria for compulsory mitigation.

5.0 UPGRADING

An important group involved in the development of evaluation procedures for older buildings is the Applied Technology Council (ATC), a non-profit group established in 1971 through the efforts of the Structural Engineering Association of California. A major emphasis of this group has been to establish procedures for evaluating the capacity of all types of existing buildings to resist seismic forces. From this work evolved several studies, culminating with ATC-14 "Evaluating the Seismic Resistance of Existing Buildings" and its sequel ATC-22 "A Handbook for Seismic Evaluation of Existing Buildings". The work was funded by the National Science Foundation.

The methodology in these publications assumes that a hazardous building endangers human lives in an earthquake if there is partial or total collapse of the building, if components fail and fall, or if exits and entry routes are blocked. The maintenance of building functions or the resultant condition of a building is not a consideration. Design forces approach that of existing codes but use varying requirements of the ratio of excess capacity of a component to the demand required of the component. The report notes the "final decision regarding adequacy, need for further study, or need for strengthening still rests with the engineer, regardless of included statement, and therefore the procedures must be applied by a knowledgeable structural engineer".

These procedures are not Code procedures and are not necessarily acceptable by Code authorities. They have been developed to help the engineer in this rather complex problem and are particularly helpful to engineers that have had little seismic assessment experience. Experienced engineers will find that the short cut methods are in total not necessarily short and in addition may tend to cloud the real deficiencies. The assessment procedures used for this report include pertinent data from ATC-14 and ATC-22, augmented by data developed by CWMM from innumerable building evaluations carried out by the firm.

The force level to which existing structures should be upgraded is difficult to establish as there is not a large enough data base to establish a risk assessment. Because of technological factors, most deem it appropriate to allow existing structures to be analyzed for forces that are lower than those required for new structures. This approach appears acceptable with respect to increased risk of property damage but not acceptable with respect to the increased risk of loss of life. The approach can be considered if:

a) Damage does not cause progressive collapse.

b) If the building is not a post disaster structure.

c) If the occupancy is low.

d) If the occupants are not prevented from escaping from the damaged building by blocked corridors, etc.

It has been noted that all the structures listed by Port Alberni are intended to be post disaster facilities, and as such will be reviewed for the full Code forces, that is, the forces associated with the probability of exceedence of 10% in 50 years.

6.0 GENERAL CONCLUSIONS

The detailed results of the investigation of each of the 14 buildings are included in the enclosed Appendix. A summary is included here.

In general, the majority of the structures in this inventory lack adequate horizontal diaphragms to transfer forces to the shear walls. This is a common problem associated with buildings constructed during the 1950's and 60's that used tongue and groove material for decking. An inexpensive plywood skin installed over the top would have alleviated this problem in the majority of cases. However, the need for a good diaphragm was not well understood at the time. The small amount of testing that has been conducted on tongue and groove construction to assess its capability as a diaphragm confirms that it has only a very small capacity, which in turn is highly dependent upon the nailing.

The majority of the structures in this inventory depend upon unreinforced masonry to act as shear walls and as load bearing walls. Historically, this type of construction has behaved extremely poorly in earthquakes, often because of the inadequate connection between the diaphragm and the wall. Most building codes, including the N.B.C. since the mid-70's, have banned the use of unreinforced masonry in areas of seismic risk such as Port Alberni. It should be noted that seismic forces in Port Alberni are anticipated by Code to be 50%

greater than in Vancouver.

The lack of adequate diaphragms, the use of unreinforced masonry walls as shear walls, the use of unreinforced masonry walls as bearing walls, and the inadequacy of connections between the seismic elements are common deficiencies that have been found in the Port Alberni inventory. These must be addressed to protect life because their inadequacy can cause the structure to collapse.

Deficiencies in non-structural elements such as unreinforced masonry non-load-bearing partition walls may cause them to fail during an earthquake. Their destruction, however, will not cause a building collapse, but could injure those nearby and may impede evacuation by filling exiting routes with debris. For post disaster facilities, the failure of non-load-bearing unreinforced masonry walls may affect function, as much of the electrical and plumbing is mounted on or passes within or on the surface of such walls.

Upgrading uptions discussed with each structure are options that are expected to be economically appropriate for the building without regard to other potential upgrading that may be contemplated by the owners. The final solution should be part of a package which includes all works.

In general, a slightly conservative approach was taken for the assessment. The conservatism was not to the extend that a building would meet Code requirements if a less conservative approach was taken, nor was it such that the viability of upgrading was affected, but rather it was in the amount of upgrading, i.e., length of shear walls, extent of new diaphragm, etc., that was required.

6.1 PA1 - JOHNSTON ROAD PUMPSTATION

This recently constructed structure meets the intent of the latest Code and as such will not require upgrading. Some minor upgrading will be required to stabilize equipment.

6.2 PA2 - SOMASS PUMPSTATION

This structure is deficient in that the roof is unable to act as a diaphragm and the unreinforced masonry walls are unable to act as shear walls. The likelihood is that the structure will collapse during a quake approaching Code magnitude. It is likely that a moderate quake will cause extensive damage. Some minor upgrading will be required to stabilize equipment.

6.3 PA3 - WALLACE STREET PUMPHOUSE

This structure is deficient in that the unreinforced masonry walls are incapable of acting as competent shear walls. Part of the deficiency is due to the physical condition of the walls. The likelihood is that the structure will collapse during an earthquake of moderate magnitude. Some minor upgrading will be required to stabilize equipment.

6.4 PA4 - 4TH AVENUE PUMPHOUSE

This structure is deficient in that the unreinforced masonry walls are incapable of acting as competent shear walls. The likelihood is that the structure will withstand a moderate earthquake but will collapse during a Code earthquake.

6.5 PA5 - BAINBRIDGE PUMPSTATION

This structure is deficient in that the roof is unable to act as a diaphragm and the unreinforced masonry walls are unable to act as shear walls. The likelihood is that it will be heavily damaged during a moderate quake and will collapse during a quake approaching Code magnitude. The generator and the chlorination equipment should be addressed.

6.6 PA6 - COWICHAN PUMPSTATION

This wood-framed structure meets the intent of the latest Code and as such will not require any upgrading except to increase the number of bolts attaching the structure to the floor. Some minor upgrading will be required to stabilize equipment.

6.7 PA7 - PUBLIC WORKS OFFICE AND MAINTENANCE BUILDING

This structure will be extensively damaged during a moderate quake and will collapse in part or in total during a Code quake. The roof diaphragm, shear walls, and the connections between the two are inadequate. Unreinforced masonry bearing walls will collapse. The non-structural aspects of the building are not severe and in general damage to them would not be life threatening. Upgrading of the structural components will alleviate many of the most serious non-structural problems (electrical panels mounted on unreinforced masonry walls, etc.).

The most serious of the non-structural items is the inadequate connection of the propane tank to its foundation, and the unknown details of its foundation.

6.8 PA8 - EQUIPMENT AND STORAGE BUILDING

The masonry portion of the structure at the north end will collapse in a moderate earthquake. It has already been damaged by the installation of the salt storage adjacent. In addition, the roof diaphragm, shear walls and the connections between the two are inadequate.

The masonry portion of the structure at the south end is in good condition but it lacks an adequate roof diaphragm, shear walls, and the connections between the two. It is expected that it will be fairly heavily damaged during a moderate quake and will collapse in part or in total during a Code quake.

The central portion of the structure does not have an adequate diaphragm but a bracing system has been recently added. This portion of the structure in its present condition will be able to withstand a moderate earthquake but will be severely damaged during a Code quake. The end portions that bear on the masonry walls will collapse during a moderate quake.

The non-structural aspects of the building are not severe and in general damage to them is not life threatening.

6.9 PA9 - PORT ALBERNI CITY HALL

The upper structure will sustain extensive damage during a moderate quake and will suffer severe damage and partial collapse during a Code quake. The roof diaphragm, shear walls, and the connection between the two are substantially inadequate. Exits are highly susceptible to falling debris.

The lower portion, including the suspended concrete slab will suffer damage but will not likely collapse during a Code quake.

Of the non-structural damage, the most extensive will be the perimeter glass damage, particularly at the building's corners and the entrances.

6.10 PA10 - FIRE HALL

This structure will sustain some damage during a moderate earthquake and will likely collapse in part or in total during a Code quake. The most vulnerable area is the truck section and the tower. The roof diaphragm, shear walls, and the connections between the two are substantially inadequate. Unreinforced masonry walls will collapse. It is expected that the truck doors will jam during a moderate quake.

The most serious of the non-structural items would be vulnerability of the dispatch centre to severe damage, by components falling off their supports, by falling debris, and by the unreinforced wall that supports much of the electrical panels, etc., collapsing. Other serious non-structural items are the probable collapse of the partition wall in the truck area between the truck parking and the maintenance area, and the inadequate bracing of the compressors, tanks, and air filling equipment.

6.11 PA11 - ALBERNI HEALTH CENTRE

This structure will be damaged during a moderate quake and will likely be severely damaged during a Code quake. Small portions may collapse. The roof diaphragms are generally adequate. However, the shear walls and the connection of the roof and floor diaphragms to the shear walls are quite inadequate. The most serious deficiencies are associated with the two-storey assembly wing.

The building is in the zone that would be affected by Tsunamis.

Some minor upgrading will be required to stabilize equipment but none is of the life threatening nature.

6.12 PA12 - CURLING RINK AND SKATING ARENA

The curling and skating rinks will be extensively damaged during a moderate quake and will collapse in part or in total during a Code quake.

The large roof areas are inadequate to act as diaphragms. Connections to the vertical elements are inadequate. There is a severe lack of shear walls. The unreinforced masonry infill walls will be effective as shear walls for a small quake. However, they will fail as the magnitude increases and the structure will collapse. No lateral restraint at all exists at the east wall of the skating rink and the west wall of the curling rink.

The most serious non-structural deficiencies are the inadequacy of some of the equipment bracing and connections in the brine room and the lighting in the arena, which can sway and smash against the trusses.

6.13 PA13 - ECHO CENTRE

This structure will be extensively damaged during a moderate earthquake and will collapse in part or in total during a Code quake. The various additions were not integrated to handle lateral forces in any consistent defined manner. Rather, as the additions were added, each relied on the other for lateral support. Unfortunately, the roof diaphragms, the shear walls, and the connections between the two are substantially inadequate. The most stable portion of the facility is the 1981 museum expansion which was designed to modern codes and included a proper roof diaphragm and reinforced masonry shear walls. It is expected that this portion will remain intact except at the interface between this addition and the original museum.

The most serious of the non-structural items would be the large glass panel at entrances and corridors, which will shatter, lighting over the pool which will likely fall, and the loose equipment in the chlorination room.

6.14 PA14 - PUBLIC SAFETY BUILDING

This structure should be able to withstand a moderate quake with little damage. During a Code quake there is likelihood of extensive damage and probably some local collapse. The lower floor should suffer only minor damage during a Code quake. This structure is one of the more capable of the inventory to resist seismic forces.

The non-structural aspects of the building are not severe and in general damage by them is not life threatening. The most important of the non-structural items is the protection of the communications equipment, both to prevent it from falling and to prevent other items from falling onto it.

7.0 GENERAL RECOMMENDATIONS

(a) It is recommended that because of the large number of structures that are deficient, and the high cost associated with upgrading the total inventory, that seismic upgrading be contemplated when other works required by changes in occupancy, changes in function or general improvements are carried out.

(b) It is recommended that upgrade emphasis be placed on structures that house communication centres such as the Fire Hall, the Public Safety Building and the Works Yard. If funds are not available for the total facility upgrade, then communication rooms and equipment should be protected or relocated to safe areas.

(c) The most critical pumphouses should be considered for early upgrade since they are extremely important for post disaster functions and the costs to upgrade are low.

(d) Where the staff, rather than the building is required for post disaster functions, as may be the case with the Alberni Health Unit, it is suggested that rather than upgrade, a post disaster operations job plan be developed for the staff. It has been noted that this building does not house any special equipment or medicines that may be required. In addition, it is situated in an area that can be affected by a Tsunami.

(e) Consideration should be given to relocating critical facilities located in areas that may be affected by Tsunamis.

(f) Generally upgrade important equipment by providing flexibility between the equipment and its piping and electrical connections.

(g) It is recommended that since the upgrading of many non-structural items can be addressed economically and easily, a mitigation program be developed and improvements be started as soon as possible. It is anticipated that much work can be done by staff, without the need for employing external contractors.

(h) It is recommended that particular attention be paid to the remediation of unreinforced masonry walls and the large glass panels that may exist along corridors and other routes that would be used for evacuation. Damage to these components can occur with moderate earthquakes that may not cause the building as a whole to collapse.

(i) Several options exist for the continued use of the Equipment and Storage Building (PA9). It is suggested that upgrading be delayed until overall planning is completed, at which time the worth of retention of this structure can be established.

(j) It is recommended that if seismic upgrading is contemplated for the pumphouse building structures, that serious consideration be given to replacing the existing buildings with wood or steel studs.

8.0 COSTING

8.1 GENERAL

It has been assumed in the costing that a competitive market exists, with several contractors capable of providing bids.

The costs have been developed for the 1990 Vancouver market. Many contractors in this area have extensive upgrading experience specifically related to seismic improvements.

During the assessment stage, most of the architectural finishes are not removed, and therefore, a thorough review of the existing structure is not possible until construction commences. Contractors bidding retrofit work are concerned about the unknowns and the surprises that appear and thus they try to adjust their prices accordingly. It is the responsibility of the mitigation designer to detail the works as completely as possible and the responsibility of the Owner to understand and be prepared for the "extras" that are part of this kind of work. It is recommended that a minimum 30% contingency be included for this type of work.

Where several solutions are available to repair a deficiency, then a representative detail has been costed.

The mitigation costs developed cannot be of the same accuracy as those developed by a contractor while costing a project from tender documentation.

The costs for architectural, mechanical, and electrical upgrading associated with the structural upgrading requirements, as well as the opportunity that the mitigation process affords for additional modifications to the structure to improve function can obviously not be included. The cost included are those to return the structure to the condition that it was prior to seismic upgrading. For example, if to install a floor diaphragm it was necessary to remove some of the walls, then the costs included are those to remove the walls and flooring, install the diaphragm, and then replace the walls and flooring to their original condition. Miscellaneous wiring, plumbing, painting, demolition and cleanup have been included.

Consulting fees, permits, and the proposed G.S.T. have not been included in the cost estimates.

Costs do not include those for upgrading non-load-bearing unreinforced masonry partitions and enclosure walls, that if damaged would not cause structural collapse. Obviously, some or all these walls should be upgraded to minimize injury to the occupants and to maintain function. Costs for these are usually left to the time when actual mitigation is decided upon and the total work is considered.

8.2 COSTS

PA1 -	Price as new structure using local cost data.	
PA2 -	Price as new structure using local cost data.	
PA3 -	Price as new structure using local cost data.	
PA4 -	Price as new structure using local cost data.	
PA5 -	Price as new structure using local cost data.	
PA6 -	Price as new structure using local cost data.	
PA7 -	Public Works facility	$628,000.00
PA8 -	Equipment and Storage Building	Too many options.
PA9 -	City Hall	$265,000.00
PA10 -	Fire Hall	$290,000.00
PA11 -	Health Unit	$140,000.00
PA12 -	Curling and Ice Rink	$865,000.00

PA13 -	Echo Centre	$655,000.00
PA14 -	Public Safety Building	$350,000.00

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APPENDIX

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1.0 PA1 TO PA6 PUMPSTATIONS

1.1 GENERAL

The collapse of a pumphouse enclosure poses minimal risk for injury or to life safety. The locations are such that the probability of a pedestrian passing a pumphouse during a major earthquake is infinitesmal. A slightly higher risk is associated with a staff member being in the facility at the time of such a quake, though this probability is still remote. Note that staff can readily escape from these structures.

Since life safety is not a consideration, the intent of the Code is met even if the structure does not meet seismic Code criteria.

However, since the function of the pumphouse is of major significance immediately after a quake, it is necessary to ensure that damage is such that it would not affect the facilities operation.

An earthquake may affect the facility in the following ways:

(a) The walls and roof collapse, impairing immediate access to the equipment. The equipment may not be damaged and the problem is only of inconvenience.

(b) The controls, located on walls that have collapsed have been damaged and electrical wiring, etc., has been severed. Repairs could take a significant time.

(c) The pumping equipment itself may be damaged. This is probably unlikely except for small valves, pressure gauges, etc. The effect of a resultant malfunction of a gauge on the operations is unknown but should be addressed by the Owners.

(d) The emergency generators, if mounted on vibration isolators such as springs will walk off their bases, overturn, and rupture fuel lines, etc. Similarly, battery packs will tear off from their supports.

(e) There is the possibility that piping may rupture at the interface of the pipe with the rigid wall. The probability is greater, however, that some of the lines will rupture elsewhere in the system.

1.2 PA1 JOHNSTON ROAD PUMPSTATION

GENERAL

This 1500 square foot facility is relatively new, constructed in 1985. It is located on high ground and thus not subject to the actions of a Tsunami. Its present use is as a water pumpstation and a reservoir.

The roof structure is constructed of metal decking supported on steel joists, which in turn are supported on reinforced masonry bearing walls.

SEISMIC BEHAVIOUR AND DEFICIENCIES

The superstructure is of good detailing and construction and meets the intent of the Code. It is unlikely that the walls will collapse.

The pumps appear adequately anchored.

The vertical risers are braced to the walls in one direction. It is recommended that they be braced in two directions and generally at a spacing of approximately 10 feet on centre.

Cradles supporting the horizontal runs off the floor may be inadequate during an earthquake, as they may overturn. It is recommended that the locations be reviewed, and where in doubt, add supports or modify the existing supports.

The electrical MCC's and panels appear adequately connected to the walls, though actual details were not accessible for review.

The solenoid control valves that are mounted on the equipment should be protected from falling debris.

The lenses from the light fixtures will probably fall out.

The standby generators are mounted on vibration isolation springs. These mountings have behaved very badly during earthquakes, including the recent Loma Prieta quake in California. It is recommended that 4 snubbers be installed to limit the movement in all directions. It is also recommended that a stiffener be installed at this location if the support channels have no cross members.

The starter battery pack should be strapped to the housekeeping pad to ensure that the batteries do not topple.

The structural adequacy of the reservoir is not in the scope of this work.

SUMMARY

The structure of this pumphouse will behave adequately during a code quake. It is recommended, however, that some minor upgrading be carried out to stabilize the equipment.

1.3 PA2 - SOMASS PUMPSTATION

GENERAL

This 525 square foot facility was constructed in approximately 1960. It is located adjacent to the Somass River and may be subject to a Tsunami.

The roof is constructed of 2 inch tongue and groove decking supported on timber beams at 4 feet on centre. The roof is supported on 14 foot high unreinforced masonry walls. The pits below grade are constructed of reinforced concrete.

Its present function is as a water intake, chlorination, pumpstation.

SEISMIC BEHAVIOUR AND DEFICIENCIES

The 2 inch decking is incapable of acting as a roof diaphragm and the unreinforced masonry walls cannot be considered as adequate to act as shear walls or bearing walls. Because of the short spans and small area, the stresses generated are small, and roof upgrade would not normally be warranted. However, because of the high walls, and the building's classification as a post disaster facility, upgrade is recommended. To ensure that the structure can withstand the Code earthquake, a 1/2 inch sheet of plywood should be installed over the roof area or alternately on a 4 foot strip around the underside perimeter of the roof. The masonry walls should be backed up by an interior stud wall with a properly installed plywood skin to which the unreinforced masonry should be attached.

As previously discussed, if the structure is not upgraded, the risk to life is minimal. However, consideration must be given to upgrading to protect the enclosed equipment.

The electrical MCC's and panels are mounted on unreinforced walls and as such are susceptible to severe damage if the wall collapses. Similarly the electrical equipment will be damaged by an overall collapse. This would probably cause the facility to cease operation.

The chlorine tanks and equipment are badly supported and should be more adequately restrained, if there is a possibility of leakage and if this leakage will affect the operation of the plant immediately after a quake.

The pumps appear adequately restrained.

The bracing of vertical equipment is presently in one direction only. This should be upgraded to two-directional bracing at a spacing of approximately 10 feet on centre.

The horizontal pipe runs should be bolted more substantially to the buttresses.

Standby generators were not noted but if they exist, then they should be reviewed in the same manner as discussed PA1.

The wet/dry pits would probably survive a Code quake unless severe geological damage occurs.

SUMMARY

The structure of this pumphouse will likely collapse during a Code quake. It is expected that a major Tsunami would destroy the structure and would make the facility inoperative even with upgrading because of damage to controls, etc. Some minor upgrading should be carried out to the supports of the mechanical equipment.

1.4 PA3 - WALLACE STREET PUMPHOUSE

GENERAL

This 200 square foot facility was constructed in approximately 1957. It is located at the back of the Public Works Yard, and it is unknown whether it would be subjected to a Tsunami.

The structure has a low pitched wood roof with a ceiling. The roof is supported on 8 foot high unreinforced masonry walls. The wet/dry pit is constructed of reinforced concrete.

Its present use is that of a sewer pumpstation.

SEISMIC BEHAVIOUR AND DEFICIENCIES

The wood roof construction, including the ceiling, is capable of acting as a diaphragm and can brace the tops of the unreinforced masonry walls. The unreinforced masonry walls cannot, by Code, be considered as adequate to act as shear walls or bearing walls. The large entrance on one side eliminates the capacity of this wall and causes torsion. Existing cracking in the wall further reduces its seismic capacity and increases the risk of collapse. It is expected that it would collapse during a moderate quake. To ensure that the structure can withstand the Code earthquake, the masonry walls should be backed up by an interior stud wall with a properly installed plywood skin to which the unreinforced masonry should be attached. The prime purpose of the upgrade would be to ensure that the electrical equipment mounted on the walls will not be damaged by a collapse of the wall.

The pumps are adequately connected to their supports. However, the brittleness of the "legs" is not known and should be reviewed.

The horizontal piping is well supported. However, the large diameter vertical pipes, particularly at intersections, should be braced in two directions.

The power pole located within feet of the building and supporting several transformers may be inadequately braced. If the pole falls, then power to the pumphouse is lost and access restricted by fallen wires.

Standby generators were not noted but if they exist, then they should be reviewed in the same manner as discussed for PA1.

SUMMARY

The structure of this pumphouse will likely collapse during a moderate quake. It is suggested that upgrading be carried out to the walls and equipment to maintain the function of the facility.

1.5 PA4 - 4TH AVENUE PUMPHOUSE

GENERAL

This 125 square foot facility was constructed in approximately 1975. It is unknown whether this structure would be subjected to a Tsunami.

The structure consists of a low pitched roof constructed of 2 in tongue and groove decking supported on a wood beam at the centre. The 8' high walls are constructed of unreinforced masonry.

Its present use is that of a sewer pumpstation.

SEISMIC BEHAVIOUR AND DEFICIENCIES

Though the roof is constructed of 2 inch tongue and groove material, a material generally considered as not capable of acting as a diaphragm, it is expected that because of the small area and short spans that the stresses would be low enough that the roof would function adequately. The connection to the masonry appears adequate.

The unreinforced masonry walls cannot, by Code, be considered as adequate to act as shear walls or bearing walls. This structure would withstand a moderate earthquake, but would be severely damaged by a Code quake. To ensure that the structure can withstand the Code earthquake, the masonry walls should be backed up by an interior stud wall with a properly installed plywood skin to which the unreinforced masonry should be attached. The prime purpose of the upgrade would be to ensure that the electrical equipment mounted on the walls will not be damaged by a collapse of the wall.

The piping is adequately supported.

Standby generators were not noted but if they exist, then they should be reviewed in the same manner as discussed for PA1.

SUMMARY

The structure of this pumphouse will be able to withstand a moderate quake but will probably collapse during a Code quake. It is suggested that upgrading be carried out to the walls to ensure the facility remains functional.

1.6 PA5 - BAINBRIDGE PUMPSTATION

GENERAL

This 700 square foot facility was constructed in approximately 1962. It is located on high ground and would not be subject to a Tsunami.

The roof of the structure is constructed of 2 inch tongue and groove decking supported on wood beams at approximately 3.5 feet on centre. The roof is supported on approximately 9' high unreinforced masonry walls.

Its present use is that of a sewer pumpstation.

SEISMIC BEHAVIOUR AND DEFICIENCIES

Though the roof is constructed of 2 inch tongue and groove material, a material generally considered as not capable of acting as a diaphragm, it is expected that because of the small area and short spans that the stresses would be low enough that the roof would function adequately. The connection to the masonry appears adequate.

The unreinforced masonry walls cannot, by Code, be considered as adequate to act as shear walls or bearing walls. This structure would withstand a moderate earthquake, but would be severely damaged by a Code quake. To ensure that the structure can withstand the Code earthquake, the masonry walls should be backed up by an interior stud wall with a properly installed plywood skin to which the unreinforced masonry should be attached. The prime purpose of the upgrade would be to ensure that the electrical equipment mounted on the walls will not be damaged by a collapse of the wall.

The piping is adequately supported.

Standby generators were not noted but if they exist, then they should be reviewed in the same manner as discussed for PA1.

SUMMARY

The structure of this pumphouse will be able to withstand a moderate quake but will probably collapse during a Code quake. It is suggested that upgrading be carried out to the walls to ensure the facility remains functional.

1.6 PA5 - BAINBRIDGE PUMPSTATION

GENERAL

This 700 square foot facility was constructed in approximately 1962. It is located on high ground and would not be subject to a Tsunami.

The roof of the structure is constructed of 2 inch tongue and groove decking supported on wood beams at approximately 3.5 feet on centre. The roof is supported on approximately 9' high unreinforced masonry walls.

Its present function is as a chlorination and water pumpstation.

SEISMIC BEHAVIOUR AND DEFICIENCIES

The wood roof construction is incapable of acting as a diaphragm and the unreinforced masonry walls cannot be considered as adequate to act as shear walls or bearing walls. The connection between the roof and walls is inadequate. To ensure that the structure can withstand the Code earthquake, a 1/2 inch sheet of plywood should be installed over the roof area or over a 4 foot strip around the perimeter of the roof. The masonry walls should be backed up by an interior stud wall with a properly installed plywood skin to which the unreinforced masonry should be attached.

As previously discussed, if the structure is not upgraded, the risk to life is minimal. However, consideration must be given to upgrading to protect the enclosed equipment.

The electrical equipment, though generally well connected, is connected to unreinforced walls and as such is susceptible to severe damage if the wall collapses.

The chlorine tanks and equipment are badly supported and should be more adequately restrained, if there is a possibility of leakage and if this leakage will affect the operation of the plant immediately after a quake.

The standby generator should be adequately mounted to ensure that it does not move or topple, rupturing feed lines.

The pumps appear adequately restrained.

The bracing of vertical pipes should be in two directions and generally at a spacing of approximately 10 feet on centre.

The piping chamber would probably survive a Code quake unless severe geological damage occurs.

SUMMARY

The structure of this pumphouse will likely collapse during a Code quake. It is suggested that upgrading be carried out to the roof, walls, generator and chlorination system to ensure that the facility remains functional.

1.7 PA6 - COWICHAN PUMPSTATION

GENERAL

This 600 square foot facility was constructed in approximately 1970. It is located on fairly high ground and would probably not be affected by a Tsunami.

The building is a wood frame structure consisting of a wood joist roof and wood stud walls. The piping pit is constructed of reinforced concrete.

Its present use is that of a water pumpstation.

SEISMIC BEHAVIOUR AND DEFICIENCIES

The wood frame structure is capable of withstanding the Code earthquake if minor upgrading is carried out. It is recommended that additional bolts be installed to connect the walls to the floor. It is recommended that since no data exists on the connection between the roof and the wall, that clips be installed to ensure adequate transfer ability.

The electrical equipment is mounted on an adequate wall.

The pumping equipment is adequately mounted.

The light fixtures do not appear adequately braced and it is likely that they will tear loose and be damaged. Similarly, the lenses should have clips installed to prevent them from falling out.

It may be advisable to brace some of the valves mounted on the equipment to ensure that they do not tear off the pumps, etc.

Standby generators were not noted but if they exist, then they should be reviewed in the same manner as discussed for PA1.

2.0 PA7 - PUBLIC WORKS OFFICE AND MAINTENANCE BUILDING

2.1 GENERAL

The Public Works Office and Maintenance Building, located at 4150 6th Avenue, was constructed in 1966. The architectural design was by Carlberg Jackson Partners, Architects.

The structure is the operations and maintenance centre for the Public Works Department and as such would be a staging area during a disaster.

It is not known whether the building is located in a zone that could be affected by a Tsunami.

2.2 CONSTRUCTION

The roof is constructed of 1/2 inch unblocked plywood supported on wood joists at 16 inches on centre. The joists span onto glulam beams which in turn are supported generally by unreinforced masonry walls. The walls are supported on strip footings. A number of small cracks exist, primarily toward the western end of the structure. It is understood that the building was constructed partially on fill, which could explain some of the cracking. Though this does have some effect on seismic performance, it is not significant.

2.3 SEISMIC PATH

In its present condition, the structure relies on the plywood roof to act as a diaphragm, transferring forces to the vertical elements. These vertical elements consist of the unreinforced masonry walls.

2.4 SEISMIC DEFICIENCIES

A. STRUCTURAL

1) The roof diaphragm is constructed of 1/2 inch unblocked plywood, with an unknown nailing pattern. Calculations show that the roof is incapable of transferring load to available walls and as such must be upgraded.

   Approximately 70% to 75% of the roof area will have to be upgraded to meet the requirements for an adequate diaphragm. Blocking and a new plywood diaphragm may be installed on the underside of the wood roof joists, and connected adequately to the vertical transfer elements. Venting must be considered. The new diaphragm will generally be installed around the perimeter of the building in 12 or 16 foot widths, with some intermediate 16 foot runs in the north/south direction. Blocking of the existing diaphragm may be considered. However, care must be taken in installing the blocking to ensure that the roofing is not damaged. In either case, some roofing will have to be removed to confirm the existing nailing pattern. Alternately, roofing and insulation may be removed from the top surface, blocking and proper nailing installed, and then roofing is reapplied. The latter will negate the extra plywood and venting requirements.

2) The use of unreinforced masonry walls as seismic shear transfer elements is not acceptable by Codes and therefore new elements or upgrade of the existing must be considered. Options would be to remove and replace appropriate unreinforced walls with reinforced masonry, cast-in-place concrete, or gunnite. Approximately 10 walls each 16 feet long will be required in each direction. The walls will each require upgraded foundations of approximately 20 feet x 1.5 feet x 4 feet. These foundations will have to take care of the vertical tension and compression forces associated with the

moment couple created by the lateral forces. The foundations will be reasonably easy to install since the structure is lightweight.

3) To transfer loads to the new shear walls, drag struts or collectors will be required to connect the diaphragm to the top of the wall. These can be constructed of steel angles or wood ledgers. This will also negate the inadequate connection of the sill to the walls, (1/2 diameter bolts at approximately 48 inches on centre).

4) Unreinforced load bearing masonry walls, that is, walls onto which the joists' span do not meet Code requirements and must be upgraded. New beam/ columns can be installed to ensure that if the wall collapses, the structure will stand up. Alternately, a new stud wall constructed adjacent to the existing masonry, and taking the gravity load, can be installed. The masonry should be connected to the new wall to allow the out-of-plane forces from the masonry to be transferred and prevent the wall from falling out, injuring personnel.

5) The unreinforced non-load-bearing walls do not meet Code requirements. Failure of such walls will not cause a collapse of the structure but may injure personnel, damage equipment, interfere with function, and impede evacuation because of debris. It is recommended that these walls be reviewed in relationship to the number of personnel that can be affected, important equipment that may be damaged, and potential interference with egress. Selective upgrading should be considered. Upgrading can consist of replacement with reinforced masonry, replacement with steel stud wall, or back up with an auxiliary wall to which the masonry would be attached. This portion of the upgrading is usually undertaken when renovations or alterations due to change in function or need are anticipated.

6) The pilasters are reinforced and can carry the required gravity loads as well as the tributary lateral loads from the "attached" unreinforced walls. Thus, if the wall collapses on either side of the pilaster, the structure should stand. However, there is the possibility that the twisting, and other effects will have a tendency to exert excessive force on the pilaster, potentially damaging its capacity to carry load. This is of some concern because the exact nature of the "as built" pilasters is not known.

It is recommended that the pilasters be upgraded to ensure both a more positive gravity load transfer and a reduced risk of lateral load transfer from the masonry walls. This can be done by either adding an independent column or modifying the existing pilaster. The possible need for this work should be reviewed once mitigation is decided upon.

7) The bearing of glulam beams on the 8 inch unreinforced masonry walls is unacceptable because of the high risk of partial collapse of the structure.

Bearing should be provided under the glulams either in the form of a new column or a replacement of a section of the existing masonry, with a concrete or reinforced masonry section. Foundations would not have to be upgraded.

8) The lintels over the large door openings cannot be considered as satisfactory to support the structure because of inadequate detailing of both the lintel and its support on the wall. This may initiate partial collapse. Steel channel lintels should be installed across the openings to transfer the loads to the upgraded pilasters. Foundations will not likely have to be upgraded.

2.5 NON-STRUCTURAL ELEMENTS

The majority of the non-structural elements such as ceiling tiles, fluorescent light lenses, shelving, etc., will not affect the operation of the facility or seriously hurt personnel if they are damaged or fall and thus need not be considered in this assessment. Some selective upgrading could be carried out at the time of mitigation. The following items, however, have greater significance.

1) The bolted connection of the propane tank to its supports appears inadequate and should be upgraded with a large bolt and some stiffening. The foundation details are not known, but experience in other areas has shown that foundations are often inadequate with the unit subject to overturning.

2) All communication equipment that is expected to be used after an earthquake should be connected to supports and protected from falling debris. Electrical service to the equipment, particularly panels, should not be attached to unreinforced masonry walls unless these walls are upgraded to ensure that they remain stable.

3) Equipment, particularly emergency generators that are mounted on spring isolators, should have snubbers or similar devices added to control movements.

2.6 PARTIAL UPGRADING

Upgrading to meet partial Code requirements is not recommended where life safety is at risk, or the structure serves a post disaster function and must remain operational after a disaster.

If such upgrading is contemplated, it should be directed to the following:

- The roof diaphragm should be upgraded.
- Unreinforced load bearing walls should be upgraded.
- Shear walls should be installed, including drag struts.
- Selective upgrading of non-load-bearing unreinforced masonry walls should be carried out in areas where communication equipment must remain functional.

3.0 PA8 - EQUIPMENT AND STORAGE BUILDING

3.1 GENERAL

This structure, located at 4150 6th Avenue, directly behind the Public Works Office and Maintenance Building, was constructed in 1966. The architects were Carlberg Jackson Partners, Architects. A salt storage extension was constructed at a later date.

This structure shelters many vehicles, which could be partly immobilized if the building collapses during an earthquake.

It is not known whether the building is located in a zone that could be affected by a Tsunami.

3.2 CONSTRUCTION

The roof of this structure is constructed of 1/2 inch plywood supported on "Truss Joists" spaced at 24 inches on centre. The joists span onto glulam beams, which in turn are supported on steel columns.

Recently, steel cross bracing was added in both the north/south and the east/west directions, in the open areas.

It is assumed that the soil is of less than average capacity. This is assumed because of the large consolidation of the soil that has occurred adjacent to the north end due to the stockpile of sale. Severe cracks have developed in the masonry.

3.3 SEISMIC PATH

In its present condition the structure relies on the plywood roof to act as a diaphragm, transferring forces to the vertical elements. These vertical elements consist of the unreinforced masonry walls and the recently installed cross bracing.

SEISMIC DEFICIENCIES

A. STRUCTURAL ELEMENTS

1) The unblocked plywood diaphragm does not have the capacity to transfer the required lateral seismic forces. Capacity can be achieved by blocking all

edges of the plywood, thus creating a blocked diaphragm.

This assumes that the new braces take their proportionate share of the loads.

2) For an east/west earthquake, the new braces can handle their required forces. Since this is new construction, it has been assumed that the actual members have been properly designed and uplift considered on the foundation and the connection to the foundation.

3) For a north/south earthquake it has been assumed that since the construction is recent, the members have been designed for the appropriate forces and that uplift has been considered. However, a collector/blocking member will have to be installed along the spine, to transfer the diaphragm forces to the level of the top of the bracing, i.e., to the underside of the glulam beams from the roof level. A length of approximately 6 bays will be required.

4) The structure at the north end is constructed of unreinforced masonry and as such does not meet Code requirements for either load-bearing or non-load-bearing structures. In addition, its seismic capacity has been substantially reduced by the cracks generated in the walls by the adjacent salt pile. This

portion of the structure will be severely damaged by a less than moderate quake. It is recommended that either:

(a) Bays 1 and 2 be dismantled with a new frame added to support line 3.

(b) Retain the masonry building, with no upgrading and demolish the first free span, i.e., the span between lines 3 and 4. No remedial work will be required for the canopy structure except to finish the edge. The space between the masonry building and the canopy structure could be cordoned off as a safety zone.

5) The structure at the south end is constructed of unreinforced masonry load-bearing and non-load-bearing walls. It will not meet Code requirements. However, it is in good condition and can be readily upgraded.

It is suggested that the load-bearing walls on lines 11, 12 and 14 be replaced by either metal stud walls or reinforced masonry. The existing roof is lightweight and can easily be shored.

The non-load-bearing walls on lines A, B and C should be upgraded by the installation of an interior stud wall to which the masonry can be attached. This will adequately transfer the required out-of-plane wall forces. Alternately, the non-load-bearing walls could be replaced in the same manner as the load-bearing walls.

6) The salt shed addition can be considered as stable. However, if the masonry structure is removed, then additional bracing or walls will be required.

3.5 NON-STRUCTURAL ELEMENTS

There are no non-structural elements that need be considered.

4.0 PA9 - PORT ALBERNI CITY HALL

4.1 GENERAL

The Port Alberni City Hall, located at 4850 Argyle Street, was constructed in 1958. The architectural design was by Wade Stockdill and Armour, Architects.

The location of the structure is such that it would not be affected by a Tsunami.

4.2 CONSTRUCTION

The roof of the two storey structure is constructed of 3 and 4 inch tongue and groove decking supported by steel joists at approximately 12 feet on centre. The joists are generally supported on steel columns but some are supported on unreinforced masonry walls. The perimeter is composed of lightweight infill panels and glazing, with some areas of unreinforced masonry.

There is a partial basement. The structure over this area is a suspended, cast-in-place concrete joist arrangement spanning onto concrete carrier beams. The slab and beams are supported on concrete columns and walls.

The structure appears sound with no sign of deterioration.

4.3 SEISMIC PATH

In its present condition, the structure relies on the wood roof deck and the concrete floor to act as diaphragms transmitting forces to the vertical seismic elements. These vertical elements are unreinforced shear walls between the roof and the suspended floor and reinforced concrete walls in the basement area.

4.4 SEISMIC DEFICIENCIES

A. STRUCTURAL

1) The roof diaphragm consisting of 3 and 4 inch tongue and groove decking cannot be considered as having any calculable diaphragm capacity, and as such, must be upgraded.

The upgrading can be accomplished by the addition of 1/2 inch plywood decking nailed to the decking to form a blocked diaphragm. The following strips of upgrade would be required:

- 16' wide strip along A from 1 to 12
- 16' wide strip along G from 1 to 3
- 12' wide strip along G from 7 to 12
- 12' wide strip along 1 from A to G
- 12' wide strip along 7 from A to G
- 12' wide strip along 12 from A to G.

It would be preferred that the upgrade be installed on the upper surface of the roof, necessitating removal and replacement of roofing and insulation. With careful detailing it would be possible to install all or part on the underside of the decking.

A horizontal steel cross bracing arrangement could be considered with panel points at the steel joists, beams and columns.

2) The use of unreinforced masonry walls as vertical seismic shear transfer elements is not acceptable by Codes and therefore alternate means of transfer must be provided.

Though cross bracing is not recommended for structures over two stories in height, it would be quite acceptable for this facility. Provide six sets of cross bracing between the roof and the suspended floor, in each direction, located at the building corners and in the vicinity of line 7. The exact location and number can be adjusted to suit the Owner's needs. The bracing must be connected to the concrete floor with inserts for a total of approximately 8 kips uplift. Collectors will be required at the diaphragm level to transfer the forces to the cross bracing.

At some locations it may be more feasible to use plywood clad shear walls installed against existing unreinforced masonry walls in place of the cross bracing. This is particularly appropriate at the eastern end of the structure where such shear walls can serve the dual purpose of transferring in-plane forces as well as resisting the out-of-plane forces generated by the adjacent masonry walls. A further step would be to remove the masonry and replace it with shear walls.

3) The load-bearing masonry walls between the roof and the suspended floor are unreinforced and as such do not meet Code requirements. Damage to these walls can cause structural collapse and thus they must be upgraded. Options include:

- The installation of steel stud walls on one side to brace the masonry wall. The wall must be attached by inserts at 24 inches on centre each way.
- Saw cut out the block wall and install concrete posts at 4 feet on centre.
- Break out or saw cut strips of block and fill with grout and reinforcement. Spacing to be 4 feet on centre.
- Shore the structure and replace the wall with appropriate alternative material.

4) The non-load-bearing masonry walls between the roof and the suspended floor are unreinforced and as such do not meet Code requirements. Damage

to such walls will not cause collapse but may injure workers. It is recommended that these be braced for out-of-plane forces or replaced. Emphasis should be on walls along exiting corridors.

5) The unreinforced masonry walls must be upgraded at locations where joists bear on them. Upgrading can be as per 3) or a small steel column can be installed and buried in either the new or the existing wall.

6) The entrance canopy appears satisfactory but at some stage the condition of the hangers including their connection to the steel frame should be checked for deterioration.

7) The ground floor suspended concrete slab is capable of acting as the required diaphragm and can transfer forces to the appropriate concrete shear walls.

8) The length of existing concrete shear walls between the ground floor and the basement is close to adequate. However, because of torsion it is recommended that additional walls be installed along line A from 10 to 12 and along line G from 10 to 11. Upgraded foundations will be required for

these walls. It may be possible to delete the requirement for these latter two walls, if structural drawings showing reinforcing details for the foundations and the suspended concrete floors become available.

4.5 NON-STRUCTURAL ELEMENTS

There are a minimal number of non-structural elements that must be considered. Those that require upgrading are not life threatening, but affect the function of a building immediately after an earthquake.

1) It is expected that the glazing, particularly at the corners, will shatter.

2) Portions of the suspended ceiling may collapse. It is recommended that the ceilings in exit corridors be seismically braced.

3) Hot water tanks and emergency generators must be adequately braced.

4) Computers and similar equipment that must be used for data processing or retrieval immediately after an earthquake should be adequately connected to

their desks and located such that falling debris does not damage them.

4.6 PARTIAL UPGRADING

Upgrading to a partial Code requirement is not recommended where life safety is at risk, or the structure serves a post disaster function and must remain operational after a disaster.

If partial upgrading is contemplated, emphasis should be placed upon improving the diaphragm action of the roof and upgrading the vertical seismic elements between the roof and the suspended floor.

5.0 PA10 - FIRE HALL

5.1 GENERAL

The Fire Hall was constructed in 1967 from a design by Carlberg Jackson Partners Architects. At present it houses the fire department as well as the ambulance service. It is understood that there is a proposal to relocate the latter service to other facilities. The building houses the receiving and dispatch facilities for fire, ambulance and other emergency services, functions that must remain operative after an earthquake. The building is situated on high ground and would not be affected by a Tsunami.

5.2 CONSTRUCTION

The building consists of three segments. The trucks are housed in a single storey structure, the roof being constructed of 2 inch tongue and groove decking supported on glulam beams. The glulam beams are supported on interior steel columns and exterior reinforced masonry pilasters. The masonry walls are unreinforced except for the east and west walls punctured by the truck doors. Portions of the roof are supported on the unreinforced walls. Continuous strip footings support the perimeter walls.

The living/office wing is a two storey structure with the roof constructed of 1/2 inch unblocked plywood on 2 x 12 joists at 16 inches on centre and the floor constructed of a ribbed reinforced concrete slab. The roof spans onto a wood framed north and interior walls and an unreinforced masonry south wall. This unreinforced wall also supports the north end of the truck area wing. The concrete floor is supported on unreinforced masonry walls between the foundation and the floor. Strip footings support the walls.

The hose tower is constructed of a concrete roof, concrete landings, and concrete stairs all supported on unreinforced masonry walls. The walls are supported on strip footings. This portion of the structure need not remain functional after an earthquake.

No basements or crawl spaces exist. Rather, the area is covered by a slab on grade.

Foundations are constructed on competent soils capable of supporting a 5,000 psf bearing pressure and liquefaction is not a consideration.

5.3 SEISMIC PATH

In its present condition, the building relies on the roofs and floors to act as horizontal diaphragms, transmitting forces to the vertical seismic elements which are the masonry and wood frame walls. The walls in turn transfer the forces to the foundations.

5.4 SEISMIC DEFICIENCIES

A. LIVING UNIT

1) The living unit roof assembly consisting of 1/2 unblocked plywood supported on 2 x 12 joists has the capacity to act as a diaphragm and does not require upgrading.

2) Inadequate shear walls exist between the roof and the second floor, both in the north/south and east/west directions.

The east and west walls, the walls around the staircase, and possibly the wall separating the sleeping area, all constructed of wood studs with gypsum board, must be upgraded by the installation of a 1/2 inch plywood skin to act as a diaphragm. The gypsum board must be removed to add intermediate blocking for the plywood. Collectors constructed of steel angles or wood ledgers must be installed at the level of the top of the wall to collect the forces and bring them to the shear wall. Additional bolting to the concrete floor will also be required.

The stud wall along the north side must be upgraded to include a 32 foot section of shear wall, constructed in the same manner as above. A collector will be required along the top to bring the forces to the new shear wall. Additional bolting will be required to the concrete floor.

The south wall of the living unit is constructed of unreinforced giant block and is not permitted by Code to be used in areas of potential high seismology such as Port Alberni. It is suggested that a steel or wood stud wall be installed along the north face of this wall, firmly attached to the masonry to prevent this material from breaking out. Generally a 24 inch square grid of attachment is adequate. A 32 foot segment of this wall should be clad in plywood to act as a shear wall as per previous discussions, while the remainder may be clad in gypsum board or other material. This latter segment will replace the gravity load carrying capacity of the masonry wall. Adequate attachment to the concrete floor will be required. A further modification to the masonry wall will be discussed in the truck parking section.

3) All the masonry walls between the foundation and the 2nd floor are unreinforced. Such walls when acting as load carrying elements are not allowed by the Code and thus must be upgraded.

Twenty foot sections of the north wall as well as similar lengths of the east and west walls will be required to act as shear walls. It is suggested that they either be replaced by reinforced concrete, reinforced masonry or gunniting be applied to the interior face. These walls must be attached to both the floor and the foundation. The exact location and extent can be adjusted to suit. Foundations will have to be enlarged to carry the forces generated by the lateral force.

The remaining masonry load bearing walls should be upgraded to carry gravity loads as per the code intent. Options include:

- drilling holes through the floor, installing reinforcing bars and grouting the reinforced voids. Attachment to the foundation will be required.
- installing a steel stud wall on the interior face of the exterior walls and on one or two faces of interior walls. It is recommended that the masonry be attached to these stud walls. Where the loads are large this solution is not recommended.
- gunnite the interior to the thickness required for gravity loading and attach to masonry to this new wall.

These solutions will not require a foundation upgrade.

B. TRUCK AREA

1) The 2 inch tongue and groove roof decking has no calculable value and should not be used alone as a diaphragm. The diaphragm can be upgraded by the installation of a 1/2 inch plywood deck attached to the existing decking. This will act as a blocked diaphragm. The minimum amount of diaphragm improvement would be the installation of a 16 foot strip around the perimeter of this area. Alternatively, it is possible to install a steel cross

bracing system under the roof. This will not require a roofing replacement but will require complex connections.

2) The south wall, being quite high, and unreinforced, is not capable of acting as a shear wall nor is it allowed by Codes to act as a gravity load carrying element. The middle 24 foot bay should be upgraded to carry the seismic forces. A new wall of reinforced masonry or gunnite can be installed against the existing wall or a heavy cross brace can be installed. A collector will be required along the top of the full length of the side to transfer the lateral forces to the wall. It is recommended that if cross bracing be used then a secondary system also be installed to ensure that components of the masonry wall will not fall in on personnel or equipment. For the remainder of the wall length, either a beam column arrangement or an interior wall must be installed to carry the gravity forces. It is recommended that an interior wall or a combination of beam and steel bracing be installed to ensure that components of the wall will not fall in.

3) The north wall assembly of the truck area cannot transfer shear forces to the foundation. It is recommended that approximately 40 feet of unreinforced

masonry between the suspended concrete slab and the roof of the truck area be removed and replaced by concrete or reinforced masonry. This is to be dowelled into the concrete slab. The area should be in the zone of the middle 24 foot bay. A collector must be installed at the underside of the truck roof to transfer the forces from the length of the roof to the new 40 foot section.

Between the concrete slab and the foundation, add a heavy steel cross brace between the two interior columns along this line. A shear wall could also be used if it does not interfere with operations. Alternately a shear wall can be installed at the location of the existing masonry wall separating the truck area from the office area. This will require special detailing but is quite feasible. Whatever solution is considered, the foundations must be upgraded to carry the forces resulting from the shear wall function. The exact amount of foundation upgrade will depend upon the length of wall that will be used. If the foundation becomes excessive, it may be advantageous to use soil tension anchors.

4) The east and west walls of the truck area can be considered as reinforced masonry, but because they are virtually all taken up by truck openings, with only reinforced pilasters left, there is no capacity to carry lateral forces. Their reinforcing and shape is such that they have some lateral capacity, but it must be increased with additional reinforcing, shear ties, and confirmation of dowelling to the foundation. It is recommended that rather than relying on this condition, that the "U" section be turned into a column of approximately 16 inches x 24 inches. This can be readily attached to the reinforced lintels and to the foundation. Whatever solution is considered, foundation upgrade will be required. At this location, this is quite simple to carry out.

C. TOWER AREA

The tower does not have the capacity to resist Code forces and does meet the requirement that masonry must be reinforced. The collapse of the tower will not affect the function of the building nor will its collapse, if it blocks a truck entrance affect operations. However, it may collapse onto the roofs of the living or truck area, injuring personnel and damaging equipment. It is recommended that the tower be upgraded nominally by the installation of horizontal tie rods around its perimeter, spaced at approximately 4 feet on centre from the top of the tower to the top of the living unit roof. This will anchor the corners, the most susceptible portion of the tower and will reduce the amount of material that would fall, thus reducing the risk. The alternate solution is to provide an interior or exterior skin of stud walls, gunniting or a grid of angles and mesh. This is not recommended as necessary at this time.

5.5 NON-STRUCTURAL ELEMENTS

1) A number of non-load-bearing walls exist, particularly in the truck area. They enclose the mechanical room, the compressor room and extend out to separate the work shop area from the trucks. If the mechanical room or compressor room walls are damaged, they may affect the equipment within the room either by damaging the components or damaging the wiring or air lines, etc. Note that many items are connected to these walls. These walls should be encapsulated to prevent this from occurring.

The wall separating the workshop from the truck area is the one most likely to be damaged. It is recommended that it be removed, replaced, or braced.

2) Some of the mechanical units are mounted on vibration isolators. These devices have behaved badly during earthquakes and it is recommended that snubbers be installed to limit all phases of the movement. These do not restrict the function of the original isolators.

3) The compressor should be mounted to the floor and non-load-bearing wall upon which the control and electrics are mounted should be upgraded to ensure that it is not damaged.

4) The dispatch centre has a number of deficiencies relating to seismic damage. Many of the components are sitting on desks and panels without attachment and could readily slide off. The desk is not fixed to the floor and can move around. The electrical wiring and some of the panels are attached to a non-load-bearing masonry wall and depend upon the stability of these walls. Such walls should be upgraded to prevent collapse.

It is understood that a new dispatch console may be installed in the near future. Protection of wiring and protection from flying debris should be part of the installation.

5.6 PARTIAL UPGRADING

Upgrading to meet partial Code requirements is not recommended where life safety is at risk or the structure serves a post disaster function and must remain operational after a disaster.

If necessary, upgrading of non-load-bearing walls can be omitted as can the upgrading of unimportant mechanical and electrical components. The tower can probably be left as there is no risk to function and the risk to life is small.

6.0 PA11 - ALBERNI HEALTH CENTRE

6.1 GENERAL

The Alberni Health Centre, located at 4711 Elizabeth Street, was constructed in 1964, with an addition in 1971. The architectural design of both the original structure and the addition was by Barley and Weismiller, Architects.

At present, this structure houses the Public Health nurses, the sanitation inspection service and many of the community health related functions such as prenatal services, immunization, etc. There are minimal strategic, emergency material stored at this location. Its prime function is that in addition to the hospital, it can be a pivotal point for some aspects of emergency health care.

The building is situated on low ground and it is expected that it is in an area that is susceptible to Tsunami action. A Tsunami is not necessarily generated by local earthquakes, nor do local earthquakes necessarily generate Tsunamis. The question as to whether the function should be moved to a safer location, because one cannot readily protect against Tsunamis should be left to local jurisdictions.

6.2 CONSTRUCTION

The building, constructed in the approximate form of a "Z" consists of three elements.

The assembly area is two stories in height. It consists of roof constructed of diagonally installed shiplap on 2 x 6 joists at 16 inches on centre, supported by glulam beams spaced approxmately 8 feet apart. Part of the roof is supported on load bearing walls. The floor is constructed of 1/2 inch plywood on diagonally installed shiplap on 2 x 8 joists at 16 inches on centre. It is supported by load-bearing walls. There is a slab-on-grade and no crawlspace. The perimeter walls are of unreinforced masonry, while the interior walls are of wood stud construction. Strip footings support the structure.

The south wing is a two storey office area. The roof of this wing is constructed of diagonally installed shiplap on 2 x 8 joists at 16 inches on centre. The roof is supported on load-bearing interior and exterior walls. The floor is constructed of 1/2 inch plywood on diagonally installed shiplap on 2 x 8 joists at 16 inches on centre. The floor is supported along the exterior on a concrete beam/frame that forms the basis of the exterior structure and on stud walls within the interior. The second floor walls are wood stud while the lower floor walls are masonry infill of the concrete beam/column frame on the exterior and wood stud in the interior. A crawl space exists under the whole of this area. Foundations are generally strip footings, with an exterior grade wall of the depth of the crawl space.

The remainder of the area is a single storey structure, with the roof constructed of diagonally installed shiplap on 2 x 8 joists at 16 inches on centre. The exterior and interior walls are generally of wood stud construction. A crawl space exists under the whole of this area. Foundations are strip footings.

There is neither data on the drawings nor a soils report available to indicate the allowable bearing capacity of the soil or whether the area is subject to liquefactions.

6.3 SEISMIC PATH

In its present condition, the building relies on the roofs and floors to act as horizontal diaphragms, transmitting forces to the vertical seismic elements, which are the masonry and wood frame walls. The walls in turn transfer the forces to the foundations.

SEISMIC DEFICIENCIES

A. TWO STOREY ASSEMBLY WING

1) The roof assembly, consisting of diagonally installed shiplap on joists, has the capacity to act as a diaphragm and does not require upgrading. However, the connection of the diaphragm to the walls (1/2 inch dia. bolts at 4 feet on centre) is inadequate. These connections will require upgrading, which can be done in combination with the wall upgrading as discussed in 2).

2) The east and west walls must act as shear walls and gravity load-carrying walls. These walls are constructed of unreinforced masonry, which the Code, because of historically poor performance of such walls will not allow to be used as unreinforced. These walls will have to be upgraded by the installation of a stud wall clad with 1/2 inch plywood to carry the lateral and gravity forces. Portions of the wall can be clad with gypsum board. Attachments at approximately 24 inches on centre each way should be installed to hold the masonry in place, to prevent it from falling out injuring passersby. The top of the wall should be attached to the diaphragm to allow

force transfer. The new walls should be installed within both stories of the structure.

Alternately, a new wall can be gunnited against the existing.

3) The north wall is constructed of unreinforced masonry and thus must be upgraded. To transfer the shear forces, three sections, two of 12 feet in length and one of 16 feet in length, must be installed. These may be stud walls similar in construction to that described in 2). A collector constructed of a steel angle or a wood ledger must be installed along the full length of the building to pick up the diaphragm loads and transfer them to the shear walls. To ensure that the masonry wall between the shear walls does not break out, it is recommended that a stud wall clad in gypsum board, with the masonry wall connected to it be installed. Alternately, a gunnited wall may be constructed for the shear wall or for the total wall.

It is usually uneconomical to install reinforcing and then grout up these reinforced voids in existing walls, but further assessments can be done at the time of actual mitigation.

Note that the glulam beams must be supported on the new wall system.

The shear walls and bracing walls should be used for the full height of the building.

4) The south wall should be upgraded in the same manner as the north wall.

5) It would be possible to delete the shear walld segment of the north and south walls between the 2nd floor and the foundation if the interior long walls along the east/west corridor are upgraded to shear walls. However, since the exterior walls' unreinforced masonry requires backing to prevent them from breaking out, it is recommended that the concepts of 2) and 3) be used. The final solution will depend upon what additional non-seismic upgrading would be anticipated.

6) The 2nd floor diaphragm, constructed of 1/2 inch plywood on diagonally installed shiplap on 2 x 8 joists meets Code requirements and needs not be upgraded except for the connection to the vertical elements. This would be done with the collector members.

7) Incorporate into the new shear wall required along the east wall of the assembly area at the 2nd floor stair level a horizontal beam to span across the two storey opening. This will reduce the effective height of the unbraced masonry wall.

8) An east/west reinforced masonry header exists across the opening for the stairs leading from the entrance hall to the 2nd floor. The single storey roof assembly attached to this header should be upgraded. The parallel joist should be bolted to the ceiling with 1/2 inch diameter inserts at 24 inches on centre. Block between the first three lines of joists with 2 x 10 blocking at 4 feet on centre. The blocking must be nailed securely to the joists.

B. TWO STOREY SOUTH SEGMENT (ROOF TO 2ND FLOOR)

1) The roof assembly consisting of diagonally installed shiplap on joists has the capacity to act as a diaphragm and does not require upgrading. The walls between the roof and the 2nd floor, as well as the connections between the walls and the diaphragm meet seismic requirements.

C. SINGLE STOREY AREA AND GROUND TO 2ND FLOOR OF TWO STOREY SOUTH SEGMENT

1) The single storey roof assembly area consisting of diagonally installed shiplap on joists and the 2nd floor of the two storey south segment, consisting of a layer of 1/2 inch plywood on diagonally installed shiplap on joists are marginal in their ability to act as shear walls. Because the capacity is marginal, and because of the irregular shape of the structure, it is recommended that additional vertical shear transfer elements be installed. This will usually be more economical than attempting to upgrade the diaphragm.

2) Approximately 65 feet of shear wall will be required in the north/south direction. The total can be made up of some new walls and the conversion of some existing by the installation of 1/2 inch plywood skins. The walls should be primarily in the general office/clinic area, with exact locations set to minimize torsional eccentricities. In addition, approximately 60 feet of existing north/south walls in the area of the two storey south wing should be converted to shear walls. Collector members consisting of steel angles or

ledgers should be installed at the top of wall level to transfer the forces to the shear walls.

3) Approximately 60 feet of shear wall will be required in the east/west direction. The walls should be primarily in the general office/clinic area, with the exact locations set to minimize torsional eccentricities. These can be either new walls or upgraded existing walls. No upgrading of walls will be required in the area of the two storey south wing. Collector members should be installed at the top of wall level to transfer the forces to the shear walls.

4) Single storey unreinforced walls occur on two sides of the east stairs in the two storey wing. Since these stairs are part of the egress route, it is recommended that an interior stud wall be installed on the inside faces, with the masonry wall attached to them. This will prevent the wall from falling out.

5) The single storey concrete frame forming the exterior structure of the two storey wing does not need upgrading. The masonry infill panels, though not reinforced, do not require upgrading since they do not have a structural

function and will not injure anyone if they break out.

CRAWL SPACE

A crawl space exists under the whole structure except for the two storey assembly area.

1) The major deficiency associated with the crawl space is the inadequacy of the connections (1/2 inch diameter bolts at 4 feet on centre) between the walls and the foundation to transfer the lateral forces. Several options are available for upgrade. The simplest, and most conservative is to install a steel angle to perimeter and interior concrete walls with 5/8 inch diameter bolts at approximately 6 feet on centre. The angle would be attached to the joists, or to added blocking, with lag screws at various spacings depending upon the level of force.

2) A new shear pony wall approximately 42 feet long will be required to run east/west in the crawl space under the general office area, possibly in line with the north wall of the two storey section. A strip footing will be required.

6.4 NON-STRUCTURAL ELEMENTS

There are few non-structural elements that must be considered. None are life threatening.

1) The brace for the hot water tank is inadequate and must be strengthened.

2) Lenses may fall out of the fluorescent light fixtures and should be clipped.

3) The shelving is generally well braced, but the contents will fall off. It is unlikely that this will interfere with the operations.

4) If any of the computers are required during an emergency, or if they store data that is not backed up, they should be attached to their supports, possibly by "velcro" pads glued to the desks. These desks should also be attached to the floor or wall. In addition, they should be located where other elements cannot fall on them.

5) The details of the window glazing is not available, but it is possible that some of the glazing will break out, primarily in the 2nd floor of the two storey south wing.

6.5 PARTIAL UPGRADING

Upgrading to a partial code requirement is not recommended where life safety is at risk or the structure serves a post disaster function and must remain operational after a disaster.

If partial upgrading is contemplated, it should be directed to works associated with the masonry walls in the assembly area and to maintain the integrity of exiting.

7.0 PA12 - CURLING RINK AND SKATING ARENA

7.1 GENERAL

The Curling Rink and Skating Arena is located on Ninth Avenue. The building was constructed in 1963 and was designed by Associated Engineering Services. One storey additions were later added at the southeast corner and at the rear of the Skating Arena.

The present function of this building is of course one of community recreation. In the event of an earthquake, however, it could serve as an emergency facility. For this reason the structure is assigned a high importance factor and corresponding high design forces.

The building is situated on high ground and would not be affected by a Tsunami.

7.2 CONSTRUCTION

The building is divided into two sections with the curling rink to the north and the skating arena to the south. At the front of the curling rink is a two storey plus basement region consisting of lobby, entrance way, washrooms, lounge, etc. A similar one storey region exists in front of the skating arena. To the rear of the curling rink is a one storey mechanical and electrical equipment area. To the rear of the skating arena is a one storey addition used primarily for equipment storage. At the southeast corner of the skating arena is a one storey area used for assembly purposes.

The overall construction of the building is a timber roof structure supported by a reinforced concrete skeleton of beams and columns. Unreinforced masonry forms the infill walls between concrete columns. The concrete columns bear on pad footings several feet below grade. The perimeter masonry walls are typically carried by concrete grade beams which span between the columns.

The roof of the Arena is made up of 2 x 6 tongue and groove decking on 6 x 12 purlins spanning between 17 foot deep timber bowstring trusses. The trusses are typically 22 feet on center matching the perimeter column spacing.

The roof of the curling rink consists of 2 x 6 tongue and groove decking on 6 x 12 purlins spanning between haunched 7 x 34.5 glulam beams. The beams are aligned with the trusses of the skating arena and bear on the concrete columns at the perimeter as well as a row of 6 inch diameter steel pipe columns down the center of the Curling Rink.

The roof structure of the rear mechanical and electrical areas is 3/4 inch plywood on 3 x 14 joists at 24 inches on center. The joists bear directly on the unreinforced masonry walls, which in turn rest on the concrete grade beams.

In the region in front of the Curling Rink, the roof consists of 3 x 6 tongue and groove decking on 5 1/4 x 13 1/2 inch glulam beams. The beams span east to west and bear on the concrete columns at the front of the building and at the front of the ice sheets, as well as an internal row of 3 inch diameter steel pipe columns. The second floor structure is virtually identical to the roof except that the beam size is 5 1/4 x 16 1/2 inches and the deck is 4 inches thick. The first floor, however, is a 5 inch thick reinforced concrete suspended slab supported by east-west spanning concrete beams. The beams once again span as the levels above to concrete columns bearing on pad footings below. Surrounding this entire section is a 10 inch thick concrete basement wall constructed integrally with the column pilasters.

The roof of the skating arena lobby is 3/4 inch plywood on 2 x 12 joists at 16 inches on center spanning between the front masonry wall, an internal built up wood beam, and the concrete beam in front of the rink itself. An internal row of 3 inch diameter steel pipe columns supports the built up 4 - 2 x 12 beam.

The southeast assembly wing consists of tongue and groove decking on cross purlins supported by glulam beams. These in turn are supported by masonry walls along the boundary with the Skating Arena and by columns in the interior and at the south boundary. These extend down to pad footings.

The one storey equipment area to the rear of the Skating Arena appears to be plywood on timber frame roof supported by the masonry walls surrounding it.

Ground floors are typically wire mesh reinforced slabs on grade.

7.3 SEISMIC PATH

In general, the building relies on the roof and floor systems to act as horizontal diaphragms, transferring lateral forces into the vertical seismic elements, primarily unreinforced masonry walls.

At grade level, the walls transfer the load into concrete grade beams which in turn deliver the load to the below-grade columns and finally, the foundations.

7.4 SEISMIC DEFICIENCIES AND UPGRADING

A. CURLING RINK

1) The roof assembly, consisting of 2 x 6 tongue and groove decking installed perpendicular to the supporting purlins, has a very limited capacity to act as a horizontal diaphragm and should not be considered as having any calculable seismic capacity. To compound this problem, the demand is high due to widely spaced supporting elements, ie, the perimeter walls of the Rink. To rectify the situation requires that some horizontal bracing be added or that a new plywood diaphragm be installed.

Since the main beams are exposed on the underside, and given the relatively large size of the roof area this region lends itself to a cross bracing solution. This would consist of horizontal steel "trusses" fastened to the underside of the purlins, and bolted to collector angles at the perimeter concrete beams. The "trusses" would be located adjacent to the perimeter walls of the rink. The difficulty with this solution, however, is the fact that the front wall of the rink itself is not able to resist lateral loads since it consists of just columns and windows. This means that the load must be transmitted right to the front wall of the curling lounge. This in turn means stripping of finishes in this region to install the bracing on the underside of the ceiling, reducing the ceiling height.

A diaphragm solution would consist of a double layer of 1/2 inch plywood nailed at 4 inches on center along boundaries. At the perimeter, the load would be transferred through the decking and existing ledgers and into the concrete beams, by way of drill-in anchor bolts.

The simplest way to install the new diaphragm is over top of the existing decking. This however requires stripping of the existing roof and as such would be most attractive if the roof was close to its life expectancy. Alternatively, the diaphragm could be installed on the underside of the purlins, however, this requires a great deal of additional blocking, as well as connection to each of the main glulam beams.

2) The non-load-bearing walls, being of unreinforced masonry are not adequate to carry seismic forces and are not permitted by Code to be used in areas

of high seismic activity. Furthermore, with a clear height/thickness ratio of as much as 30, these walls are susceptible to out-of-plane failure and are a hazard even as non-load-bearing walls. The simplest solution is to remove the 20 foot high walls entirely and replace them with either steel stud or, where required, new 10 inch thick reinforced masonry shear walls. Approximately 44 feet of shear wall is required along the north boundary and 110 feet along the boundary with the skating arena. In the north-south direction approximately 48 feet of shear wall is required at the rear of the Curling Rink and at the front face of the Curling Rink lobby.

Transferring the forces from the concrete beams at the roof level into the masonry walls and from the masonry walls into the grade beams requires that rebar dowels be drilled and grouted into the beams at approximately 24 inches on center.

3) At present, the only means of transferring loads from the grade beams to the foundations below is by way of the concrete columns. The concrete does not possess high ductility and the columns are inadequate to transfer the shear forces. As such, a short height of concrete wall matching the new shear walls

above must be cast below the grade beams and span between the columns.

4) At the locations of the new shear walls, it will be necessary to enlarge the pad footings to accomodate the increased overturning forces. The amount of enlargement will depend on what length of new wall is used as shear wall.

B. SKATING ARENA

1) The roof deck suffers from the same problem as that of the Curling Rink in that the tongue and groove decking has very little capacity to act as a diaphragm. To correct this requires the addition of a new diaphragm of double 1/2 inch plywood or a horizontal cross bracing system.

A horizontal cross bracing system could consist of 3 inch steel angle members fastened to the bottom of the bottom chord of each truss and connected to a collector angle at the four perimeter walls. The collector would then be bolted to the concrete beams at approximately 24 inches on center. A pair of diagonal angle members would connect to the midpoint of each truss and would extend to the perimeter collectors. Each pair of angles would also be connected to the other trusses at the intersection points.

For this system it would also be necessary to add extensive cross bracing in the vertical plane between trusses, in order to transfer the load from the actual deck level to the plane of the bottom chords. This would be accomplished by way of angle bracing, acting in tension only, lining up with the intersection points of the horizontal bracing and trusses below.

A diaphragm solution would be essentially the same here as for the Curling Rink. For this case it is necessary to ensure that the end walls forming the face of the trusses at the front and rear are upgraded to be able to act as vertical diaphragms.

2) The walls, being of unreinforced masonry up to 20 feet high should be removed and replaced with steel stud or reinforced masonry or, where required, with new masonry shear walls. Approximately 66 feet of masonry shear wall is required along the south boundary and a minimum of approximately 48 feet along the east and west walls of the Rink. The north wall is discussed in conjunction with the Curling Rink. The building would

not collapse if the non-shear wall portion was not upgraded. The purpose to upgrade would therefore be to protect people from falling debris.

3) The east wall (front wall) of the Skating Rink is discontinuous, in that at the ground floor level it is simply a row of columns with no infill masonry. This, of course, is necessary for viewing the ice surface from the front lobby. As such, shear walls would be very undesirable at this location. Here it is recommended that a minimum of two full bays (48 feet) of vertical steel cross bracing be added. This bracing would likely be in the form of wide flange sections bolted to the beams above and at ground floor level.

4) As with the Curling Rink it will be necessary to construct short concrete walls beneath the grade beams surrounding the Skating Arena, wherever shear walls or cross bracing exist above.

5) Pad footings will have to be enlarged beneath the columns at the ends of the shear walls and beneath cross bracing sections. The enlargement will depend on the length of shear wall used.

C. CURLING RINK LOBBY AREA

1) The roof diaphragm or bracing system and its connection to the supporting structure are handled in conjunction with the overall Curling Rink.

2) The second floor of this area, being tongue and groove decking, has minimal capacity to carry diaphragm loads. The possibilities here are to add a layer of 1/2 inch plywood and assume that only the extreme north and south walls of this region carry east-west lateral loads, or to upgrade the interior masonry walls between first and second floor levels to act as shear walls and leave the floor as is. In any case, upgrading will be required in connecting the floor to the masonry shear walls. This would likely be in the form of a steel perimeter collector angle lag bolted into the 4 inch deck and bolted to the masonry.

To make approximately 50 feet of the interior walls behave as shear walls would require the addition of either steel stud or wood stud with plywood panelling attached to the face of the masonry. Attachments should be spaced at approximately 24 inches on center.

3) The first floor structure, being of reinforced concrete has significant capacity to transfer loads as a diaphragm and, as such, does not require upgrading.

4) The north and south walls of this section are required to act as shear walls and therefore must be upgraded. Because they themselves are supported laterally by the various floors the clear height is not that large. For this reason it makes sense to clad them with a plywood shear panel attached as per 2). This avoids the more costly alternative of removing the unreinforced masonry for this case.

5) The east or front wall is discussed in conjunction with the overall Curling Rink.

6) The west wall of this region (ie., the wall separating the lobby from the Curling Rink) must be able to carry nominal lateral loads from the first and second floors. This means that it will be necessary to add one bay of steel cross bracing or approximately 24 feet of shear wall between the first and second floors in front of the ice sheets. Bracing would seem to be the better choice for viewing the ice from the lobby.

7) Below grade the concrete basement walls extend right down to the foundations in this area, so no additional walls are required below grade.

8) Pad footings will require enlargement beneath shear walls and braced regions.

D. SKATING ARENA LOBBY AREA

1) Assuming a plywood deck, this roof is adequate to carry the diaphragm forces to the various walls. Once again, however, the unreinforced masonry walls are not adequate to carry lateral loads. These low walls can be reinforced with plywood on stud panels and converted into shear walls. The north and south walls of this region, as well as at least one internal wall should be treated in this manner.

In the north-south direction approximately 24 feet of shear wall is required at the front of the building. At the junction with the Skating Arena, lateral resistance is provided by the lateral bracing as discussed in B. 3).

2) Below-grade walls and possibly increased footing sizes beneath shear walls will be required, as previously discussed.

E. REAR MECHANICAL, ELECTRICAL, EQUIPMENT AREAS

1) The assumed plywood roof diaphragms in these areas are adequate to transfer forces to the supporting walls. The walls are unreinforced masonry and therefore must be replaced or strengthened. It is recommended that the walls surrounding these areas be upgraded by affixing studs and sheeting with plywood. Because of the relatively small size of these sections, no special upgrading is required of the foundations.

F. SOUTH ASSEMBLY WING

1) The roof diaphragm is constructed of 3 inch tongue and groove decking, inadequate to transfer horizontal forces to the supporting walls and must be upgraded by the use of 1/2 inch plywood sheets over 50% of the area. A collector will be required at the junction of the roof and wall to transfer forces to the shear walls. The unreinforced masonry walls must be upgraded

by either replacing or strengthening. As with other areas of low walls, the simplest method of improvement is to add plywood on stud shear panels.

2) The roof beams spanning north-south apparently bear at their north end on the unreinforced masonry walls adjoining the Skating Arena and lobby. If, as discussed in B. 2), these walls are replaced, then new support details will be required for the glulam beams. Alternatively, new post supports can be added adjoining the wall.

3) The south wall of this wing is virtually devoid of any lateral resistance since it is almost entirely windows. To add lateral resisting capacity to this wall a new plywood shear wall roughly 16 feet in length should be constructed, closing off some window space. The wall should be situated between existing column locations in order to minimize any foundation work.

7.5 NON-STRUCTURAL ELEMENTS

1) Of primary concern among the non-structural elements are some of the brine tanks and other mechanical equipment located in the rear portions of the building. It is

important that all tanks be adequately restrained by way of tying with cable or steel banding or other means to the permanent structure, which resists lateral loads. Tying to unreinforced masonry walls is not adequate in this regard. Note that electrical panels are attached to non-load-bearing, unreinforced masonry walls.

2) Inside the building, hanging lamps are a concern wherever their swinging motion may lead to impact with nearby structural elements. Lamps should be tied with heavy wire or light cable in 3 directions to avoid damage.

3) All non-load-bearing masonry, including interior partition walls and chimneys, are susceptible to damage and brittle failure. To minimize this aspect, chimneys should be restrained above the roof by tying to the structure. Interior partitions should be adequately fastened to the underside of the floor or roof structure above. It is generally preferred to replace the unreinforced non-load-bearing masonry walls with an alternate material.

4) Heaters and baffles in the Skating Arena should be braced.

7.6 PARTIAL UPGRADING

Upgrading to a partial Code requirement is not recommended where life safety is at risk or the structure serves a post disaster function and must remain operational after a disaster.

8.0 PA13 - ECHO CENTRE

8.1 GENERAL

The Echo Centre, located at 4255 Wallace Street, was constructed in several stages. The original construction in 1966 consisted of the Aquatic Centre and the Community Centre. Subsequently, the Museum and Library were constructed in 1971. A Craft Centre was added to the Community Centre and a sauna added to the Aquatic Centre in 1973. The most recent expansion was an extension to the Museum in 1981. The architects for all the works were Carlberg Jackson Partners, Architects.

The location of the structure is such that it would not be affected by a Tsunami.

Since several additions were constructed, without consideration of the seismic action of the total structure, one can expect that even with upgrading, some damage could occur at the junctions unless a total upgrading is contemplated.

8.2 1981 MUSEUM EXPANSION

8.2.1 CONSTRUCTION

The roof of the 6,000 square foot expansion is constructed of 3 inch tongue and groove decking upon which is attached a skin of 3/8 inch plywood. This assembly is supported on glulam beams. The glulams are either supported on steel pipe columns or on reinforced masonry walls. Some glulams are attached to the unreinforced masonry east wall of the existing Museum. The perimeter walls are of reinforced masonry.

8.2.2 SEISMIC PATH

In its present condition the structure relies on the 3/8 inch plywood sheet to act as a diaphragm, transmitting forces to the vertical seismic elements. These vertical elements are reinforced masonry walls.

8.2.3 SEISMIC DEFICIENCIES

1) The 3/8 inch plywood diaphragm has the capacity to behave as the diaphragm if its nailing pattern is satisfactory. The requirement is that the nails be spaced at 3 inches on centre along its edges. It is expected that this is not the case and the roofing should be removed for a 30 foot distance at the north and south ends and that the correct nailing pattern be applied. The remainder of the areas would function with nominal nailing, which is what probably exists.

   Ledgers and anchor bolts used to transfer the lateral forces to the shear walls appear adequate.

2) The west end of the addition is attached to an unreinforced masonry wall belonging to the original Museum construction. Since the wall is load-bearing and its failure will cause structural collapse, it is recommended that columns be installed at the locations of the glulams to carry loads to the foundation.

   8.3.3, item 3) in SEISMIC DEFICIENCIES requires that 16 feet of this wall be removed and replaced by other materials capable of acting as a shear wall.

3) The southern end of the west wall is load-bearing and as such must be braced by the installation of an adjacent stud wall, to which the masonry must be attached. This prevents failure from out-of-plane forces.

8.2.4 PARTIAL UPGRADING

This portion of the Echo Centre comes very close to meeting the present Code and as such, it is not reasonable to do a partial upgrade.

It is recommended that the primary item to upgrade would be item 2).

8.3 1971 MUSEUM AND LIBRARY ADDITION

8.3.1 CONSTRUCTION

The roof of this addition is constructed of 3 inch tongue and groove decking supported by glulam beams. The glulams are supported on either steel columns or reinforced masonry pilasters. The perimeter walls are of unreinforced masonry, some in long continuous sections and some broken by reinforced pilasters at 10 feet on centre.

8.3.2 SEISMIC PATH

In its present condition, the structure relies on the wood roof deck to act as a diaphragm to transmit forces to the vertical seismic elements. These vertical elements are unreinforced masonry walls.

8.3.3 SEISMIC DEFICIENCIES

1) The existing roof diaphragm, consisting of 3 inch tongue and groove decking, cannot be considered as having any calculable diaphragm capacity and as such, must be upgraded. A 1/2 inch sheet of plywood, covering approximately 60% of the roof area, must be added and adequately nailed to the decking.

2) Collectors consisting of steel angles or ledgers must be added at the level of the underside of the roofing to transfer the forces to the new shear walls.

3) The use of unreinforced masonry as vertical shear transfer elements is not acceptable by Codes and therefore alternative means of transfer must be provided.

For the easterly wall of this segment (westerly wall of the 1981 Museum expansion), in addition to the items discussed in the previous section on the Museum expansion, it is necessary to replace approximately a 16 foot segment of the wall with a reinforced masonry or a reinforced concrete wall.

The grid lines referred to in the following section refer to those noted on the 1971 drawings.

For the wall on line 8, which is the east wall of the Library, it is necessary to remove approximately 50 feet of the wood finish and replace it with a 1/2 inch sheet of plywood to create a shear wall. It is expected that the existing plywood is unsatisfactory because of the probable inadequate nailing patterns used at the time.

For the wall on line 4, which is the west wall of the Library, it is necessary to remove approximately 20 feet of the wall and replace it with a reinforced masonry or reinforced concrete shear wall. This should be done in a length where no pilasters exist, since such sections cannot carry the concentrated loads of the glulams and also have to be upgraded for other reasons. The reinforced pilasters can carry the glulams and thus do not require upgrading.

For line M, which is the south wall of the Museum, there is no need to upgrade for shear forces since the reinforced pilasters can transfer these forces as well as the forces associated with the tributary walls that span onto them.

For wall line L, which is the wall between the Museum and the Museum storage, it is necessary to remove approximately 10 feet of wall and replace it with a reinforced masonry or reinforced concrete shear wall. The steel columns embedded in the masonry can handle the forces associated with the walls between. However, there does not appear to be any method of transfer of the forces to the columns (continuous ladder reinforcement, etc.) and it is expected that they will break apart. This will not cause structural collapse, but may injure passersby.

For wall line G, which is the north wall of the Museum storage, it is

3) The use of unreinforced masonry as vertical shear transfer elements is not acceptable by Codes and therefore alternative means of transfer must be provided.

For the easterly wall of this segment (westerly wall of the 1981 Museum expansion), in addition to the items discussed in the previous section on the Museum expansion, it is necessary to replace approximately a 16 foot segment of the wall with a reinforced masonry or a reinforced concrete wall.

The grid lines referred to in the following section refer to those noted on the 1971 drawings.

For the wall on line 8, which is the east wall of the Library, it is necessary to remove approximately 50 feet of the wood finish and replace it with a 1/2 inch sheet of plywood to create a shear wall. It is expected that the existing plywood is unsatisfactory because of the probable inadequate nailing patterns used at the time.

For the wall on line 4, which is the west wall of the Library, it is necessary to remove approximately 20 feet of the wall and replace it with a reinforced masonry or reinforced concrete shear wall. This should be done in a length where no pilasters exist, since such sections cannot carry the concentrated loads of the glulams and also have to be upgraded for other reasons. The reinforced pilasters can carry the glulams and thus do not require upgrading.

For line M, which is the south wall of the Museum, there is no need to upgrade for shear forces since the reinforced pilasters can transfer these forces as well as the forces associated with the tributary walls that span onto them.

For wall line L, which is the wall between the Museum and the Museum storage, it is necessary to remove approximately 10 feet of wall and replace it with a reinforced masonry or reinforced concrete shear wall. The steel columns embedded in the masonry can handle the forces associated with the walls between. However, there does not appear to be any method of transfer of the forces to the columns (continuous ladder reinforcement, etc.) and it is expected that they will break apart. This will not cause structural collapse, but may injure passersby.

For wall line G, which is the north wall of the Museum storage, it is necessary to remove approximately 10 feet of wall and replace it with a reinforced masonry or reinforced concrete shear wall.

For wall line A, which is the north wall of the Library, the same upgrading as for line M is required.

Walls C and F, which are the north and south walls of the kitchen are load-bearing unreinforced masonry walls and as such must be upgraded. They should either be replaced by a stud or reinforced masonry wall or upgraded by the installation of a beam/column system or an interior stud wall to which the masonry could be attached.

Wall line 1, which is the westerly wall between the kitchen and the Community Centre, requires upgrading. It is recommended that columns be installed to support the roof beam. It is recommended also that approximately 10 feet of shear wall, probably constructed of studs with a plywood skin be installed. This will tend to keep this addition stable and separate from the Community Centre.

4) The south covered walkway between the addition and the Community Centre

should have an independent support at the Community Centre end to ensure that the complex footprint of the facility does not cause unusual forces to tear it down. Note that this should be considered as an egress and thus should remain undamaged.

8.3.4 PARTIAL UPGRADING

Partial upgrading of this section is not feasible. Upgrading to a partial Code requirement is not recommended where life safety is at risk or the structure serves a post disaster function and must remain operational after a disaster.

8.4 1966 COMMUNITY CENTRE AND 1973 STAGE ADDITION

8.4.1 CONSTRUCTION

The roof of the centre is constructed of 3 inch tongue and groove decking supported by glulam beams. The glulams are supported on steel columns. The perimeter walls are of unreinforced masonry with masonry pilasters at each steel column location. The masonry pilasters cannot be considered as reinforced. A crawl space occurs under the club rooms and kitchen area while the remainder is covered by a slab-on-grade. The crawl space is constructed of 3/4 inch plywood on solid laminated 2 x 4 lumber, supported on timber beams.

The roof of the 1973 addition is of similar construction to that of the community centre. A basement exists under the whole addition, with the floor constructed of 1/2 inch plywood on 4 inch tongue and groove decking on glulam beams.

8.4.2 SEISMIC PATH

In its present condition, the structure relies on 3 inch decking to act as a diaphragm, transmitting forces to the vertical seismic elements. These vertical elements are unreinforced masonry walls.

8.4.3 SEISMIC DEFICIENCIES

The grid lines referred to in this section refer to those noted on the 1966 and the 1973 drawings or the facility.

1) The existing roof diaphragm, consisting of 3 inch tongue and groove decking, cannot be considered as having any calculable diaphragm capacity, and as such, must be upgraded. A 1/2 inch sheet of plywood, covering approximately the following areas must be added:

- a 24' wide strip, full length along lines 21 and 33
- an 8' wide strip, full length along line A
- a 24' wide strip, full length along line E
- a 24' wide strip, full length along line O.

Alternate locations would be acceptable, with the above noted to describe the work required.

2) Collectors consisting of steel angles or ledgers must be added at the level of the underside of the roofing to transfer the forces to the new shear walls.

3) The use of unreinforced masonry as vertical shear transfer elements is not acceptable by Codes and therefore alternative means of transfer must be provided.

For the wall on line 33, which is the easterly wall of the Centre, it is necessary to remove approximately 40 feet of the unreinforced masonry and replace it with a reinforced masonry or reinforced concrete shear wall.

The wall on line 21, which is the westerly wall of the Centre and includes the 1973 addition, should have the northern 40 feet removed and replaced with a reinforced masonry or a reinforced concrete shear wall. The 1973 work has made this segment of the wall extremely susceptible to seismic damage.

For the wall on line E, which separates the club rooms from the remainder of the Centre, it is necessary to upgrade approximately 65 feet of the stud wall to allow it to perform as a shear wall. This would be carried out by adding blocking and installing a 1/2 inch plywood sheet. The length of shear wall would be reduced if plywood is added to both sides of the existing wall. By upgrading this wall, there will be no requirement to close up any of the windows on line A.

The upgrading must be carried through the crawl space to the foundation.

For the wall on line 0, which is the southernmost wall of the Centre, it is necessary to remove approximately 10 feet of the wall and replace it with reinforced masonry or reinforced concrete.

4) The failure of unreinforced masonry partition walls will not cause collapse of the structure, but may injure occupants. Such walls, particularly along exit routes, should be considered for upgrading.

5) Because of the uncertainty of the nailing pattern of the plywood over the crawl spaces, it is recommended that an additional plywood diaphragm wall be installed within the crawl space. This wall would be approximately 40 feet long and run in the north/south direction.

Alternately, the flooring can be removed and the 3/8 inch plywood diaphragm improved by upgrading the nailing pattern.

6) The number of anchor bolts connecting the beams on lines 33 and 25 to the grouted masonry in the crawl space are inadequate and should be increased.

7) A collector will be required along line E in the crawl space to transfer lateral forces from the wood deck into the concrete slab. A similar detail may be

required to connect the deck to the block wall.

8.4.4 PARTIAL UPGRADING

Partial upgrading of this section is not feasible. Upgrading to a partial Code requirement is not recommended where life safety is a risk or the structure serves a post disaster function and must remain operational after a disaster. If necessary, the floor diaphragm could be delayed.

8.5 1966 AQUATIC CENTRE AND 1973 SAUNA ADDITION

8.5.1 CONSTRUCTION

The building is a single storey structure with two roof levels, a lower level over the change areas and a high level over the pool.

The roof of the change area consists of 3 inch tongue and groove decking supported by glulam beams. The beams are supported on steel columns. The majority of the perimeter and interior walls are non-load-bearing and constructed of unreinforced masonry.

The roof of the pool area consists of 3 inch tongue and groove decking supported by glulam beams. The glulams are supported on perimeter wide flange columns. The exterior walls are constructed of unreinforced masonry with pilasters at each column line. The drawings indicate that ladder type reinforcement exists at every 2nd block course.

The roof over the sauna area consists of 3 inch tongue and groove decking supported by glulam beams. The glulams are supported on steel pipe columns embedded in the new wall and to brackets welded to existing columns on the north side of the pool.

8.5.2 SEISMIC PATH

In its present condition, the structure relies on the wood roof decks to act as diaphragms to transmit forces to the vertical seismic elements. These vertical elements are unreinforced masonry walls. The Sauna depends upon the north pool wall for part of its seismic support.

8.5.3 SEISMIC DEFICIENCIES

The grid lines referred to in the following section refer to those noted on the 1966 drawings of the Aquatic Centre and the 1973 drawings of the Sauna addition.

1) The existing roof diaphragms, consisting of 3 inch tongue and groove decking, cannot be considered as having any calculable diaphragm capacity, and as such, must be upgraded. A 1/2 inch sheet of plywood covering approximately the following areas must be added:

   - an 8' wide strip, full length along line 21
   - an 8' wide strip, full length along line 16
   - a 20' wide strip, full length along line A
   - a 20' wide strip, full length along line P
   - the total area of the pool segment, with special framing around the raised area above the diving boards
   - the full area of the Sauna addition.

2) Collectors consisting of steel angles or ledgers must be added at the level of the underside of the roof to transfer the forces to the vertical shear walls.

3) The use of unreinforced masonry as vertical shear transfer elements is not acceptable by Codes and therefore, alternative means of transfer must be provided.

The wall along lne 21, the common wall between the Aquatic Centre and the Community Centre has been discussed in Section C.

Upgrade the wall along line P, the south end of the change area to a shear wall by the installation of a 1/2 inch plywood skin.
Upgrade approximately 16 feet of wall on line A or C to a shear wall by removing the unreinforced masonry and replacing it with reinforced masonry or reinforced concrete.

For walls on lines 1 and 16, which are the west and east walls of the pool area, it is necessary to replace approximately 20 feet of each wall with a reinforced masonry or reinforced concrete shear wall. Proper transfer from the diaphragm to the wall must be included.

For walls on lines C and 0, which are the north and south walls of the pool area, it is necessary to replace approximately 20 feet of each wall with a reinforced masonry or reinforced concrete shear wall.

4) The unreinforced masonry walls around the perimeter of the pool area are non-load-bearing and as such would not cause a collapse of structure if they failed. The walls are reinforced horizontally with ladder type reinforcement at every 2nd course. The walls will span horizontally to the steel columns, which are capable of taking these loads and transferring them to the foundations and to the diaphragm. However, the walls do not meet Code requirements and it is expected that they will be damaged during a Code quake.

Since the structure is to be a post disaster facility, it is recommended that at some stage, consideration be given to bracing or replacing these infill walls.

The walls around and within the change areas and the sauna should be considered similarly, though because of their height, there is probably less risk. The risk would be reduced further if the tops of the walls were clipped to the deck with angles spaced approximately 6 feet on centre.

The wall on line 2, which is the west end of the Sauna addition, should be replaced by a reinforced masonry or reinforced concrete shear wall.

The most westerly 10 foot section of the Sauna addition wall along line A should be replaced by a reinforced masonry or reinforced concrete shear wall.

5) Because of the non-uniform nature of the pool, it is expected that it will crack. Reinforcing details are not available to confirm this. Failure of the pool, however, is not a risk to life.

8.5.4 PARTIAL UPGRADING

Upgrading to a partial Code requirement is not recommended where life safety is at risk or the structure serves a post disaster function and must remain operational after a disaster. The Sauna area could possibly be delayed for upgrade.

8.6 NON-STRUCTURAL ELEMENTS

1) Ensure that all the loose equipment in the filter room is connected to the floor or to a wall that has been upgraded to prevent it from falling out. Escaping chlorine

may make the facility inoperable for a period of time.

2) Ensure that all walls that support electrical panels and controls are upgraded to prevent them from falling out.

3) All lighting over the pool area must be braced to ensure that the units do not loosen and fall into the pool.

4) Ensure that all the equipment in the first aid room is stored in cabinets that are attached to adequate supports and the cupboard doors do not open easily. Walls around the first aid room should be upgraded to ensure that they do not fall out.

5) Lockers in the change rooms and elsewhere should be connected to the floor and other adequate supports.

6) The Library shelf units should be braced and interconnected.

7) All units in the furnace room including any emergency generators should be adequately anchored. Units on vibration isolators (springs) should have buffers installed to limit movements.

8) Large glass panels, particularly at entrances and along corridors (between the Library and the Community Centre) will probably be broken, potentially injuring those nearby. It is suggested that they be clad on the interior with security film.

9.0 PA14 - PUBLIC SAFETY BUILDING

9.1 GENERAL

The Public Safety Building, located at 4110 6th Avenue, was constructed in 1966, with a small addition in 1971. A new roof assembly that changed the roof profile was added in 1984. The architectural design of the original structure as well as the additions were by Carlberg Jackson Partners, Architects.

This structure houses the R.C.M.P. Detachment for the area and as such will probably play a strategic function in the event of an earthquake. It is not certain whether the location of the structure would be susceptible to a Tsunami.

9.2 CONSTRUCTION

The roof of the structure is constructed of 3 inch tongue and groove decking supported by glulam beams. Around the perimeter, these glulam beams are supported on glulam trim beams, which in turn are supported on reinforced masonry pilasters. In the interior, the beams are supported on unreinforced masonry walls.

There is a partial basement. The structure over this section is a suspended, cast-in-place concrete slab. This slab is supported partially on cast-in-place concrete walls, and partially on unreinforced masonry walls. The remainder of the ground floor is a concrete slab-on-grade.

In 1984 a new 24 gauge metal roof was installed on pony walls at 6 feet on centre. The pony walls are supported directly on the wood decking. It appears that the purpose was to provide some additional architectural shape to the structure.

9.3 SEISMIC PATH

In its present condition, the roof relies on the wood roof deck and the concrete floor to act as horizontal diaphragms, transmitting forces to the vertical seismic elements, which are unreinforced masonry walls between the ground floor and the roof and reinforced concrete walls in the basement.

9.4 SEISMIC DEFICIENCIES

A. STRUCTURAL

1) The 24 gauge new metal roof cannot be considered to act as a diaphragm because of its minimal thickness, inadequate seismic connection details and locations of pony walls. The 3 inch tongue and groove decking must be considered as the lateral transfer element. There is no plywood skin attached to the decking. This type of decking has no calculable diaphragm capacity and as such must be upgraded.

The upgrading can be accomplished by the addition of 1/2 inch plywood nailed to the decking. A strip 8 feet wide around the perimeter of the building as well as on lines "K" and "6" will be required. This material may be attached on either the upper surface of the undersurface. Attachment to the upper surface has been made difficult by the construction of the new roof. However, if it is to be installed at that location then, either the insulation must be removed over the whole strip, or the insulation must be removed to accommodate a 2 foot by 2 foot grid of 2 x 4 sleepers.

2) The connection between the diaphragm and the edge members is inadequate and must be upgraded. This can be accomplished by the addition of approximately 500 feet of collectors. These collectors may be either steel angles or wood ledgers, that carry the forces to the shear walls.

3) To ensure that the weight of the metal deck and the required 25% of the snow load can be transferred to the wood deck, ensure that the pony walls are capable of transmitting approximately 125 pounds per linear foot in shear through their connection to the wood deck. In addition, ensure that sections of each pony wall, particularly the high walls, are cross braced with standard metal straps.

4) The existing reinforced masonry perimeter pilasters are acceptable to be used for transferring the lateral forces from the walls to the diaphragms. However, no data is available on the connection of the pilasters to the glulam assemblies above. This connection must do the final transfer. It is recommended that it be assumed that some minor upgrade work will be required, the extent to be determined when and if mitigation work is to take place.

5) The unreinforced masonry cannot be used as the vertical shear transfer element without upgrading. Approximately 50 feet of upgraded wall will be required in each direction. It is suggested that the following sections of walls be upgraded in approximately 8 foot widths:

East/west walls

Line A	5 - 6
Line K	1 - 2
Line Ex	11 - 12
Line L	11 - 12
Line L	3A - 3A+8'
Line Q	3A - 3A+8'

North/south walls

Line 1	A - B
Line 6	F - G
Line 12	Kx - L
Line 10A	N - 0
Line 3A	P - Q

8' in the 1970 addition

These walls may be upgraded with 4 inches of gunnite, or reinforced concrete. The existing masonry wall must be attached to the new wall with attachments at 24 inches on centre each way. Alternately, the section of wall may be removed and replaced with concrete or reinforced masonry.

6) The unreinforced masonry exterior perimeter walls between the pilasters can most likely transfer their lateral forces to the pilasters, and as such need not be upgraded. There may be some damage to these walls but there will not be a structural collapse. Though this does not meet Code, it does meet the intent. Operations should review these walls to determine whether some must not suffer any damage. In this case a backup wall as discussed later should be installed. The connection must be upgraded as noted in 4).

7) The interior load bearing walls are unreinforced, and as such do not meet Code requirements. Damage to these walls can cause structural collapse and thus they must be upgraded. Options include:

- the installation of a steel stud wall on one side to brace the masonry wall. The wall must be attached with nserts at 24 inches on centre each way.
- saw cutting out the block wall and install concrete posts at 4 feet on centre.
- break out or saw cut out strips of block and fill with grout and reinforcement. Spacing to be at 4 feet on centre.
- shore the structure and replace the wall with an appropriate alternate

material.

Note that the reinforcing need not be installed directly under the concentrated loads because it is expected that a number of concrete fill blocks exist under each load.

The reinforcing must be dowelled into the slab.

8) The suspended concrete floor has the capacity to act as a diaphragm.

9) The existing reinforced concrete walls between the basement floor and the ground floor meet the requirements of shear walls and are adequate in number and location.

10) The unreinforced masonry bearing walls between the basement and the ground floor must be upgraded because damage by the out-of-plane forces may cause collapse. They may be upgraded by:

- drilling holes at 4 feet on centre through the ground floor slab and installing reinforcing bars and grout.

- replacing by a reinforced masonry or cast-in-place concrete wall.
- add a gunnited wall.

The reinforcing must be dowelled into the slab or foundation.

9.5 NON-STRUCTURAL ELEMENTS

There are minimal non-structural elements that must be considered. Those that should be upgraded are not life threatening, but affect the function of the building immediately after an earthquake.

1) The Communications Room must be carefully addressed. The computers should be attached to the tables and the tables attached to the floor. The equipment must be located so that falling material from the ceiling or the bookcases does not damage the communication components.

2) Shelving is generally unbraced, and if data located on these shelves is required for use immediately after a quake, then the shelves should be braced and the contents prevented from falling.

material.

Note that the reinforcing need not be installed directly under the concentrated loads because it is expected that a number of concrete fill blocks exist under each load.

The reinforcing must be dowelled into the slab.

8) The suspended concrete floor has the capacity to act as a diaphragm.

9) The existing reinforced concrete walls between the basement floor and the ground floor meet the requirements of shear walls and are adequate in number and location.

10) The unreinforced masonry bearing walls between the basement and the ground floor must be upgraded because damage by the out-of-plane forces may cause collapse. They may be upgraded by:

- drilling holes at 4 feet on centre through the ground floor slab and installing reinforcing bars and grout.

- replacing by a reinforced masonry or cast-in-place concrete wall.
- add a gunnited wall.

The reinforcing must be dowelled into the slab or foundation.

9.5 NON-STRUCTURAL ELEMENTS

There are minimal non-structural elements that must be considered. Those that should be upgraded are not life threatening, but affect the function of the building immediately after an earthquake.

1) The Communications Room must be carefully addressed. The computers should be attached to the tables and the tables attached to the floor. The equipment must be located so that falling material from the ceiling or the bookcases does not damage the communication components.

2) Shelving is generally unbraced, and if data located on these shelves is required for use immediately after a quake, then the shelves should be braced and the contents prevented from falling.

3) The hot water tank should be braced.

4) It should be confirmed that the emergency generator, located in an enclosed metal structure outside the building, is adequately connected to the foundation to prevent its movement. If mounted on spring isolation pads, then snubbers should be added to control the movement.

9.6 PARTIAL UPGRADING

Upgrading to meet partial Code requirements is not recommended where life safety is at risk or the structure serves a post disaster function and must remain operational after a disaster.

If partial upgrading is contemplated, it should be directed to works associated with the unreinforced masonry walls and to the protection of the communications room.