



STRUCTURAL REPORT ON

THE QUEEN MARY BUILDING
AT ROYAL PRINCE ALFRED HOSPITAL

FOR
GALE PLANNING GROUP

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REPORT HISTORY

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1.0 INTRODUCTION

MPN Group was engaged by Gale Planning Group to carry out a structural appraisal of a desired redevelopment option for the Queen Mary Building at the Royal Prince Alfred Hospital in Camperdown. The building is situated between Brodie Street to the North and Grose Street to the South. It is approximately 55 years old and was originally constructed for Nurses Accommodation. It consists of thirteen stories with a "U" shaped floor plan from second floor level through to roof level (refer to Appendices A and B).

The original building structure was designed by Julius, Poole & Gibson Consulting Engineers. The original architect was Stephenson & Turner Architects. MPN have conducted an extensive search and managed to obtain a copy of the original structural and architectural drawings that were thought to no longer exist. A selection of the drawings are included in Appendix B.

2.0 BUILDING STRUCTURE DESCRIPTION

The building consists of a concrete encased steel framed structure. The main structural elements of the building are as follows:

- a. Reinforced concrete pad footings of various sizes (the majority with concrete pedestals).
- b. Concrete encased steel columns of various sizes.
- c. Concrete encased steel primary floor beams of various sizes on the column lines in one direction.
- d. Concrete encased steel secondary floor beams of various sizes on the column lines in the orthogonal direction.
- e. Insitu reinforced concrete rib floor beams (generally 12" deep x 6" wide) that span between the primary floor beams.
- f. Reinforced concrete floor slabs (generally 5" thick) that span between the rib beams.
- g. Insitu reinforced concrete cantilever balcony slabs of various thicknesses.

The internal partition walls generally consist of non-loadbearing masonry. The external façade walls generally consist of non-loadbearing cavity masonry walls supported by floor beams at each level.

The lateral resisting system of the building is not immediately apparent. A modern building of this height and size would usually have a reinforced concrete lift shaft and/or stair shafts to provide lateral stability. Buildings constructed at the time (early 1950's) were not designed for earthquake loading and the design approach for wind loading would most likely have been fairly "primitive" (an Australian Wind Code was first published in 1952).

The lift and stair cores in this building appear to consist of non-loadbearing masonry. The lateral stability of the building is most likely provided by frame action between the steel floor beams and steel columns. For frame action to be effective the connections between the columns and beams must provide some rigidity. The connections between steel beams and columns as documented on the Julius, Poole & Gibson drawings are ambiguous. Further on-site investigation of the connections is required. It should be noted that the columns and



beams are concrete encased. It is likely that the concrete encasement would provide additional strength and rigidity to both the steel members and their connections. For this concrete encasement to provide additional strength some composite action is required between the steel and concrete. To ultimately confirm the lateral stability of the building, a detailed lateral computer analysis of the building structure would need to be performed. ?

3.0 PROPOSED ALTERATIONS

The proposed alterations and additions to the Queen Mary Building consist of some or all of the following:

- a. Conversion from a nurses home to residential apartments.
- b. Complete stripping of the building back to its basic structure.
- c. Addition of floor area to the south side of the building by way of infill on multiple levels to create larger floor plates.
- d. Addition of terraces/ balconies to northern face- where there are none currently.
- e. Provision of internal stairs between levels where required to create two storey apartments.
- f. Conversion of basement level to parking.

4.0 STRUCTURAL IMPLICATIONS OF THE PROPOSED ALTERATIONS

The structural implications of the proposed alterations and additions described in Section 3 above are as follows:

- a. The design loads for the floor structure are not specified on the original structural drawings. The position of the new non-loadbearing partition walls for a residential conversion would inevitably be different to the position of the existing walls. Consequently, the existing floor structure will need to be analysed to determine the structural implications of the new partition wall layout. The choice of partition (e.g. masonry, lightweight, etc.) and its arrangement will have an influence on the structural adequacy and performance of the floor structure. Any deficiencies in the structure (only expected to be local if any) would require structural enhancements/modifications.

The design load for balconies is typically higher than the design load of the area providing access to the balconies. The existing balconies will need to be analysed to determine their structural adequacy. If the balconies are found to be structurally inadequate, it may be prudent to demolish the balconies and replace them with new structure. The new balcony structure could consist of new slabs connected to the existing floor structure with new columns on all levels located near the perimeter of the balcony. Alternatively, new cantilever steel beams could be connected to the existing building to support new balcony slabs. The second option would locally restrict the available headroom and be more difficult to construct.

The vertical capacity of the columns and footings will also have to be assessed if the building use was changed to residential apartments. However, there would most likely be a reduction in the load on the footings and columns because the residential apartment



sizes would probably be larger and therefore the wall density smaller than the current nurses quarters.

- b. The proposed stripping of the building back to its basic structure is structurally feasible. As mentioned in Section 2 the existing internal partition walls and external façade walls are generally non-loadbearing. Whilst the non-loadbearing masonry walls are usually not relied upon to provide lateral strength or stiffness to a framed structure, their physical presence does add to the lateral strength and stiffness of the building. Further, the masonry walls can become partially loadbearing if the gap provided between the top of the walls and the floor structure is insufficient to cater for the floor deflection. Consequently, the removal of the masonry walls may cause some movement in the structure both vertically and laterally. However, this would not be cause for structural concern. A lateral analysis will need to be undertaken to determine the behaviour of the building under wind and earthquake loads.

Additional requirements that the structure needs to comply with are fire and durability. The fire resistance level of reinforced concrete structural members is reliant on the amount of concrete cover to the reinforcing steel as stipulated in Australian Standard AS 3600 Concrete Structure Design Code. Cover to the reinforcing steel is specified on the original structural drawings and this as well as the concrete thickness can be used to assess the fire resistance level of the reinforced concrete floor elements.

The fire resistance level of structural steel members is reliant on the thickness of concrete encasement around the structural steel as stipulated in the BCA. The properties of the structural steel section are also used to determine their fire resistance period. The concrete cover to the structural steel floor beams and steel columns is not specified on the original structural drawings. Consequently, to assess their fire resistance level, the concrete encasement around the steel floor beams and steel columns will need to be removed locally on site in various locations throughout the building and the thickness and also the reinforcement in the encasements measured.

The durability of a reinforced concrete element is evaluated by considering concrete strength and concrete cover as stipulated in Australian Standard AS 3600 Concrete Structure Design Code. The concrete strength is not specified on the original structural drawings. Testing of the floor concrete will need to be carried out to determine the concrete strength in various locations throughout the building. The internal areas will most likely comply with the current AS 3600 standard. It is highly unlikely that the external areas (e.g. balconies) will be found to comply with the current AS 3600 standard. The durability of external areas that are found to be non-compliant can be enhanced by the application of protective surface coatings. Alternatively, the balconies could be demolished and replaced with new structure as mentioned above in section a.

- c. The proposed addition of structure to the south side of the building by way of infill on multiple levels to create larger floor plates is structurally feasible. There are two main options to achieve this (refer to Figures 1 and 2 in Appendix B). The first option is to install new columns that "punch" through the existing floors at lower ground floor, ground floor and first floor within the footprint of the infill floor plates. The new column positions would need to be structurally determined to avoid clashing with the existing columns, footings and floor beams. Whilst the existing floor structure can remain at lower ground floor, ground floor and first floor levels with this option, the



number of columns in the lowest two stories is increased. This will have an impact on the flexibility of the proposed carpark layout.

The second option is to demolish the existing structure at lower ground floor, ground floor and first floor levels within the footprint of the infill floor plates and replace with new structure. The advantage of this option is more flexibility to locate the new columns in architecturally favourable positions.

In either option, the new structure would have to be independent of the existing building in terms of the support structure for vertical loads. However the lateral resisting structure of the new and existing buildings may be independent or connected. This would depend on the lateral strength of the existing structure to resist wind and earthquake loads to current Australian Standards. If the existing structure is found to be laterally inadequate, it may be possible to tie the existing structure to the new infill structure. If this were done, the new infill structure would be designed to resist the full lateral loads acting on the building resulting from wind and earthquake. The original structural drawings do not detail every beam-to-column connection. These connections are critical in the determination of the frame resistance of the steel structure to lateral loads. If the existing building were to remain laterally independent of the new infill building, the existing connections would have to be determined. This would require the insitu removal of the existing concrete encasement to expose some of the beam-to-column connections. Alternatively, new steel bracing or concrete shear walls could be retrofitted to the existing building.

- d. The proposed addition of terraces/ balconies to the northern face (where there are none currently) is structurally feasible. The new balcony structure could consist of new slabs connected to the existing floor structure with new columns on all levels located near the perimeter of the balcony. Alternatively, new cantilever steel beams could be connected to the existing building to support new balcony slabs. The second option would locally restrict the available headroom and be more difficult to construct. The existing edge beams that would support the new balconies may require strengthening to support the additional load of the balconies.
- e. Proposed internal stairs between levels to create two storey apartments is structurally feasible. New steel support beams would be required to support the existing floor slabs at the proposed stair penetrations. The position of the stair voids should preferably be located to suit the existing structural floor framing.
- f. The proposed conversion of the basement level to carparking is structurally feasible. The existing basement slab consists of a 100mm thick reinforced concrete slab-on-ground. This is considered adequate to serve a carpark function. If the condition of the slab is found to be unserviceable or unsightly (e.g. extensive cracking) those portions of the slab may be demolished and replaced.



5.0 SITE OBSERVATIONS

A brief site inspection of the lower levels of the Queen Mary building was undertaken by MPN on 21 October 2009. Visual inspections were undertaken both inside and outside the building. Internally, ceilings and floor finishes covered most of the structure. The original structural drawings of the building were not obtained until after the site inspection. Visual checks were made primarily to determine the various structural systems utilised for the building. The areas investigated were also assessed for evidence of abnormal movement, structural defects and water ingress.

Water ingress was evident in many of the internal areas inspected (refer to Photo 7 in Appendix A). The source of the water was not determined. The water could be due to faulty internal plumbing or inadequacies in the external fabric of the building. The presence of water has the potential to cause corrosion of the reinforcing steel in the floor slabs and to the structural steel beams/columns. Any defective structural elements would need to be repaired. This would best be investigated and treated following the complete strip out of the building when all the structural elements become visible and easily accessible. If the source of the water ingress is through the roof slab, the existing waterproof membrane (if one exists) would need to be replaced or a membrane installed if there is currently no form of waterproofing.

With the exception of water ingress, internally, the structural elements observed appeared to be in sound condition. This indicates that the structure is performing in a satisfactory manner.

Significant corrosion of reinforcement was observed in the concrete elements exposed to the weather (e.g. balcony slabs). The corrosion of the reinforcement has lead to significant spalling of the cover concrete (refer to Photos 8, 9 and 10 in Appendix A). The balconies need to be investigated further to determine the extent and severity of the corrosion. The defective areas, if not extensive, may simply be repaired using typical concrete repair methods. If the corrosion is found to be extensive, it may be simpler and more beneficial to demolish the balconies and replace them with new structure as described in Section 4d above.

Defects were observed in the masonry walls both internally and externally. On the basis of a complete strip out of the non-structural elements of the building, including the complete removal of the internal and external masonry walls, these defects are not discussed in this report.



6.0 SUMMARY

- a. The proposed conversion of the existing Queen Mary building to residential apartments is structurally feasible. The existing internal masonry walls and façade cavity masonry walls are non-loadbearing and can be demolished. This provides a substantial degree of flexibility for planning around the remaining structure. The proposed extension of the floor plates, balcony additions, internal stairs and basement carpark to facilitate the conversion to residential apartments are structurally feasible.
- b. Some site investigation works will be required to determine existing concrete covers, concrete strengths and existing connections to check for compliance with current Australian Standards and BCA.
- c. Repair of existing concrete spalling and reinforcement corrosion will be necessary to prevent further deterioration and to prolong the service life of the building.
- d. Lateral strengthening of the existing structure may be required to comply with current Australian Standards for Wind Load and Earthquake Load.

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APPENDICES



APPENDIX A
PHOTOGRAPHIC RECORD

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Photo 1: The Queen Mary Building during construction.

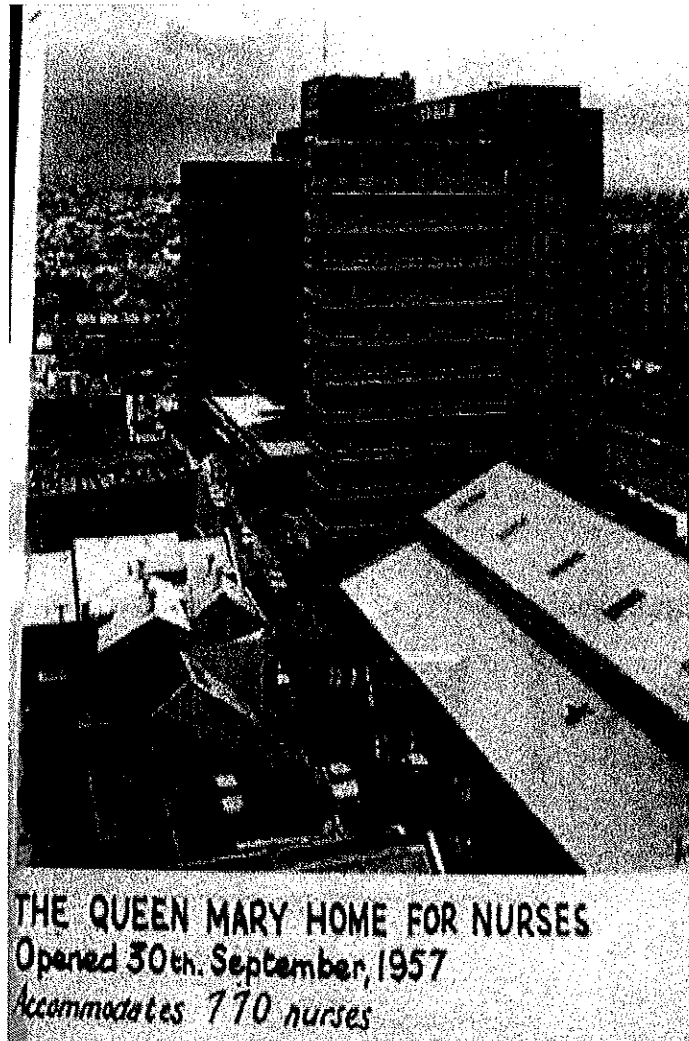


Photo 2: The Queen Mary Building at Opening.



Photo 3: South West Elevation of the Queen Mary Building.



Photo 4: South East Elevation of the Queen Mary Building

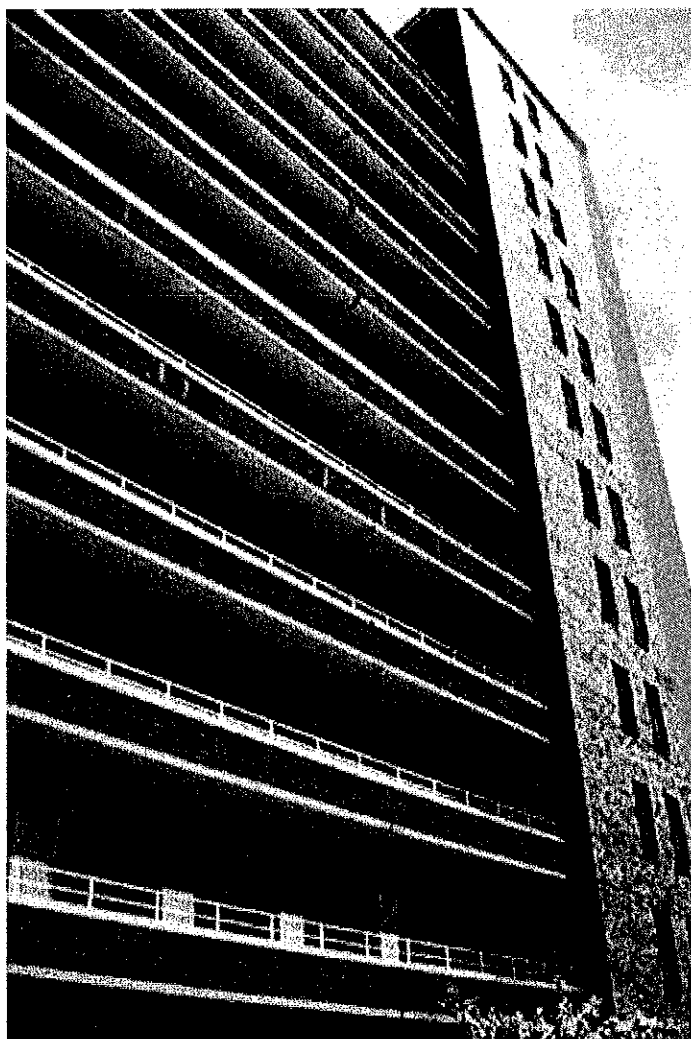


Photo 5: East Elevation – Queen Mary Building.



Photo 6: North West Elevation – Queen Mary Building.

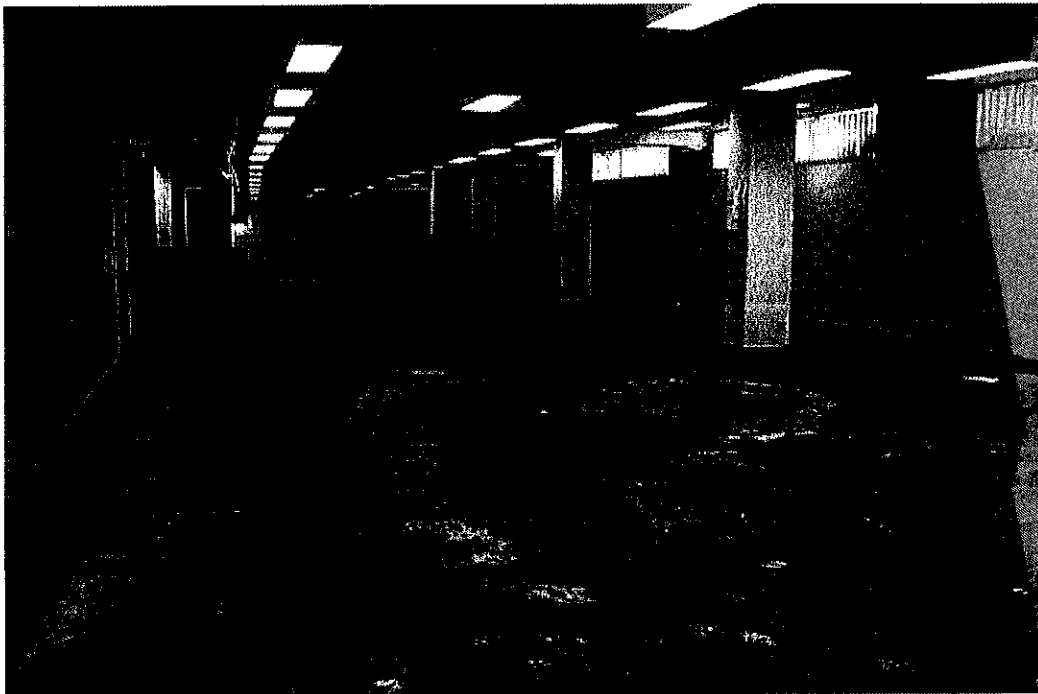


Photo 7: Evidence of water ingress.

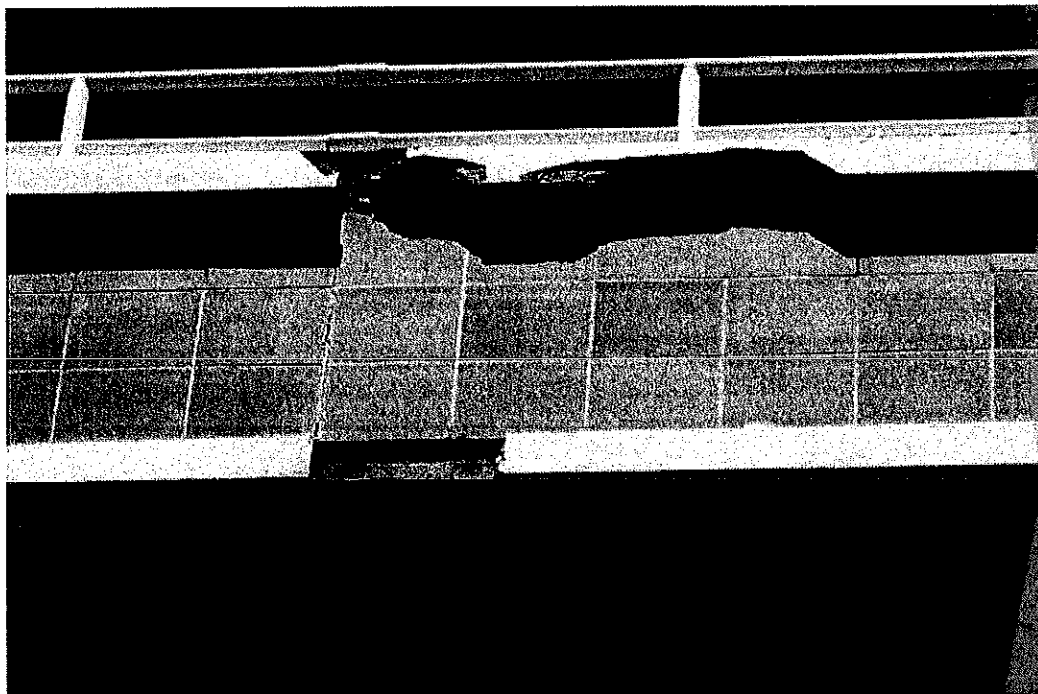


Photo 8: Concrete spalling and corrosion of reinforcement in balcony parapet and slab.

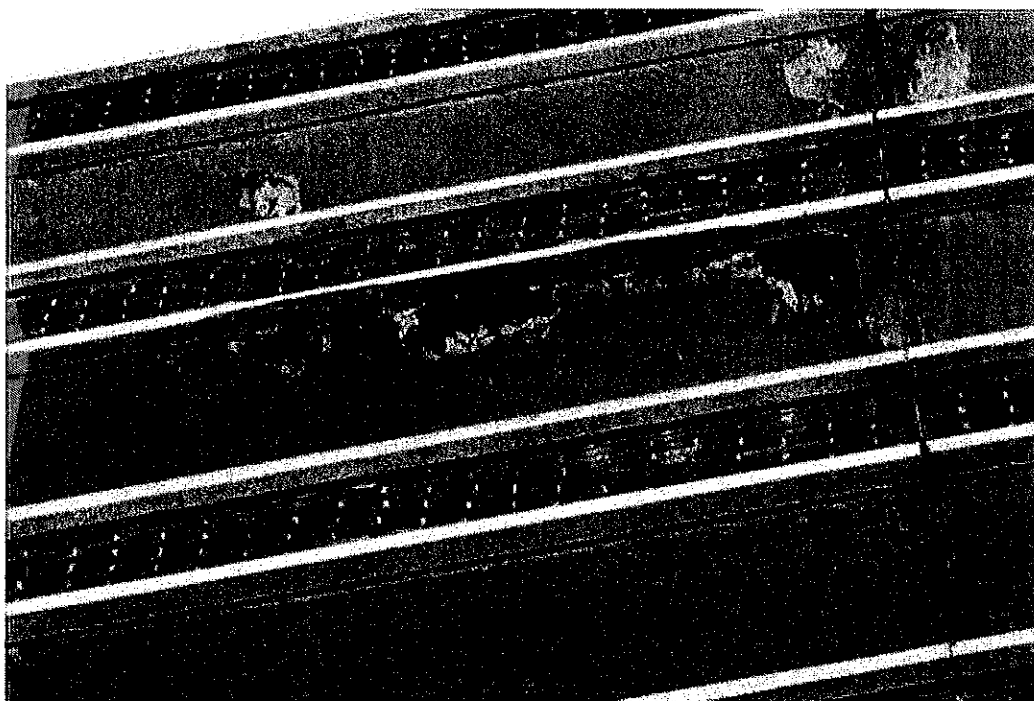


Photo 9: Concrete spalling and corrosion of reinforcement in balcony floor slabs.

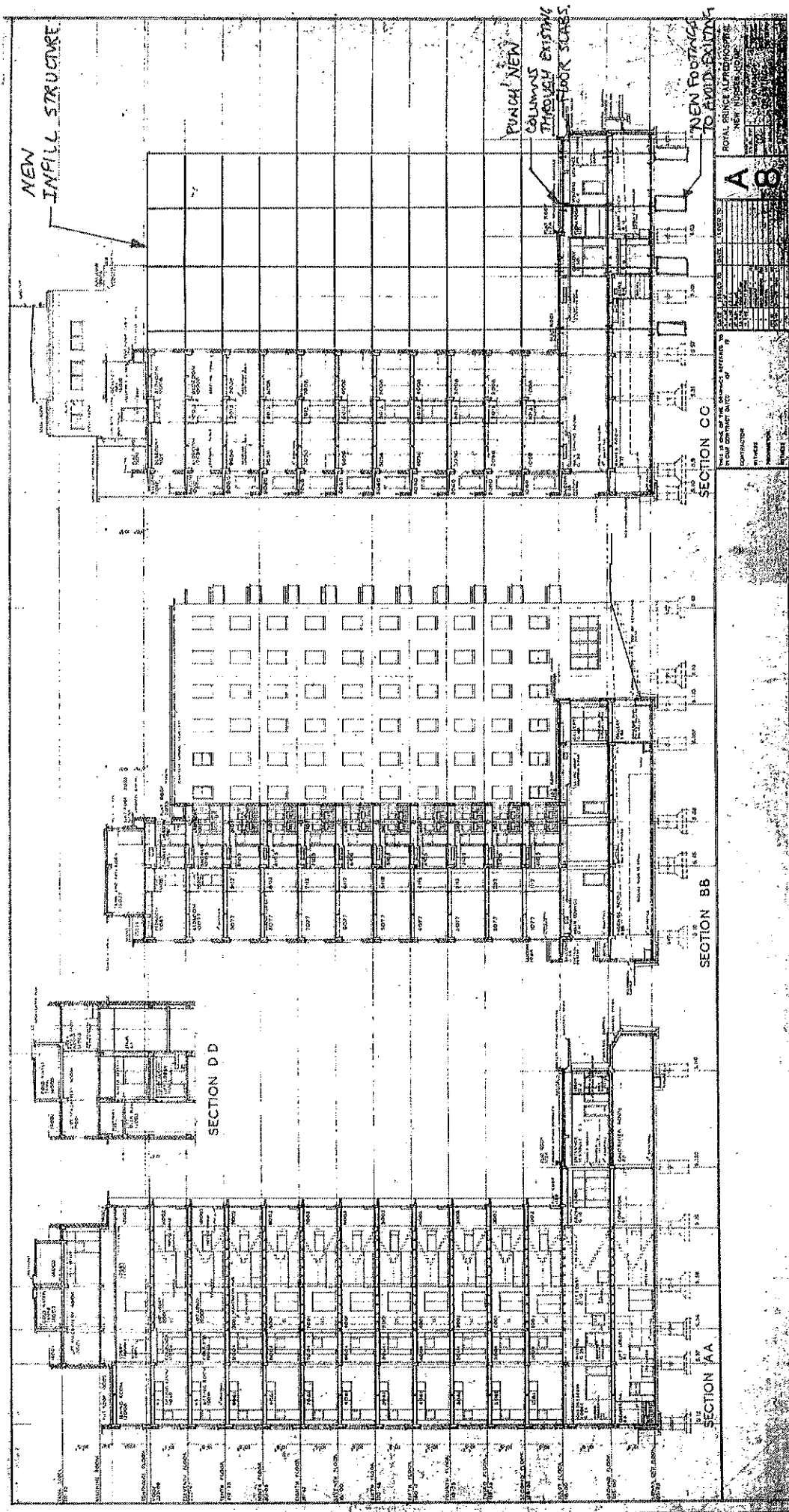


Photo 10: Concrete spalling and corrosion of reinforcement in sunhoods.



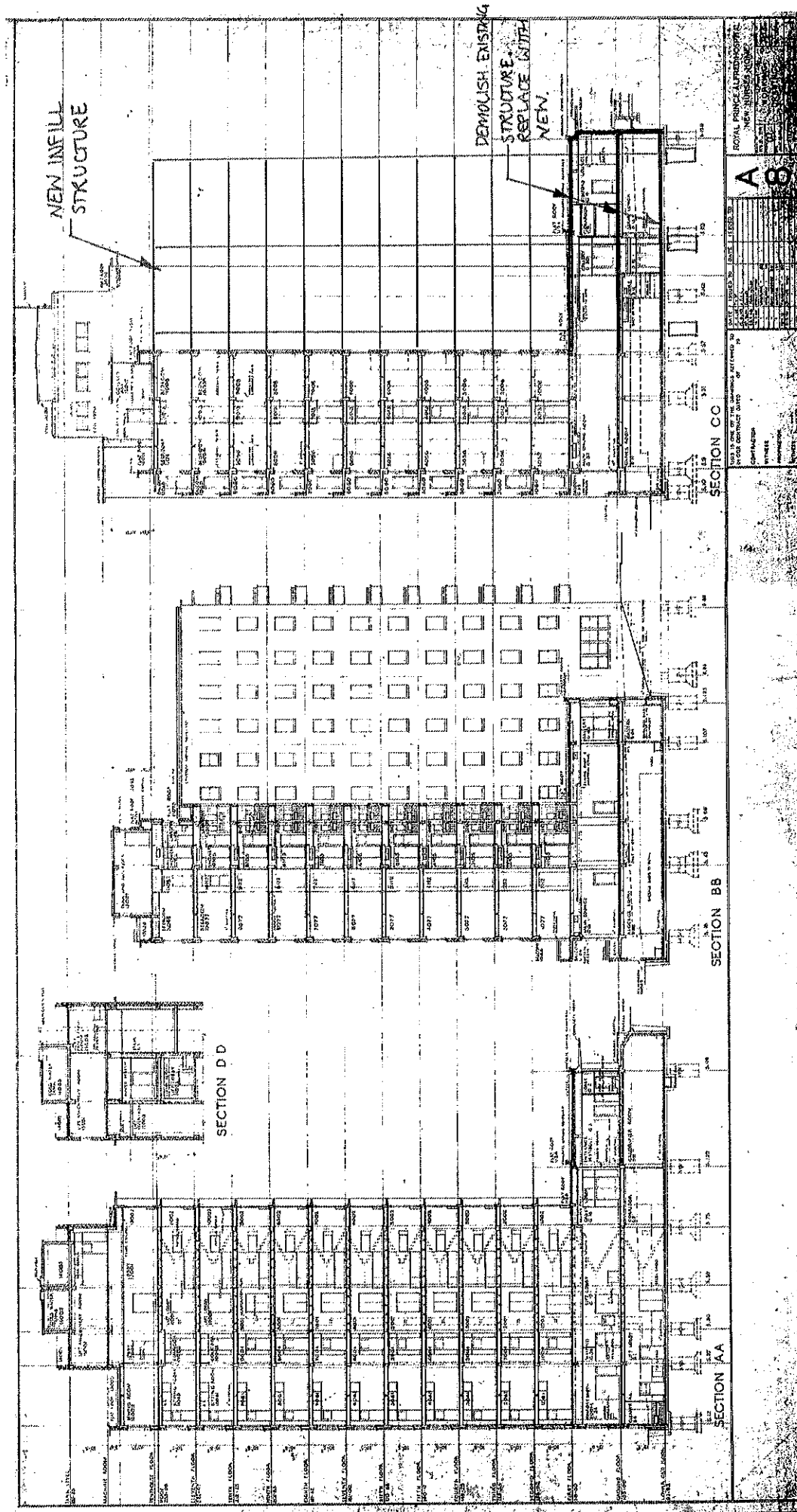
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APPENDIX B
DRAWINGS



INFILL STRUCTURE
OPTION ONE

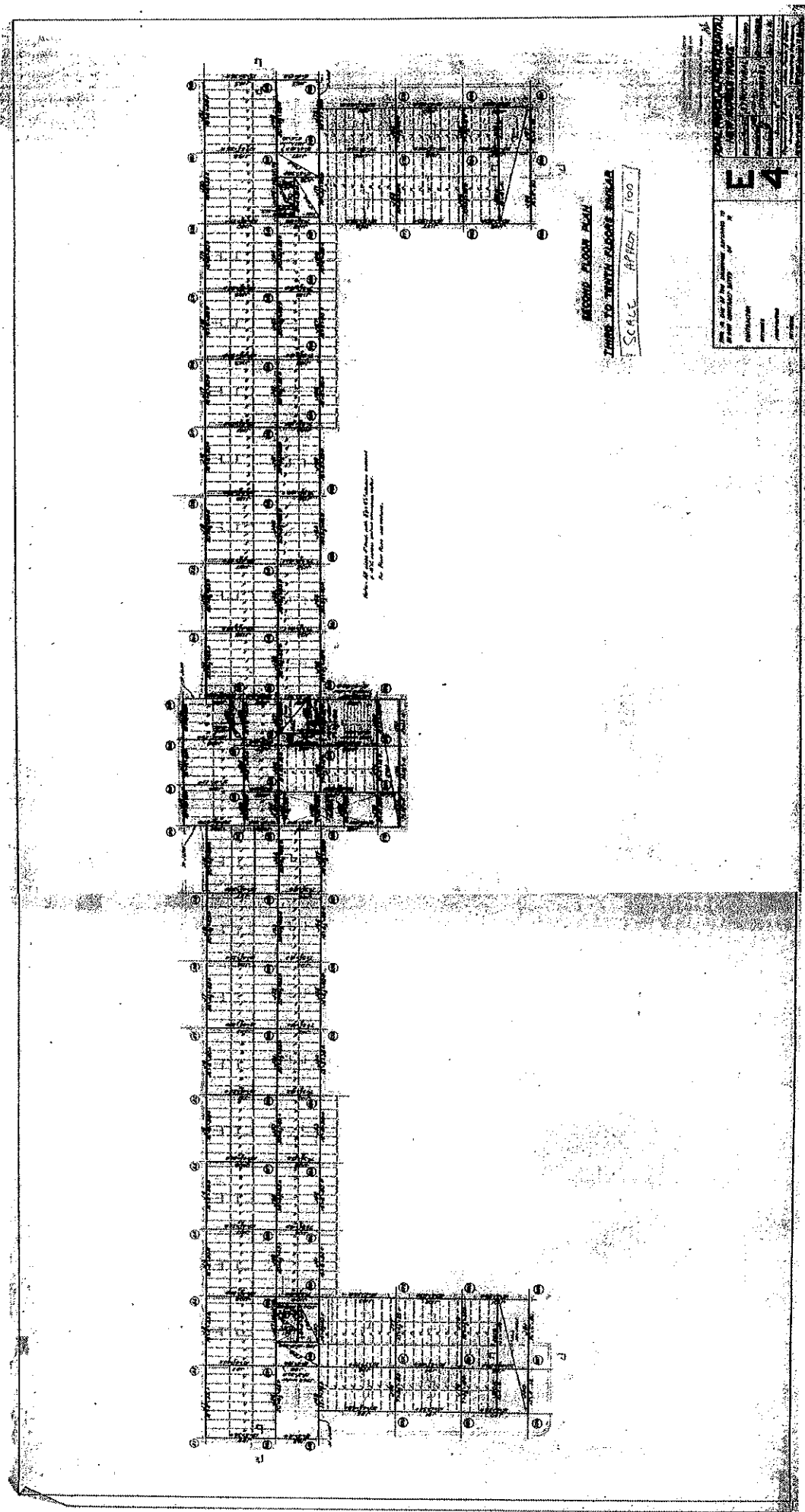
FIGURE 1.



INFILL STRUCTURE

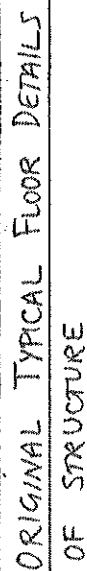
OPTION TWO

FIGURE 2.



ORIGINAL STRUCTURAL DRAWING
OF TYPICAL FLOOR

FIGURE 3.



OF STRUCTURE

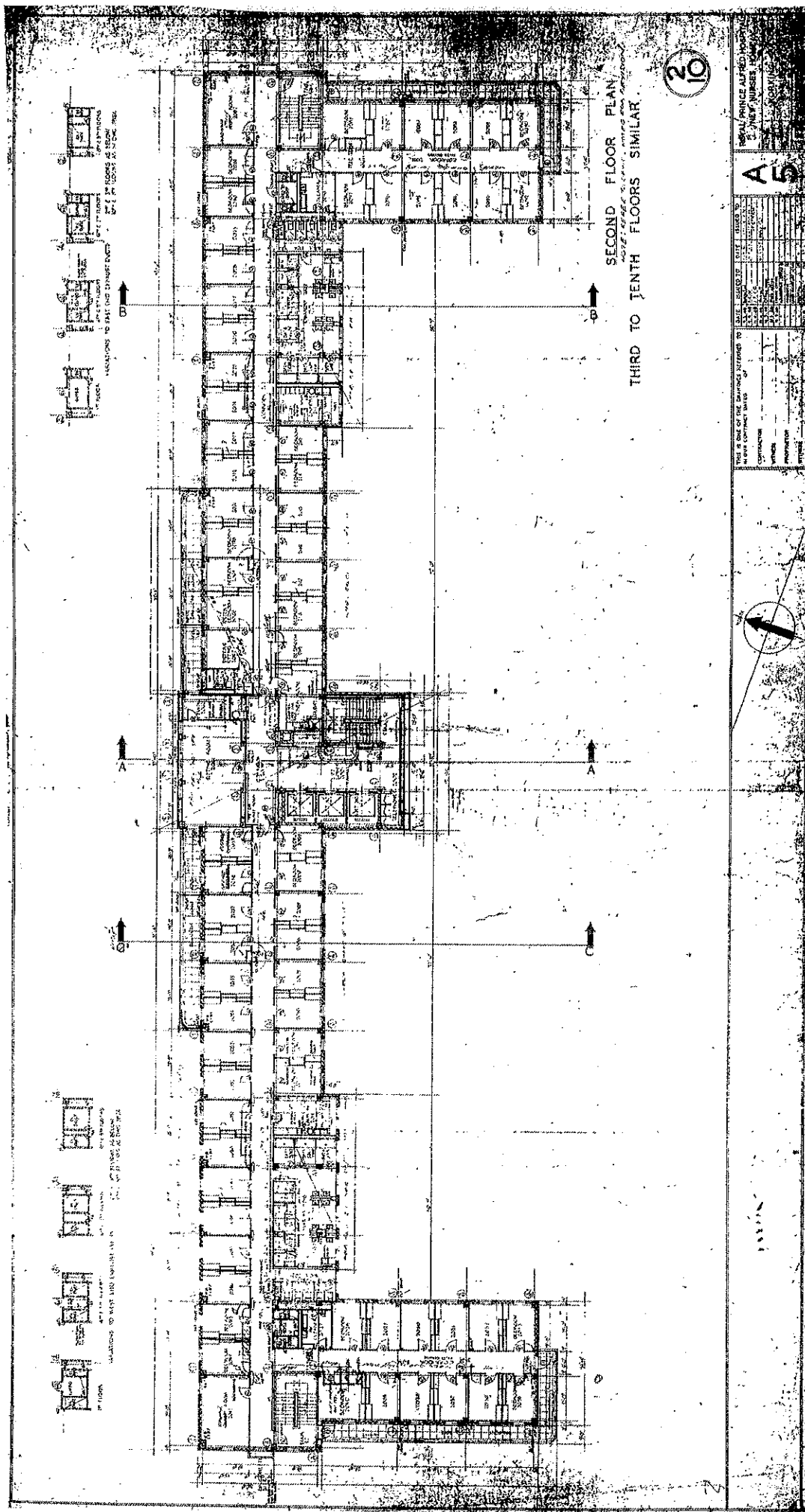
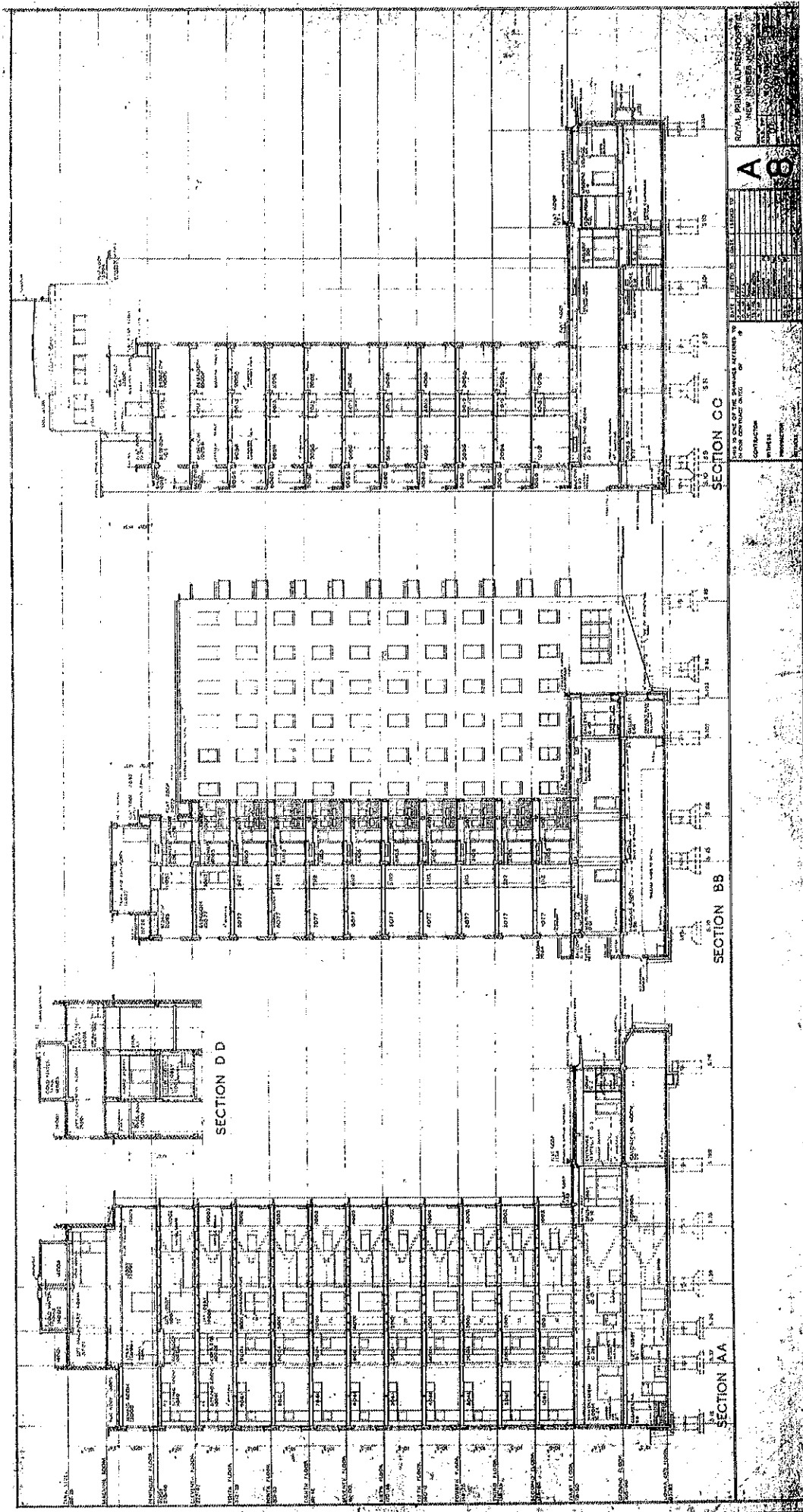
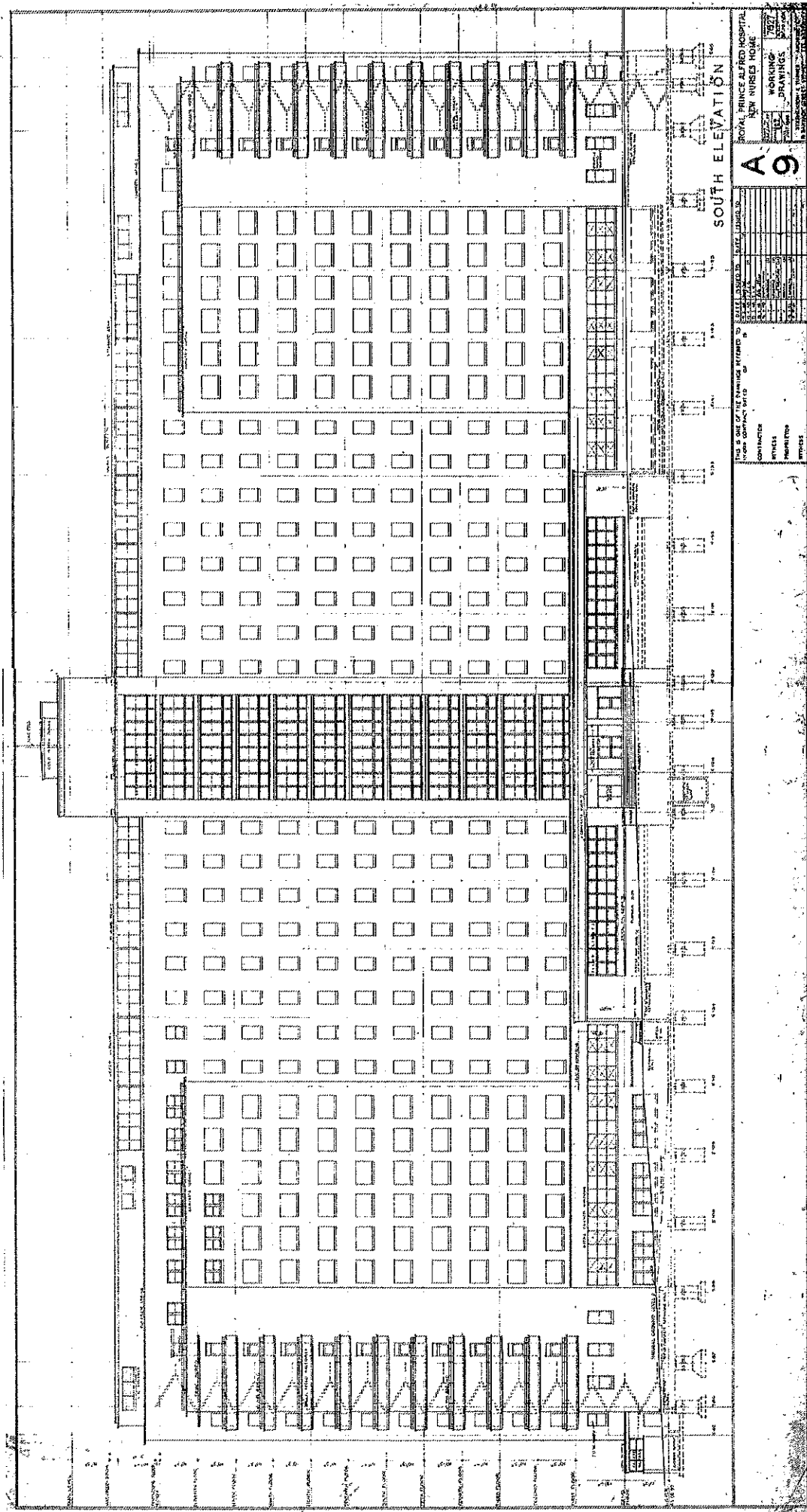


FIGURE 5.

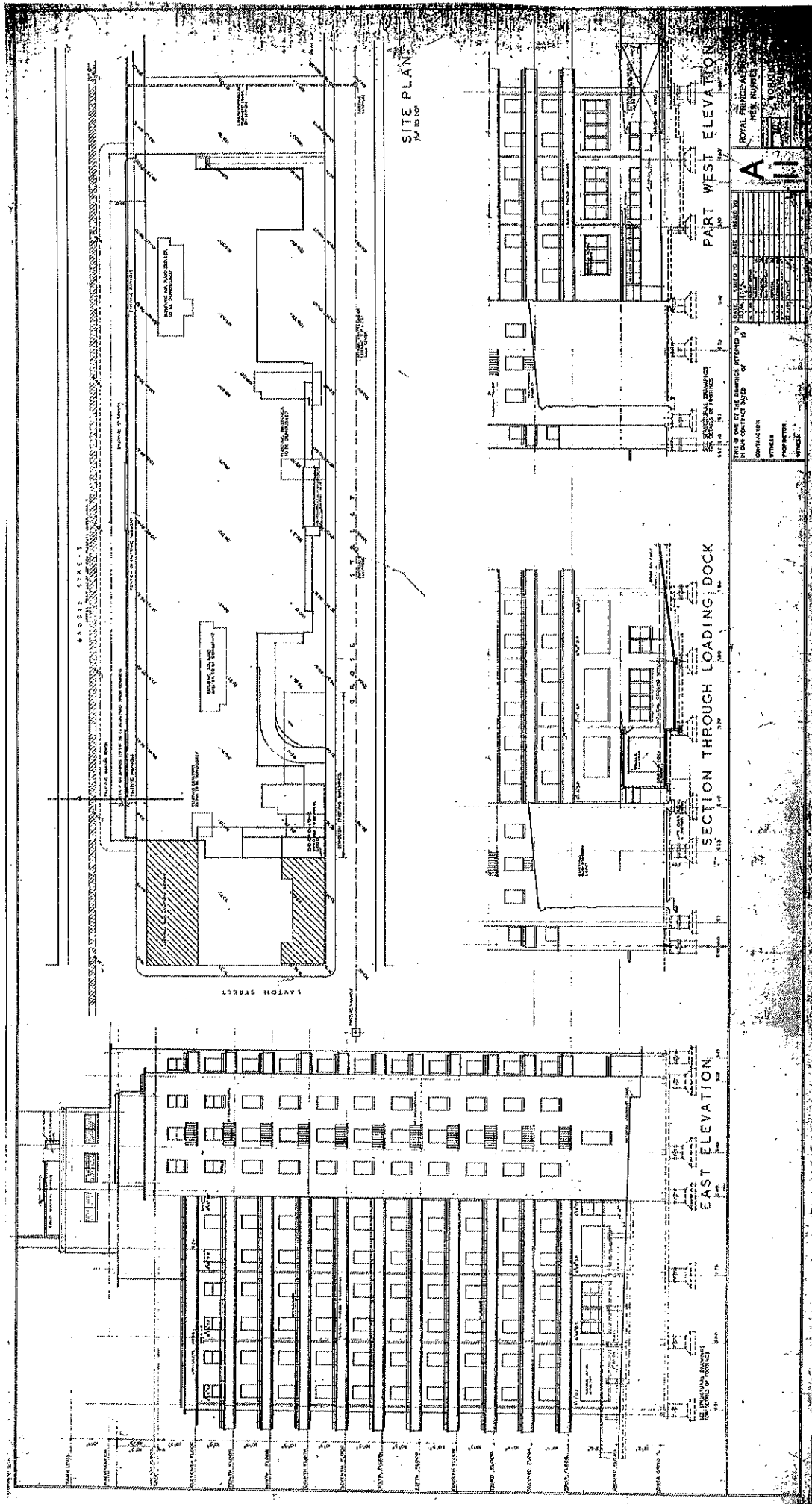


ORIGINAL ARCHITECTS DRAWING
OF BUILDING CROSS SECTIONS



ORIGINAL ARCHITECTS DRAWING
OF BUILDING ELEVATION

FIGURE 7.



ORIGINAL ARCHITECTS DRAWING
OF BUILDING ELEVATIONS & SITE PLAN

