Detailed Seismic Assessment

45 Hania Street, Mount Victoria, Wellington

Dated 22nd September 2022

CHP consulting engineers

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Big Feta Holdings Limited (c/o Antonios Focas)

Attn: Antonios Focas

Introduction

Background

The following report sets out the results of a Detailed Seismic Assessment (DSA) of the building located at 45 Hania Street, carried out by CHP Consulting Engineers for the Client, Big Feta Holdings Limited. The report provides an assessment of the seismic capacity of the building in relation to accepted building standards and guidelines at the time of reporting, outlining any key risks, and providing recommendations for further action.

The methodology for the DSA follows the 2017 Guidelines for building seismic assessment jointly produced by MBIE, the NZ Society for Earthquake Engineering (NZSEE), the Structural Engineering Society (SESOC) and the NZ Geotechnical Society (NZGS). Concrete components have been assessed in accordance with the revised Part C5 of the aforementioned guideline document dated 2018.

Limitations of Report

This report has been prepared for Big Feta Holdings Limited. The reliance by any other parties on the information contained in this report shall, without the written approval of CHP Wellington Limited, be at such parties' sole risk.

Building Description

The building is a three-storey structure with a small light weight mezzanine floor within the upper-level roof space and is currently used for commercial purposes. The building was constructed circa mid-1960s and consequentially it was designed prior to the introduction of modern seismic codes. The original documentation provided appears to indicate alteration works to the mezzanine level were carried out late 1990s which largely involved a partial demolition of mezzanine floor areas and reconfiguration of lightweight partition walls.

The structure generally comprises of reinforced concrete frames with un-reinforced masonry or block work infill in the longitudinal building direction which was a common form of construction in the era constructed. A reinforced concrete infill wall runs along the eastern boundary ground floor level which is partially retaining the neighbouring property. Floors typically comprise of cast in-situ reinforced concrete spanning between reinforced concrete beams and frames.

The lift shaft and stairwell along the southern side of the building are typically bound by short reinforced concrete walls up to roof level. The roof structure comprises of a series of steel beams at column lines with timber purlins and lightweight steel iron roofing.

Foundations appear to comprise of shallow pad footings beneath columns which, the foundation pads are tied together with a series of reinforced concrete ground beams with a reinforced concrete slab overtop.

Refer figure 1 below indicating the overall site plan and nominated longitudinal and transverse directions.

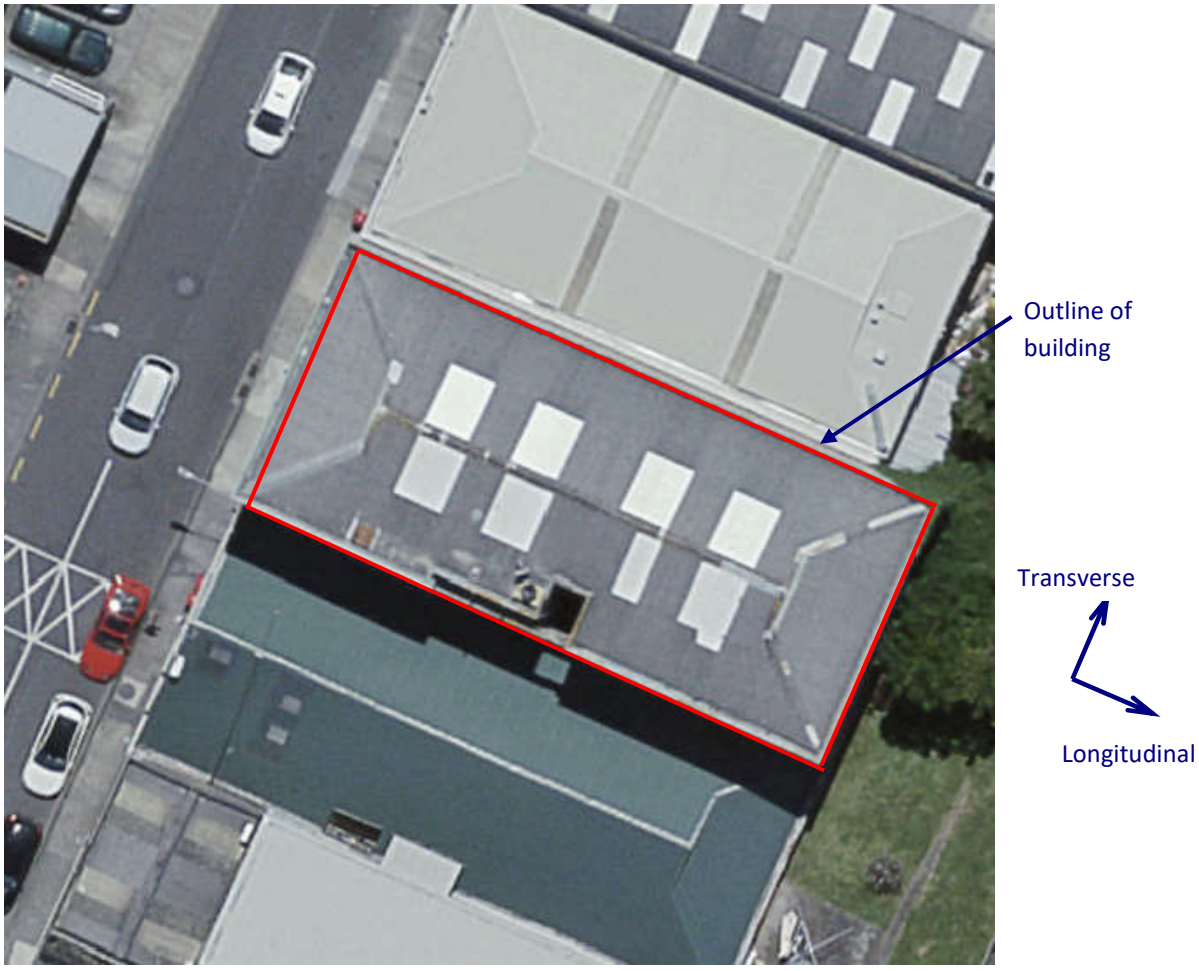


Figure 1: Aerial View of 45 Hania Street (GWRC Web Map)

Site Geotechnical Considerations

A site-specific geotechnical investigation involving in-situ ground testing, has not been carried out for the purposes of this DSA. We have however drawn a number of conclusions based on available documentation and data at the site.

Based on a study on the site subsoil class types in the Wellington CBD, 2019 Kaiser et al. the site sits within an area expected to have a type C 'Shallow Soil' site subsoil classification as defined in NZS 1170.5:2004. The NZ Geotechnical Society Database was inspected and does not appear to have any relevant specific information in the immediate vicinity. Subsequently, a site subsoil class type C was adopted for the purposes of this DSA.

Review of the Greater Wellington Regional Council (GWRC) hazard maps shows geohazards at the site including slope stability, tsunami, liquefaction and lateral spreading are a low risk and furthermore the site is not situated within a known fault rupture zone.

Based on the original 1963 drawings it appears the pad foundations are cast progressively deeper toward Hania Street, presumably to found on an adequate bearing substratum which tapers off toward the West. For the purposes of this DSA, and following a review of pad sizes noted on existing building documentation, we have assumed a foundation bearing failure is unlikely and furthermore is deemed not a critical life safety hazard with respect to seismic performance of the existing structure.

A site-specific geotechnical study involving in-situ ground testing would be advantageous to enable any future building structural alterations or retrofits and confirm assumptions made within our assessment.

Lateral load resisting systems

Transverse lateral load resistance is provided by the reinforced concrete members comprising of walls, frames and cantilevering columns. Specifically upper-level transverse resistance is provided by a mix of cantilever columns, moment frames at the front and rear of the building and also the short walls bounding the stairwell and lift shaft. The mid-level transverse resistance is provided largely by moment frame action with some contribution from the stair and lift shaft walls. Due to the relative stiffnesses between the shear walls and moment frames at ground level, the moment frames provide limited contribution and subsequently the rear shear wall and the short walls bounding the lift shaft provide the predominant ground level transverse direction resistance.

Longitudinally, the lateral load resistance is provided by the infilled reinforced concrete moment frames along the North and South boundaries. Additional resistance is provided by the reinforced concrete walls bounding the stairwell and lift shaft.

To transfer and distribute seismic load into the lateral load resisting elements, the building relies on the in-situ reinforced concrete floor diaphragm at first and second floor levels.

Building condition assessment

A specific building condition assessment and survey, including material testing, was not undertaken and is outside the scope of this report.

Figure 2: View of building from Hania Street looking southeast.

Basis for this Assessment

The following information was made available and/or discovered as part of the detailed seismic assessment.

- Building consent documentation consisting of the original plans dated 1963 retrieved from the Wellington City Council records.
- Visual inspections of the building by CHP conducted in May 2021 and June 2022 to largely confirm structural systems and if these are in-line with existing building documentation made available.
- Scratch testing of un-reinforced masonry interior surfaces undertaken August 2022.

Figure 3: Adjacent Buildings to the North [above left] and to the South [above right].

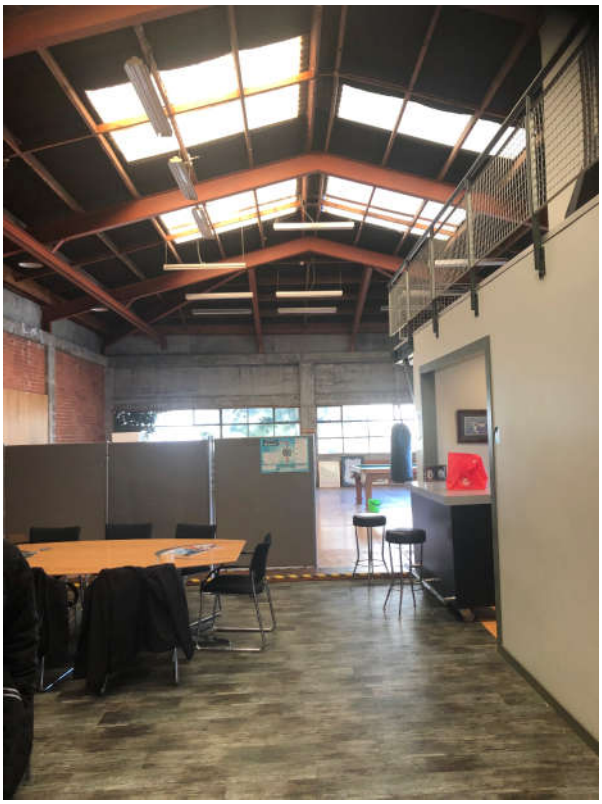


Figure 4: Upper Level Floor Space.

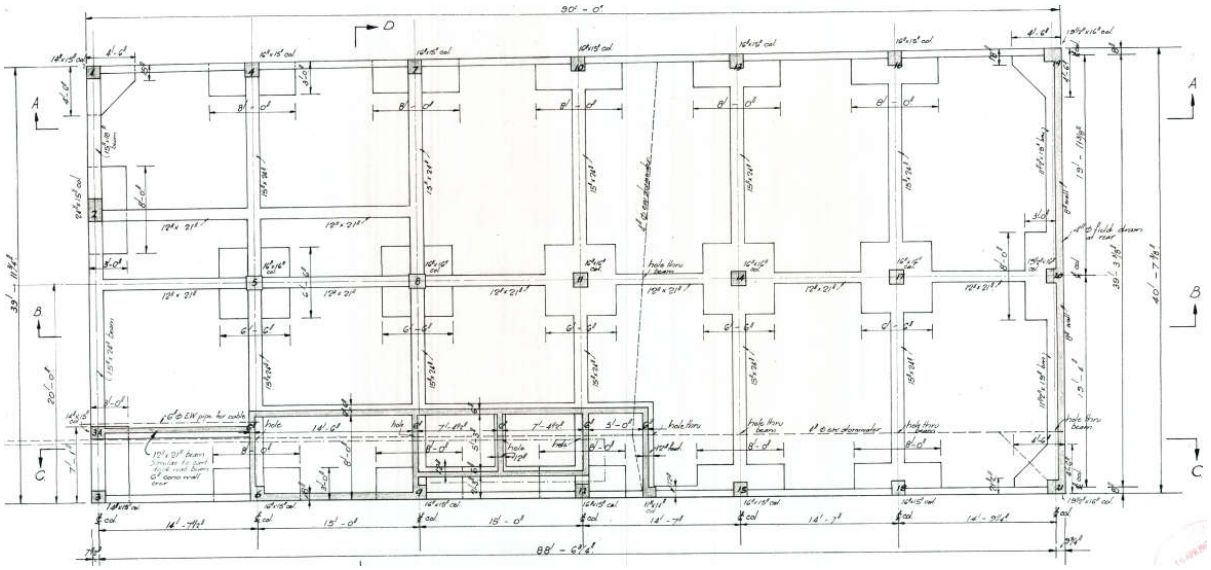


Figure 5: Foundation and Typical Floor Plan – Original Drawings, 1963

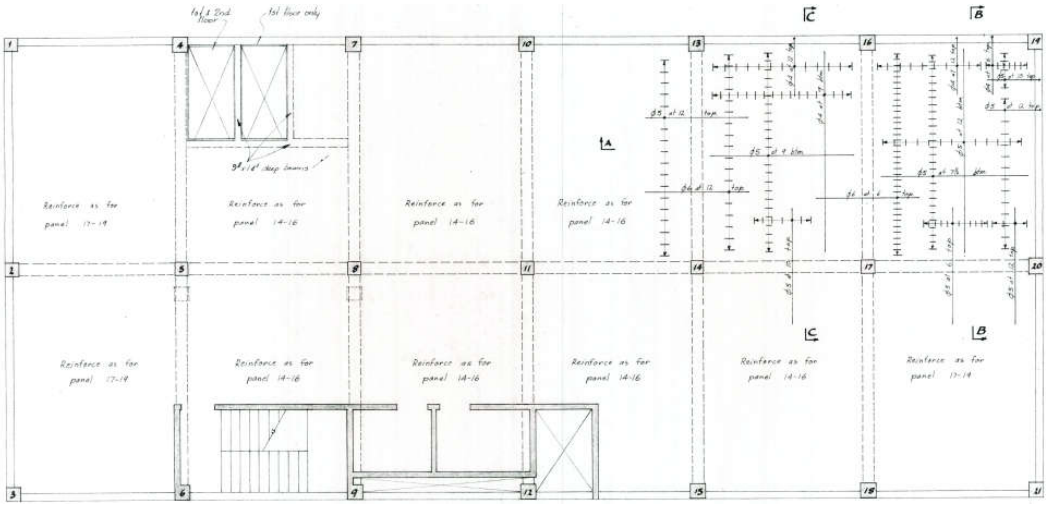


Figure 6: Typical Floor Plan – Original Drawings, 1963

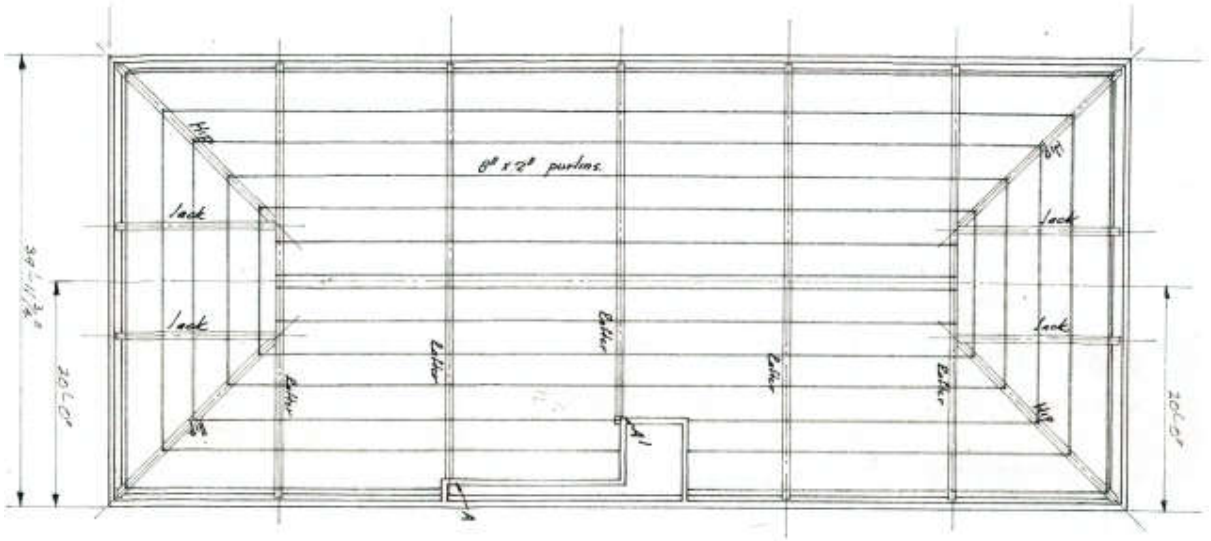


Figure 7: Roof Plan – Original Drawings, 1963

Assessment Procedure

The assessment procedure for the primary lateral structure was a displacement based SLaMA non-linear pushover analysis from Section C2 of the *Assessment of Existing Buildings - Technical Guidelines for Engineering Assessments*, July 2017. Other sections of Part C applied were: C5 – Concrete [revised 2018], C7 – Moment Resisting Frames with Infill Panels, C8 – Un-reinforced Masonry Buildings.

Force based procedures, in accordance with the guidelines, were used to carry out the analysis of the upper-level cantilever columns, the reinforced concrete parapets and the reinforced concrete diaphragm. The building was considered to be Importance Level 2 as per NZS 1170.0.

To determine the %NBS rating for the building we assessed the following:

Longitudinal direction:

- The east and west boundary infilled reinforced concrete framed walls
- The floor diaphragms

Transverse direction:

- Cantilevered reinforced concrete columns in the upper level
- Reinforced concrete moment frames in the lower and mid-level
- Reinforced concrete walls in the ground floor and adjacent lift shaft / stairwell
- The floor diaphragms

Other items:

- Un-reinforced masonry (URM) infill partitions out of plane
- Reinforced concrete parapet
- Seismic pounding to neighbouring buildings to the North and South

Assessed Seismic Risk

The results of the DSA indicate the building's seismic rating to be **~15%NBS (IL2)** for the required 1 in 500-year earthquake event. Therefore, this is a **Grade E** building following the New Zealand Society of Earthquake Engineering (NZSEE) grading scheme. Grade E buildings represent a risk to occupants >25 times greater than that expected for a new building, indicating high risk exposure.

Summary of Results

Specific building element capacities that were investigated are presented in the following table.

Transverse direction

Element	Failure mode	%NBS
Whole building combined action – (displacement-based assessment)	Reinforced concrete moment frame beam column joint shear failure (round bar). Soft storey /	30%

	column sway mode was predicted between first and second level.	
Cantilever Column – L2	Flexural lap splice failure	**15%**
Reinforced Concrete Moment Frame East and West Walls – L2		100%

Note: The displacement-based assessment procedure for in-plane actions on frames as prescribed in the MBIE guidelines gives a single result for multi-story frames rather than a result for each level. Secondly, as this building has rigid concrete floors, all of the primary structural elements are connected and must be assessed together. Hence the analysis produces a single result for the building as a whole which cannot be broken down further into more individualised results.

Longitudinal direction

Element	**Failure mode**	**%NBS**
Reinforced concrete shear wall		100%
Infilled reinforced concrete moment frames – L2		100%
Infilled reinforced concrete moment frames - L1	Column Shear Failure	20%
Infilled reinforced concrete moment frames – LG	**Column Shear Failure**	**15%**

Other components

Element	**Failure mode**	**%NBS**
Reinforced concrete floor diaphragm – L2 (pESA)		100%
Reinforced concrete floor diaphragm – L1 (pESA)	Shearing of floor starter bars connecting floors to concrete shear rear wall	75%
Reinforced concrete lift shaft – out of plane		100%
Un-reinforced Masonry Infill Walls [North Wall Panels] Out-of-plane – L2	Crack Mid Height	45%
Un-reinforced Masonry Infill Walls [South Wall Panels] Out-of-plane – L2		100%

Un-reinforced Masonry Infill Walls Out-of-plane – L1		100%
Blockwork Infill Walls Out of plane – LG		100%
Reinforced Concrete Parapet		100%

Critical structural weaknesses are defined by the lowest scoring structural weakness identified during the course of the analysis. Therefore, the critical structural weakness for this building is the cantilevering concrete columns in the upper-level transverse direction and the infilled reinforced concrete moment frames in the longitudinal direction.

These infilled frames have been assessed to fail in a brittle column shear mode near the base. The cantilevering concrete columns in the upper level have been assessed to fail in a flexural lap splice mode. Generally, a number of reinforced concrete elements have been assessed to fail prematurely largely due to round bar detailing with limited anchorage or splice capacities.

Seismic pounding effects of adjacent buildings were assessed to have no impact on the overall buildings NBS rating, based on the simplified checks established in the Part C2 guideline document appendix C2B.

Recommendations

We recommend the building is strengthened to at least 67% NBS (IL = 2) which is the minimum level recommended by the New Zealand Society of Earthquake Engineering (NZSEE). We believe the building is receptive to strengthening and subsequently we have outlined a strengthening scheme to achieve this minimum which would suit the existing building characteristics.

<u>(Strengthening target to greater than 67% NBS)</u>

- Undertake a site-specific geotechnical study. Confirm probable bearing parameters under existing foundations and also provide design parameters for new foundations.
- Install three new reinforced concrete shear walls to all levels. These walls shall be located to reduce torsional response of the building at ground level and to also provide a lateral load resisting system which has stiffness compatibility with the existing structural configuration. Associated steel drag beams fixed to the floors would be required. Associated foundation beams, pile caps and micro piles would be required subject to geotechnical engineering findings. Where possible the shear walls could be located alongside existing concrete walls alongside the lift shaft and stair well to minimise impact on the floor plan and current use.
- In order to prevent shear failure of the column bases along the northern boundary infilled wall, install new reinforced concrete skin over the end bays at ground and mid-level.
- In order to ensure adequate transverse bracing is provided at the upper level, install supplementary roof bracing to ensure perimeter walls and frames are tied back to the new reinforced concrete shear walls.

- In order to ensure the northern boundary URM (brick) wall is adequately restrained, install new RHS steel strong backs spaced at regular centres over the inside face of the wall. Alternatively connecting the outer and inner wythe with a suitable grid of fixings, such as Python C Screws, could be explored subject to detailed design.

It should be appreciated that the strengthening option presented is conceptual in nature and details of which shall be confirmed during detailed design. We recommend the Client considers the retrofit scheme presented herein and also undertakes any early costing exercises as deemed required.

Refer attached drawings 75222-01 to 75222-03 depicting the concept option noted above.

Conclusions

The existing building rating has been assessed as **~15%NBS (IL2)**. This is ultimately limited by both the transverse cantilever column capacity in the upper level and the infilled reinforced concrete moment frame shear capacity in the longitudinal direction.

We recommend the building is strengthened by implementing structural improvements to a number of key areas in particular the introduction of three new reinforced concrete shears walls at each level in the transverse direction and also new reinforced concrete wall overlays in the lower and med end bays of the north boundary wall. New steel roof bracing would also be required to distribute load into the new shear walls.

We trust the above report is of value to the Client. We would welcome the opportunity to discus and explore these options further and progress the detailed design of the seismic strengthening scheme presented here in.

Yours Sincerely,

CHP Consulting Engineers

Report Prepared By:

Jared Greig
BE(hons) MEngNZ

Report Reviewed By:

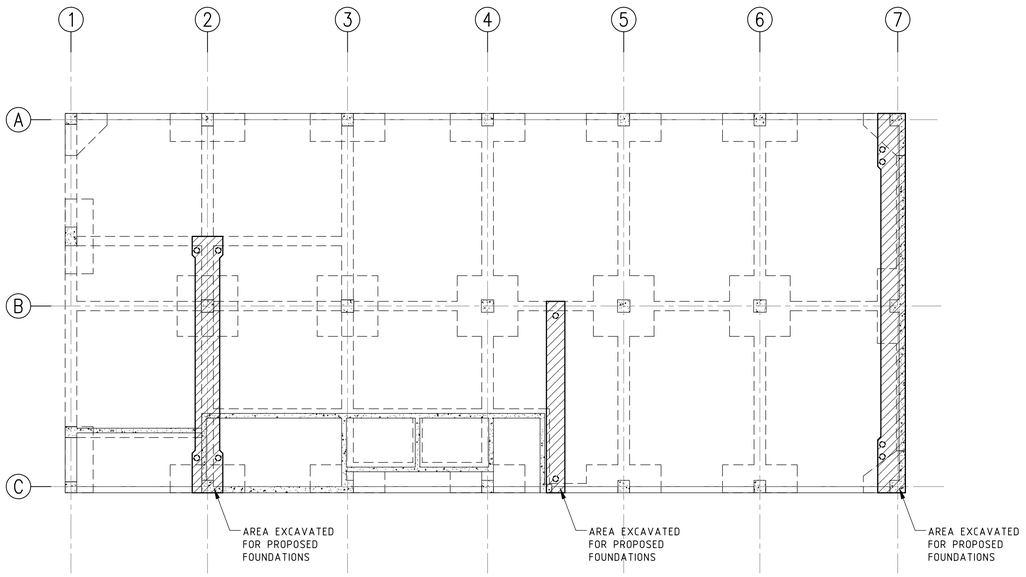
James Mulligan
BE(hons) CMEngNZ CPEng IntPE(NZ)

Attachments:

- Concept Seismic Strengthening Scheme to 67%NBS; CHP drawings 75522-01 to 735522-03 dated September 2022.
- Original Building Drawings dated 1963.

Attachment – Seismic Strengthening Concept Drawings 67%NBS (IL=2)

NO	REVISION	BY	DATE



EXISTING FOUNDATION LAYOUT/PROPOSED DEMOLITION

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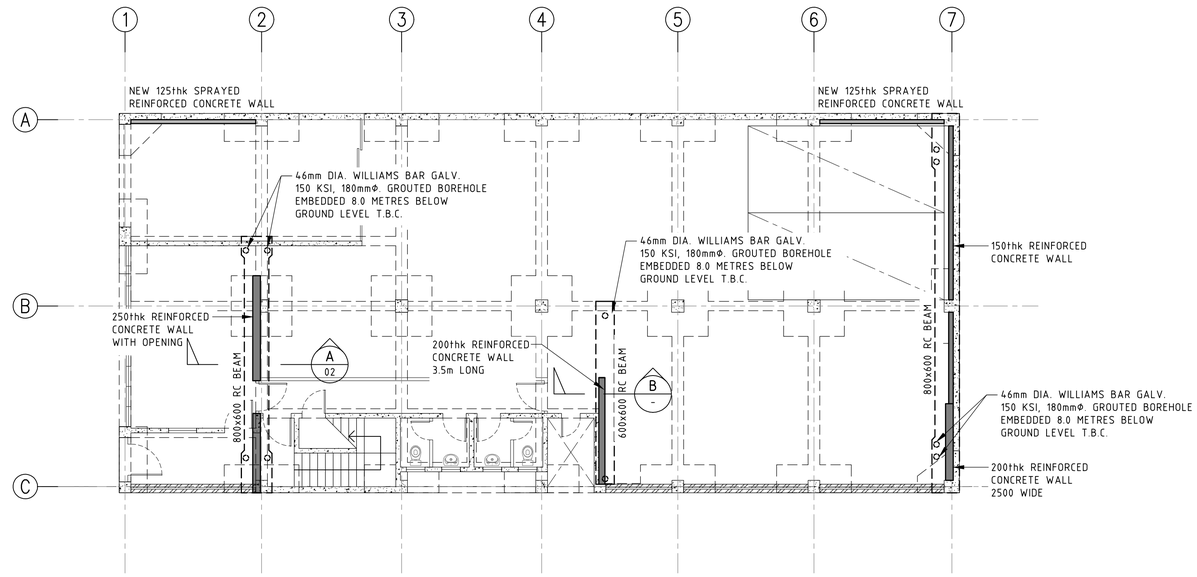

PROPOSED GROUND FLOOR STRENGHENING PLAN

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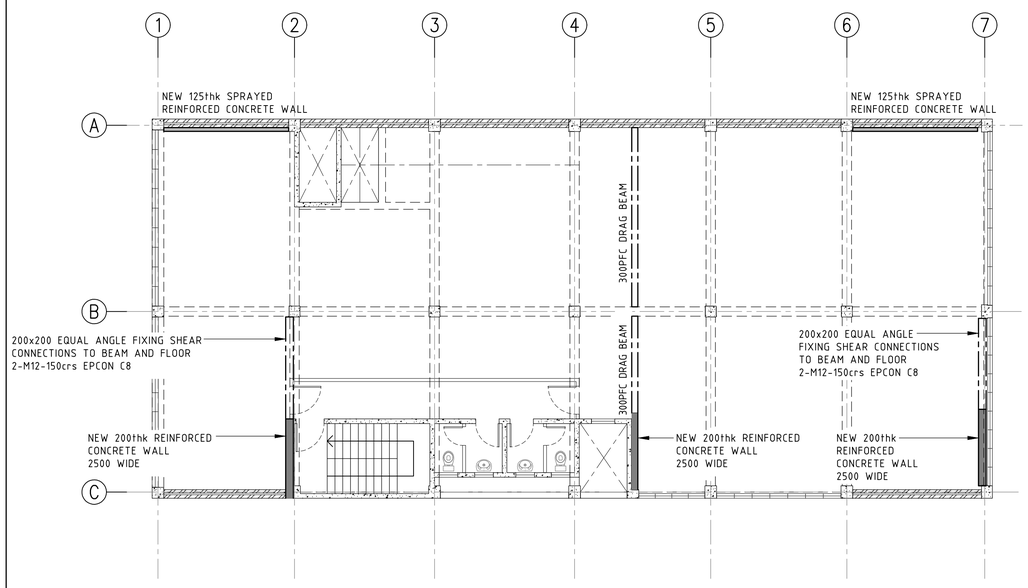
SEISMIC STRENGTHENING
CONCEPT DRAWINGS
67%NBS (I.L.=2)



PROPOSED FIRST FLOOR STRENGHENING PLAN

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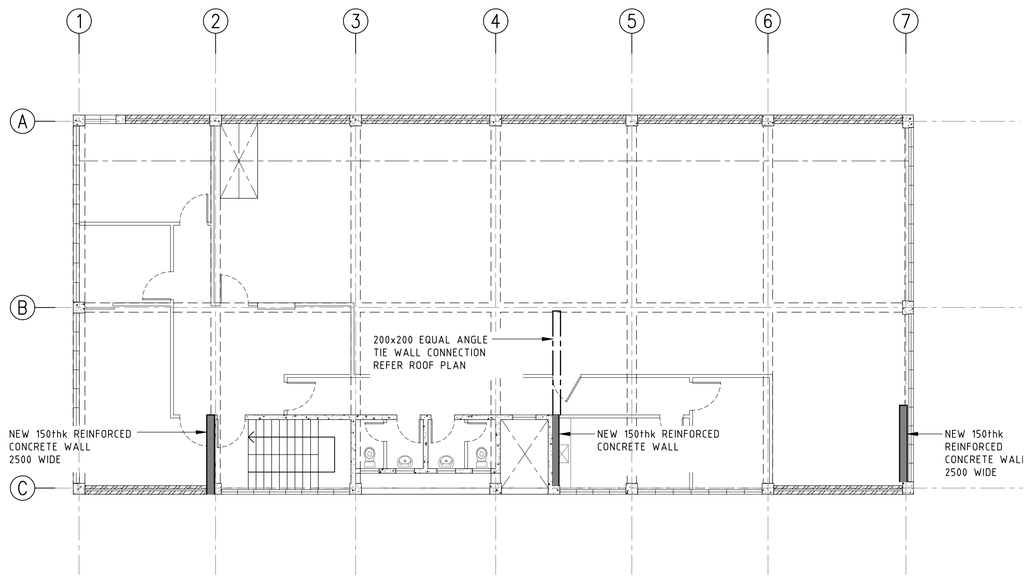

PROPOSED SECOND FLOOR STRENGHENING PLAN

1:100 @ A1
1:200 @ A3

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DRAWING TITLE
PROPOSED STRENGTHENING FLOOR PLANS

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DRAWN	AQN	FIELD BOOK	
CHECKED	JRM	DATE	AUG 2021

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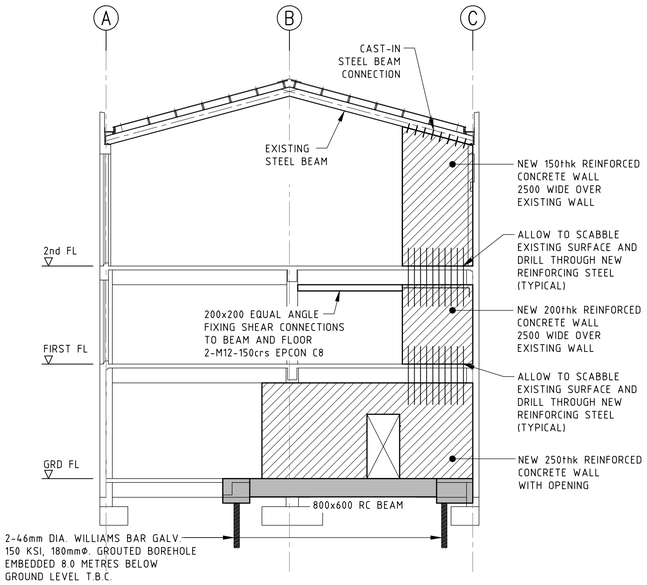
DRAWING NUMBER: 75222-01
REV:



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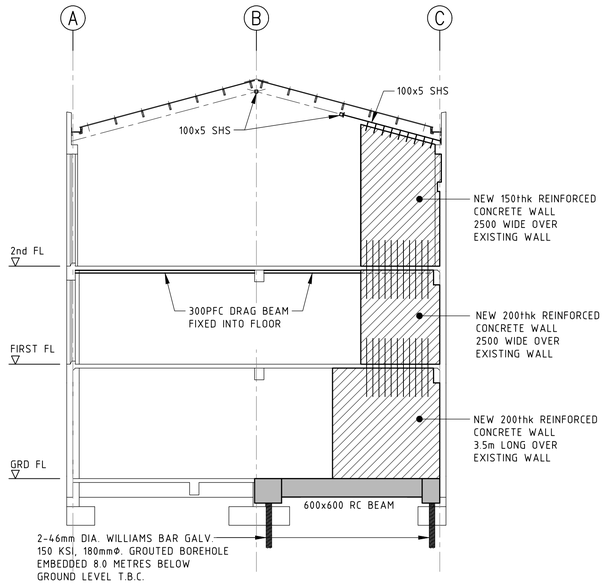
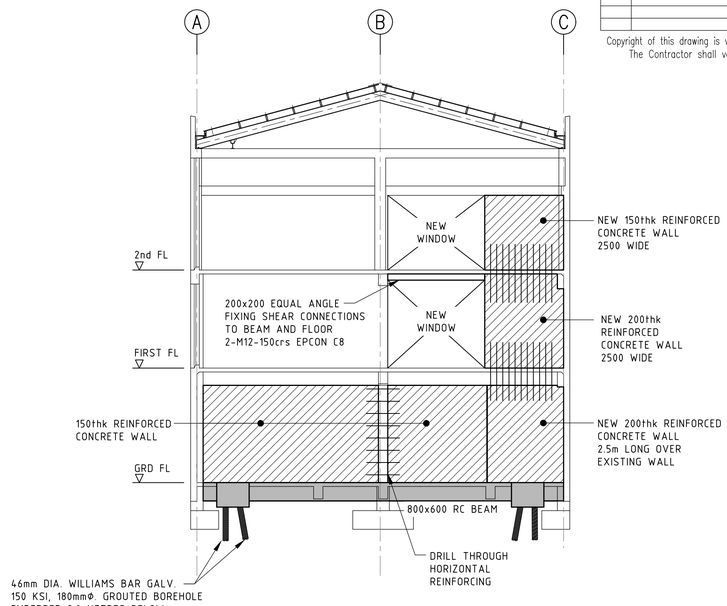


INDICATIVE SECTION BETWEEN GRIDS 4 & 5



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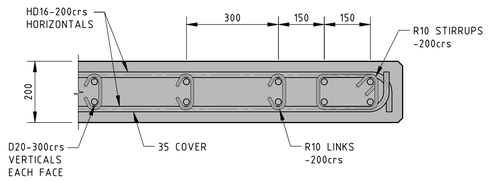

TYPICAL 200THK WALL END DETAIL

INDICATIVE TO GAUGE
REINFORCING STEEL CONTENT

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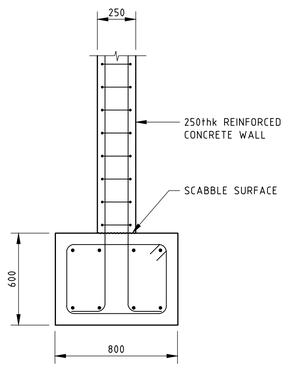
1:20 @ A3



SECTION A 01

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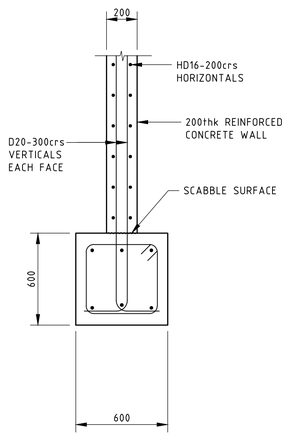

SECTION B 01

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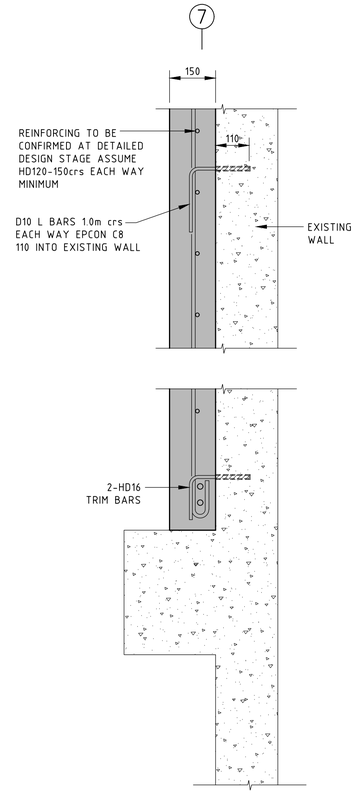

150THK WALL DETAIL

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NO	REVISION	BY	DATE

SEISMIC STRENGTHENING
CONCEPT DRAWINGS
67%NBS (I.L.=2)

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DRAWING TITLE
PROPOSED SECTIONS
AND DETAILS

DESIGNED	JRM	CAD FILE	76521-01
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CHECKED	JRM	DATE	AUG 2021

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AS SHOWN	A1

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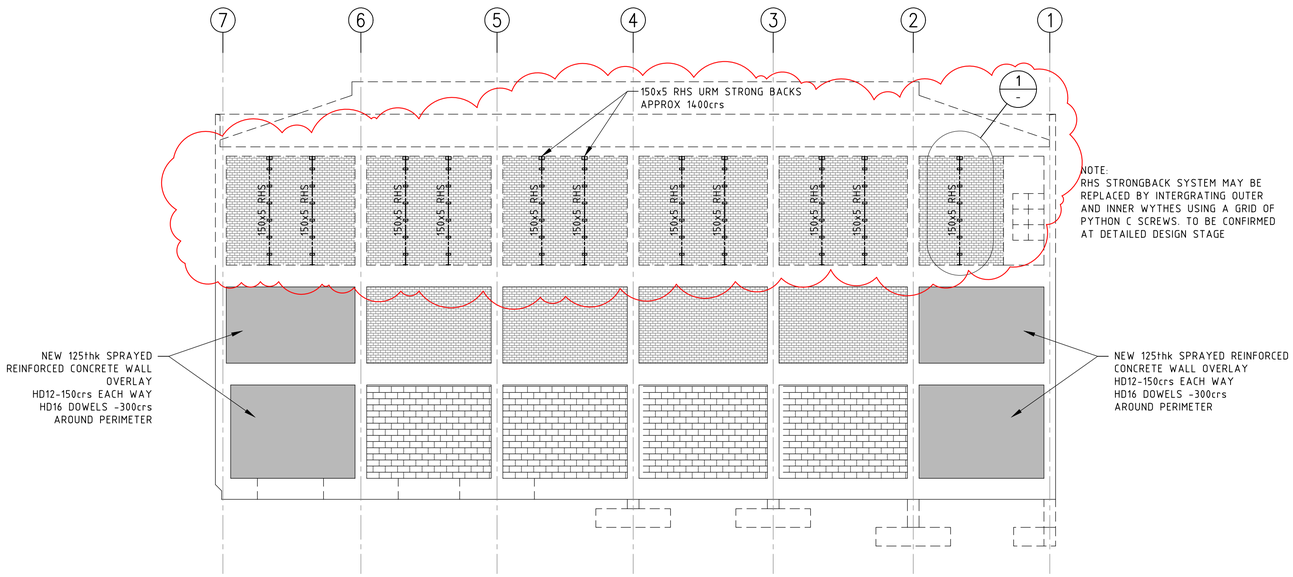
NO	REVISION	BY	DATE



NORTH ELEVATION
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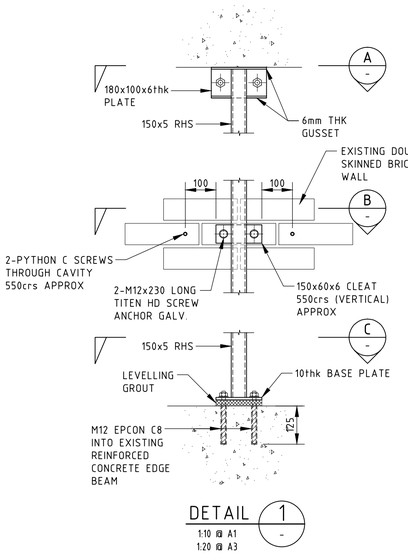

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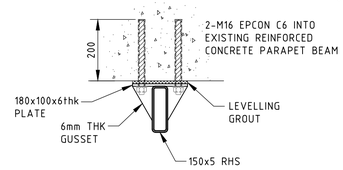

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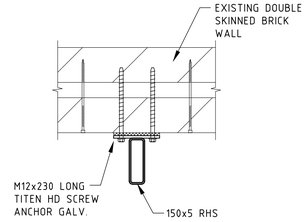

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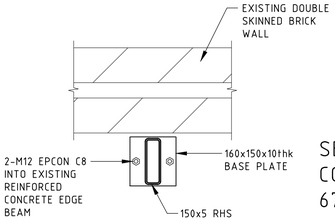

SECTION C
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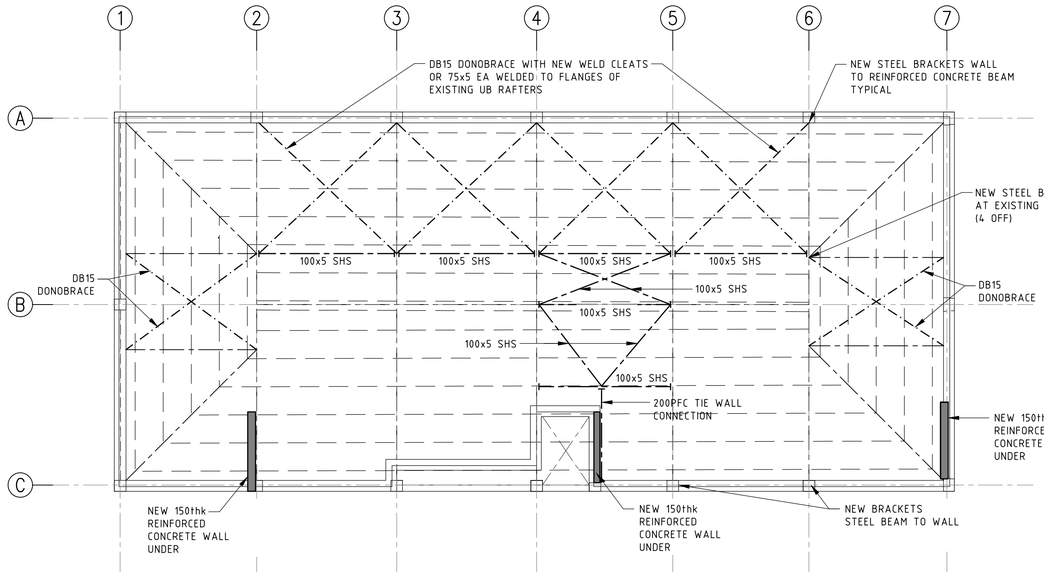
SEISMIC STRENGTHENING CONCEPT DRAWINGS 67%NBS (I.L.=2)



ALL NEW STEEL ROOF FRAMING/BRACING SHALL HAVE A NEW CLEATS/BRACKETS SITE WELDED TO EXISTING STEEL AT ALL JOINTS

PROPOSED ROOF STRENGHENING PLAN
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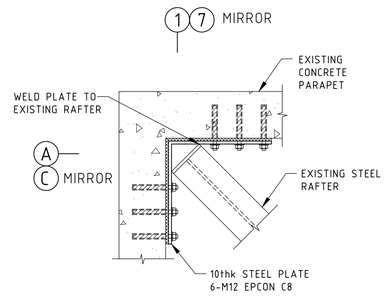
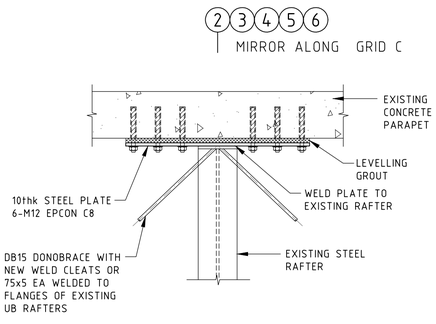
1:200 @ A3





BRACKET/PLATE TO EXISTING WALL
1:10 @ A1
1:20 @ A3

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NORTH ELEVATION
ROOF STRENGTHENING PLAN
AND DETAILS

DESIGNED	JRM	CAD FILE	76521-01
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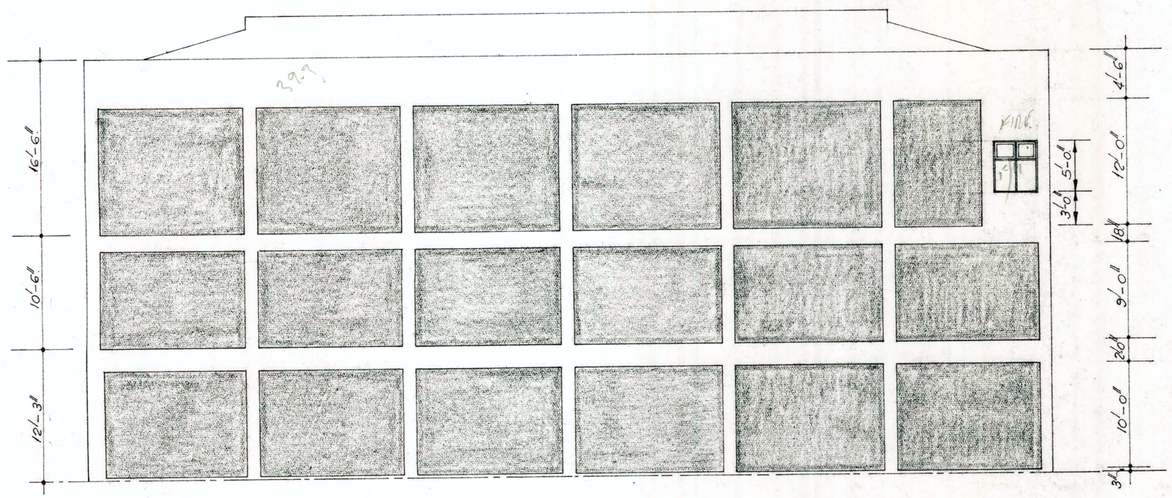
CHECKED	JRM	DATE	AUG 2021

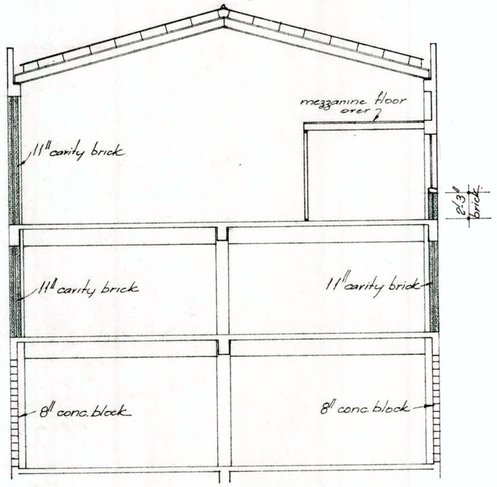
SCALE	SIZE
AS SHOWN	A1

DRAWING NUMBER	REV
75222-03	

Attachment – Original Drawings Dated 1963



NORTH ELEVATION.



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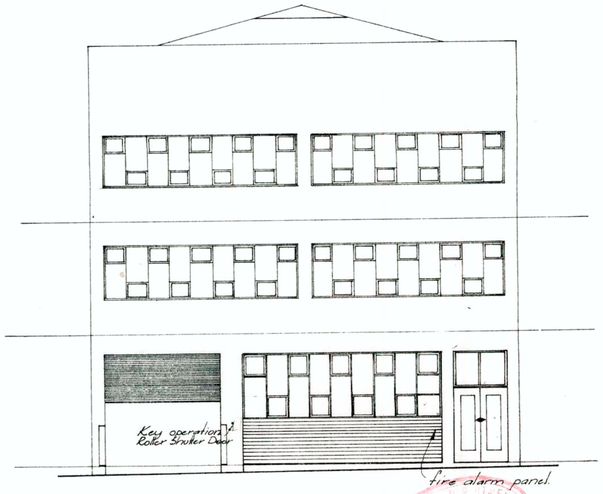
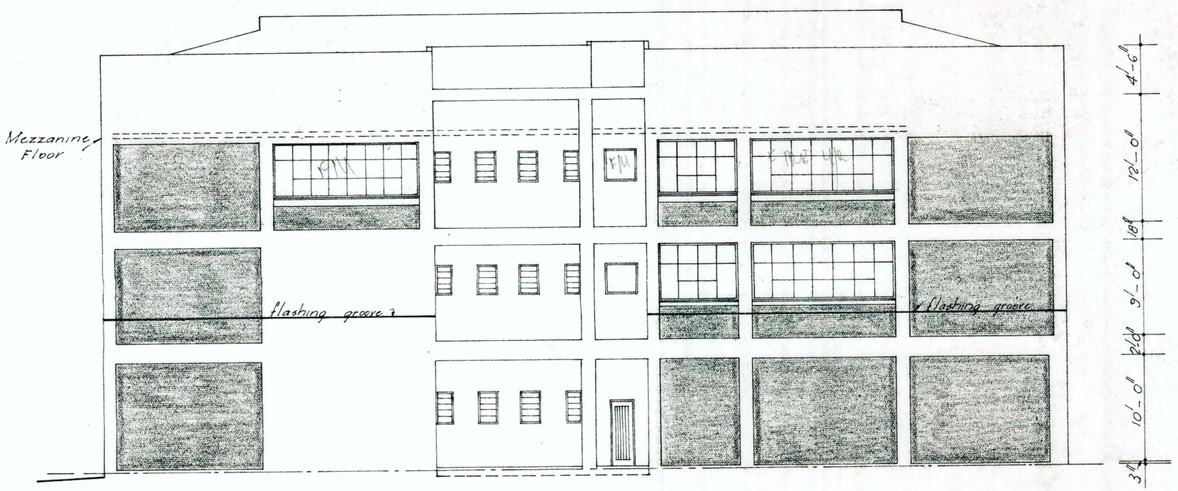
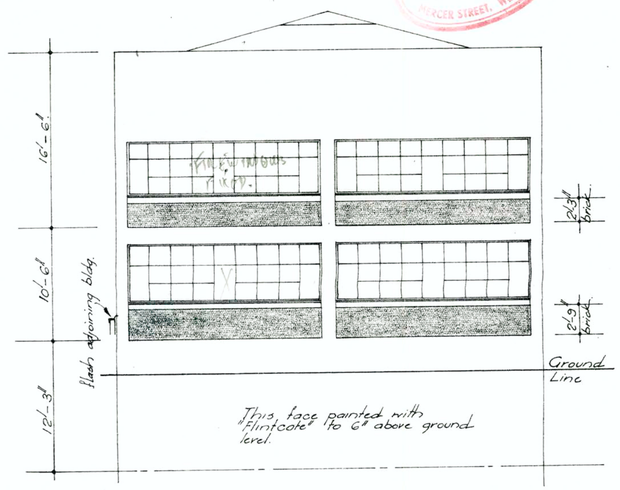


FRONT ELEVATION.



SOUTH ELEVATION.



REAR ELEVATION.

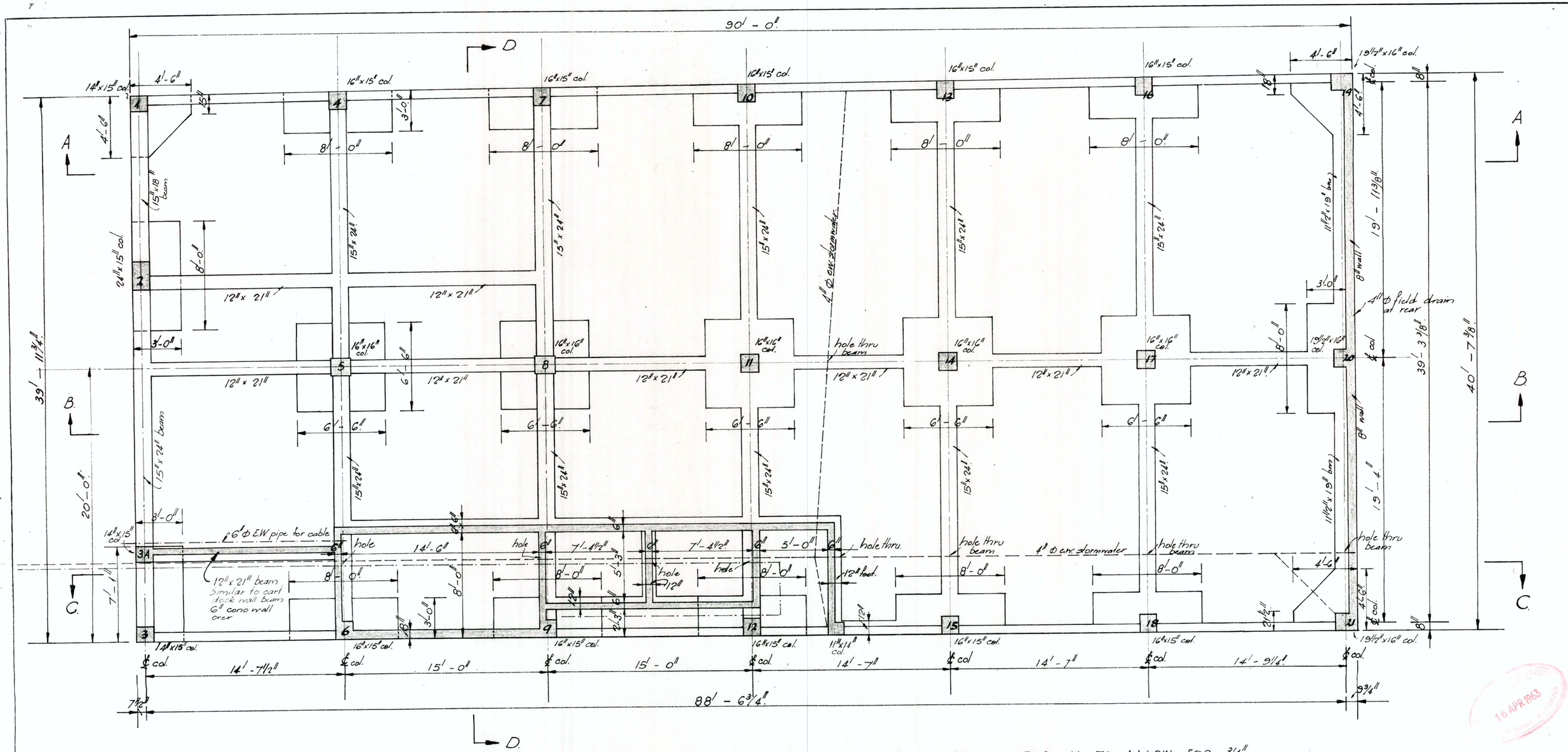
BOND & PARTNERS.
CONSULTING ENGINEERS.
P.P. 10/4/63

PROPOSED WAREHOUSE FOR WILLIAM H. LEE LTD. 6 NELSON ST.

ELEVATIONS.

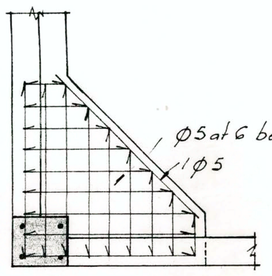
DRAWING No. J 520/3A		DATE March 1963.
CHKD	DRN DeTerte	SCALES 1/8" = 1'-0"

BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.

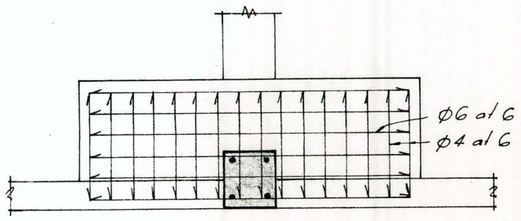


FOUNDATION LAYOUT PLAN.
1/4" = 1'-0"

CONTRACTOR IS TO ALLOW FOR ¾" PLASTER ON 4 OUTSIDE FACES.

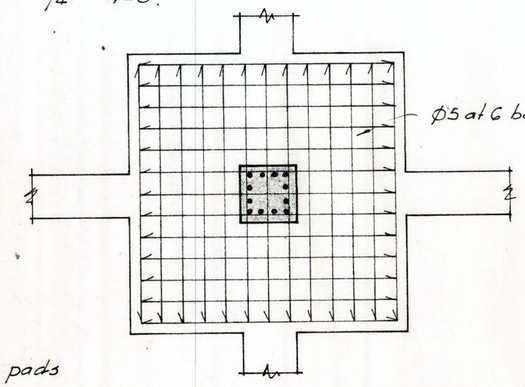


COL. PADS. 1, 19, 21.

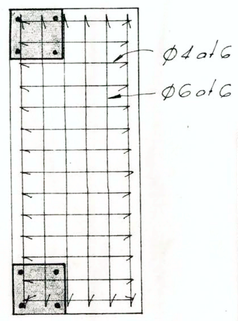


COL. PADS 4, 6, 7, 9, 10, 12, 13, 15, 16, 18. 2, 20.

NOTE:- All pads 24" thick.



COL. PADS. 5, 8, 11, 14, 17.



COL. PAD 3-3A

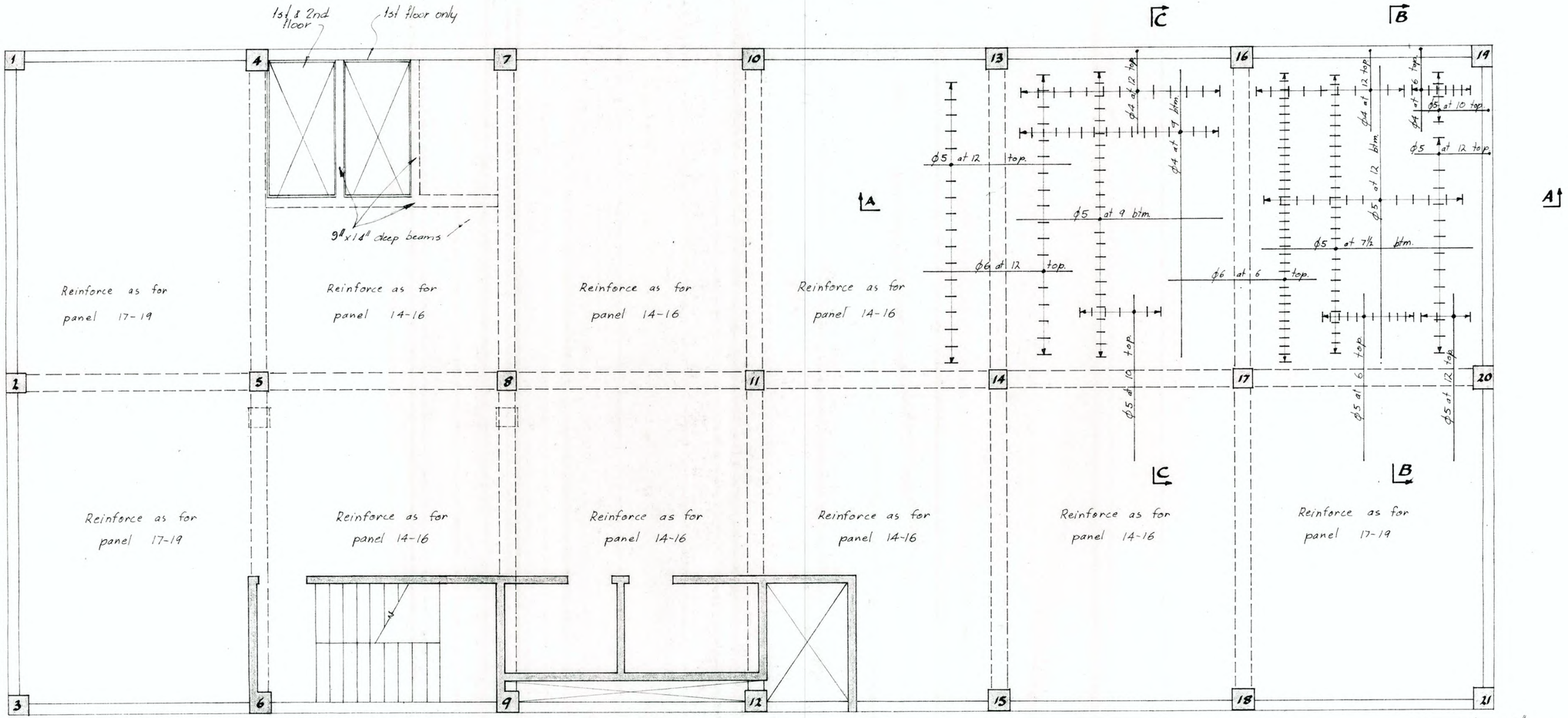
BOND & PARTNERS
CONSULTING ENGINEERS.
P.P. 10/4/63.

WAREHOUSE FOR WILLIAM H. LEE. LTD.
6 NELSON STREET

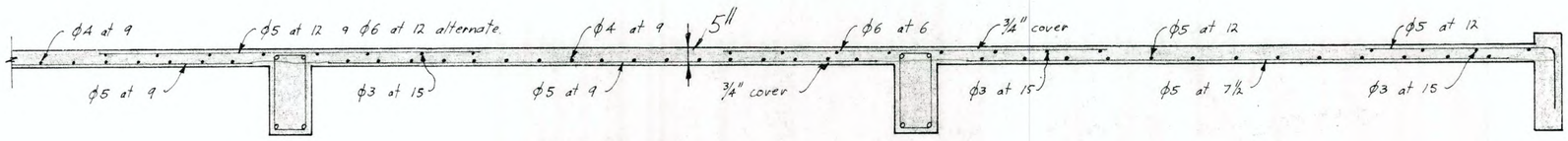
FOUNDATION PLAN

DRAWING No. 520/4A		DATE November, 1962
CHKD	DRN DeTerte	SCALES 1/4", 1/2" = 1'-0"

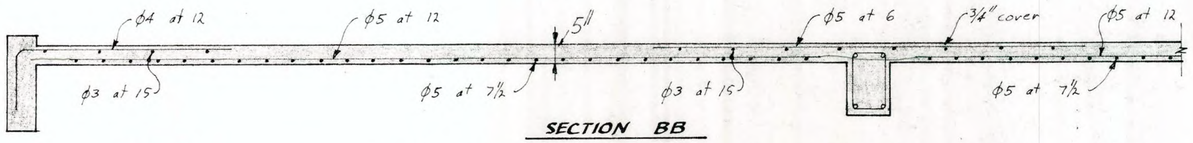
BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.



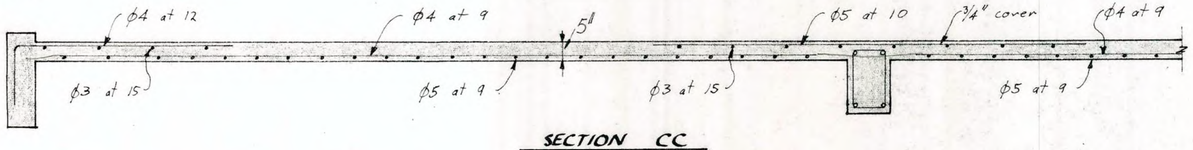
FIRST & SECOND FLOOR SLABS



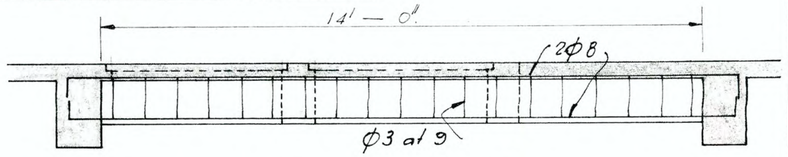
SECTION AA

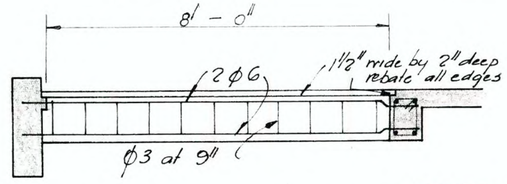


SECTION BB



SECTION CC





9" x 14" TRIMMER BEAMS TO HATCH WAY OPENINGS.

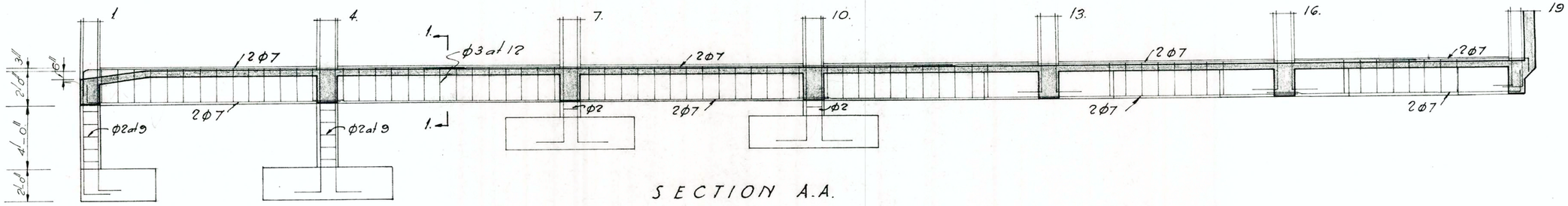
BOND & PARTNERS CONSULTING ENGINEERS P.P. 10/4/63

WAREHOUSE FOR WILLIAM H LEE LTD.
6 NELSON STREET

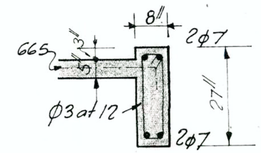
5" FLOOR SLABS

DRAWING No. J 520/6		DATE November, 1962
CHKD	DRN G.H.Tovey	SCALES ¼" & ½" to 1'-0"

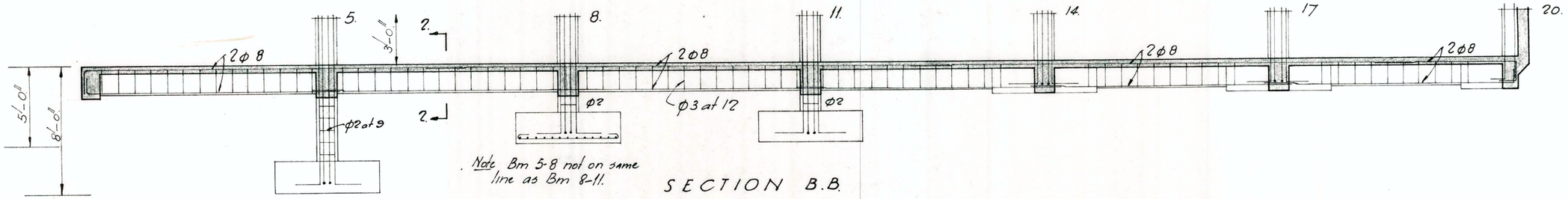
BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.



SECTION A.A.

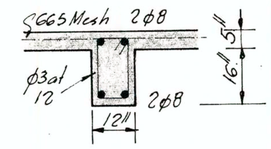


SECTION 1.1.

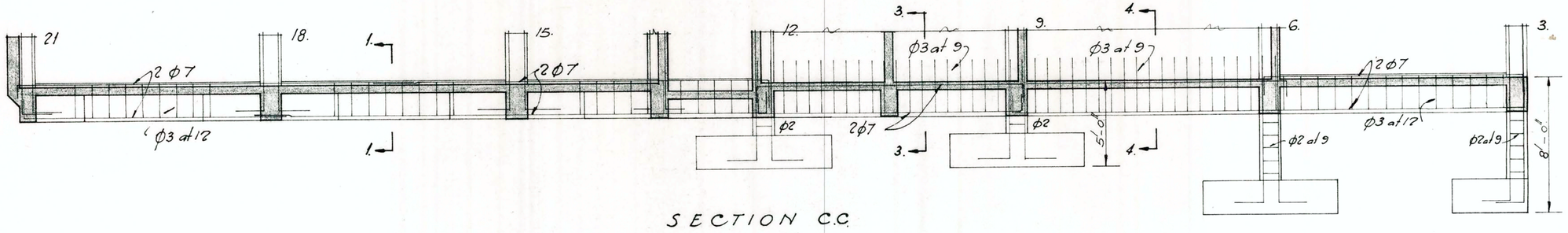


Note Bm 5-8 not on same line as Bm 8-11.

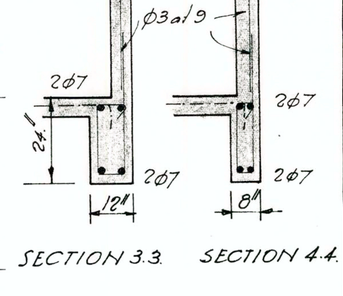
SECTION B.B.



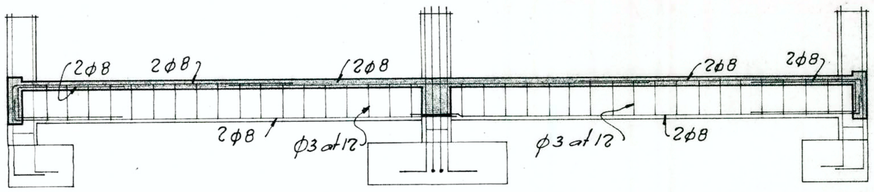
SECTION 2.2.



SECTION C.C.

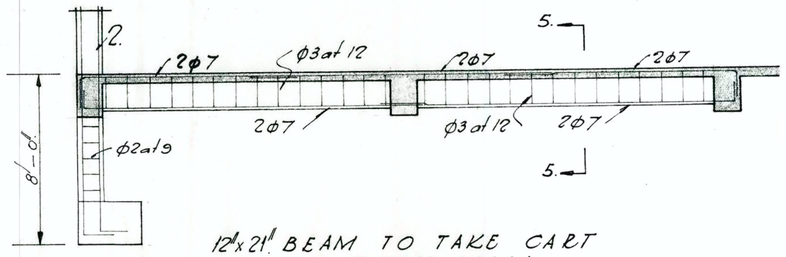


SECTION 3.3. SECTION 4.4.

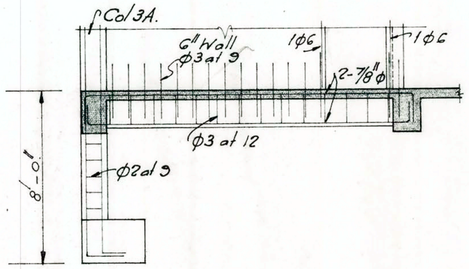


SECTION D.D.

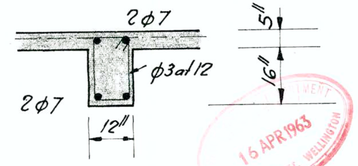
Tie beams 10-12., 13-15., 16-18., 4-6 similar.



12"x21" BEAM TO TAKE CART DOCK WALL.



12" x 21 BEAM AND WALL AT ENTRY.



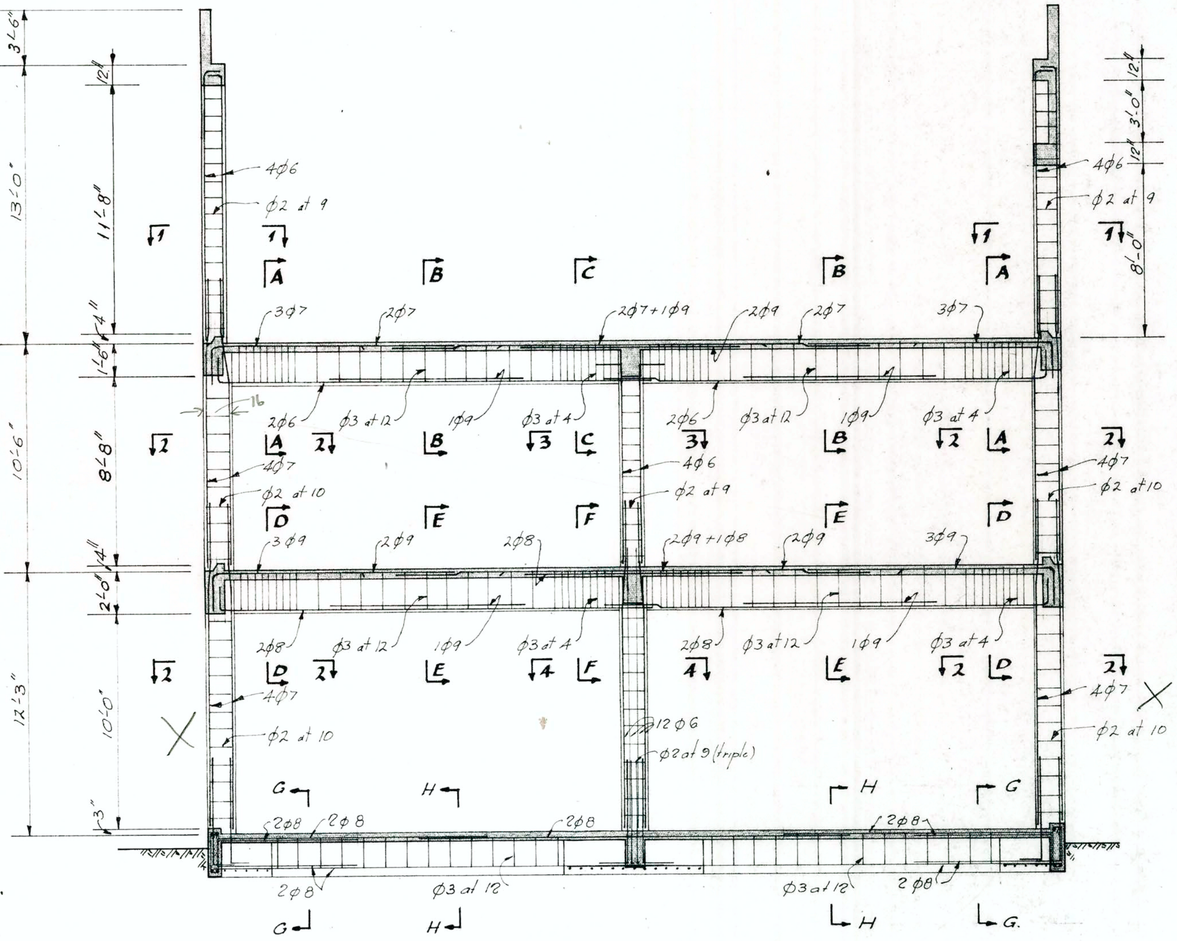
SECTION 5.5.

BOND & PARTNERS, CONSULTING ENGINEERS. P.P. 10/4/63.

Note. Footings to 6" walls of area, toilet block & staircase are as shown by Section 3.3 above.
Ground floor slab 5" reinforced with 665 Mesh to be laid in approved pattern.

WAREHOUSE FOR WILLIAM. H. LEE LTD. 6 NELSON STREET.		
FOUNDATION BEAMS. etc.		
DRAWING No. 520 / 5A		DATE December 1962
CHKD	DRN DeTerte	SCALES 1/4, 1/2" = 1'-0"

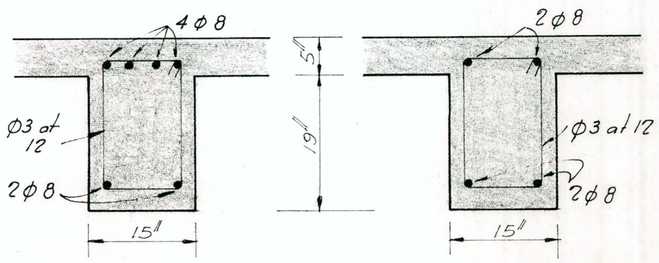
BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.

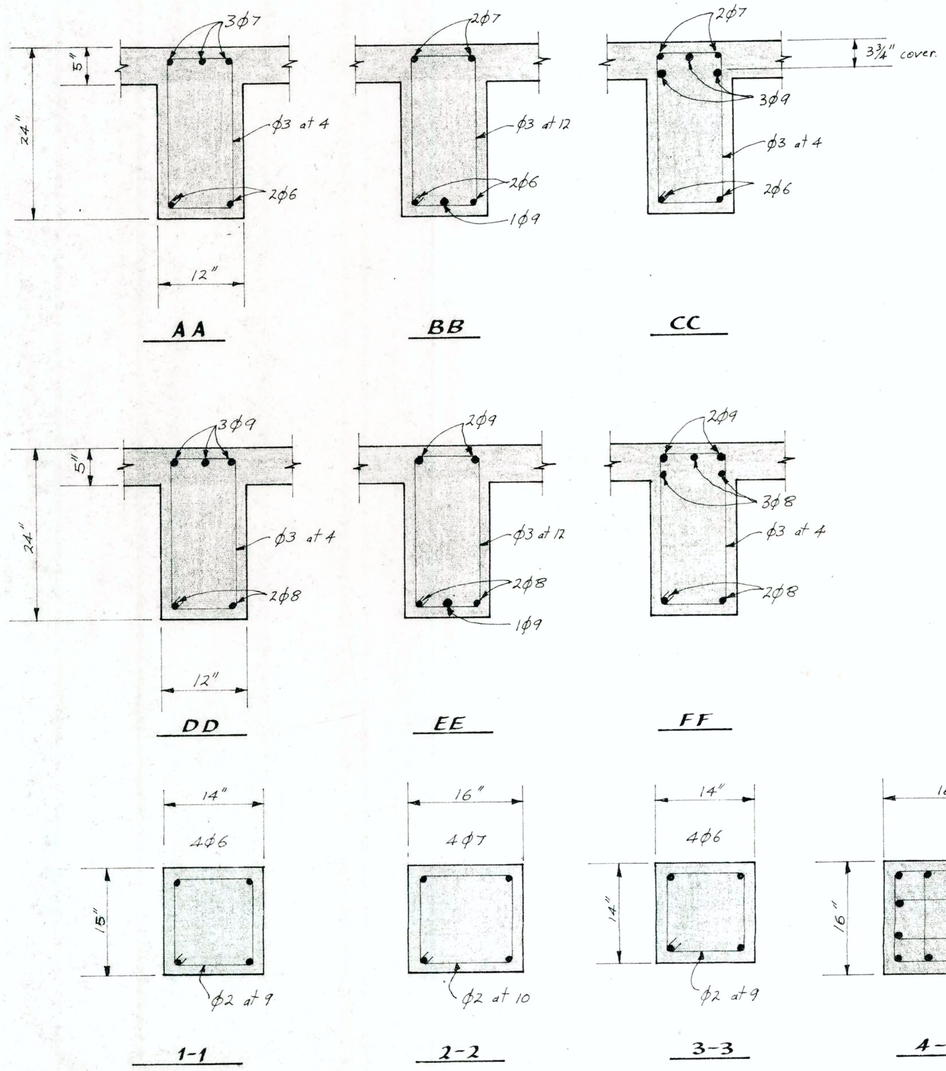


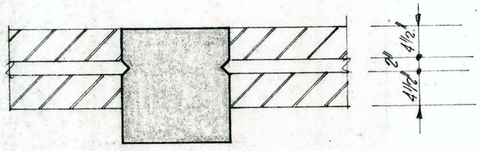
Foundations shown above refer to Cols 13~15, 16~18.
See Sheet 1520/5. for fdns to Cols 4~6, 7~9, 10~12.

Cols 4, 7, 10, 13, 16 **Cols** 5, 8, 11, 14, 17 **Cols** 6, 9, 12, 15, 18

FRAMES 4-5-6., 7-8-9., 10-11-12., 13-14-15., 16-17-18.







TYPICAL COLUMN FACE NOTCHING AT ALL BRICK PANELS.

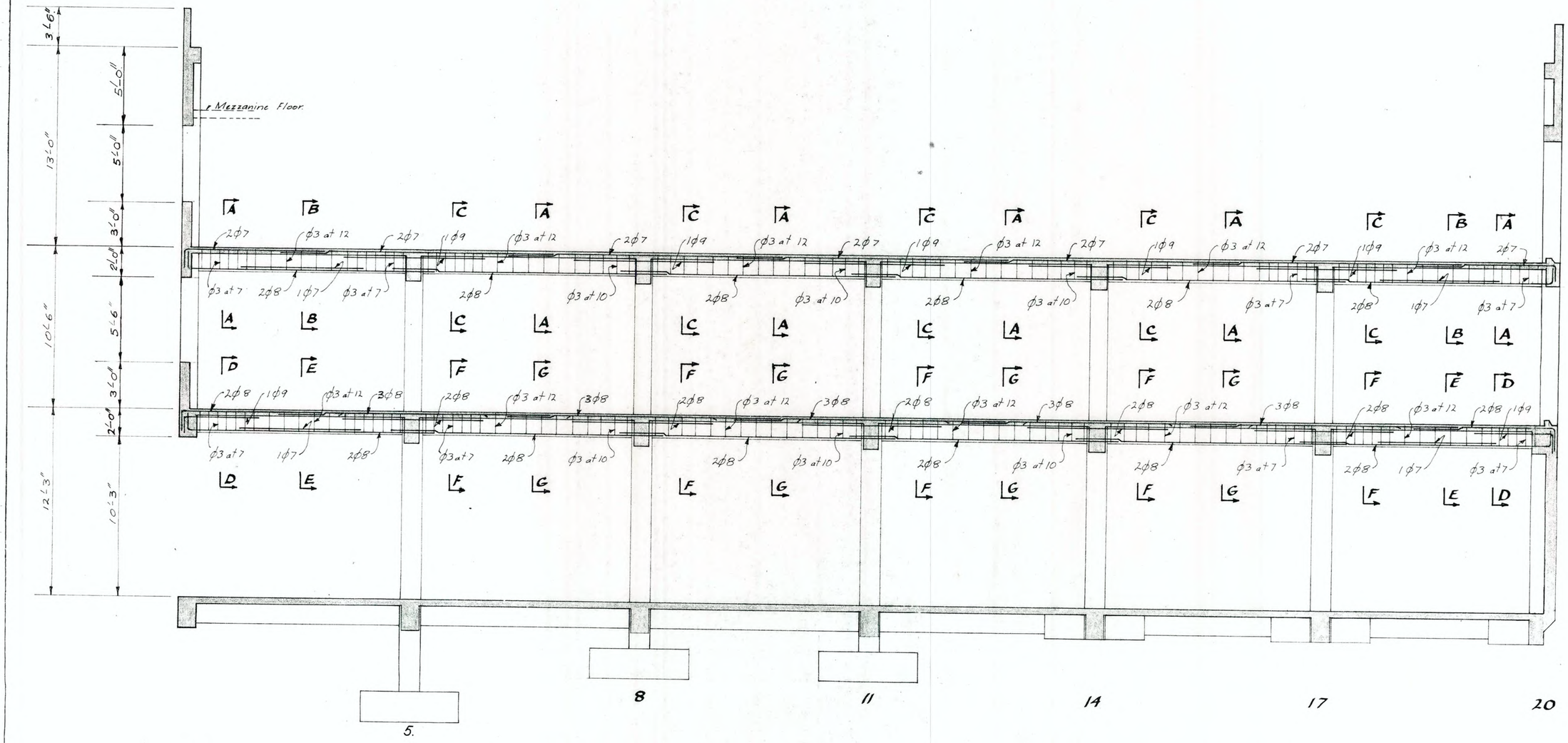
WAREHOUSE FOR WILLIAM H. LEE LTD.
6 NELSON STREET

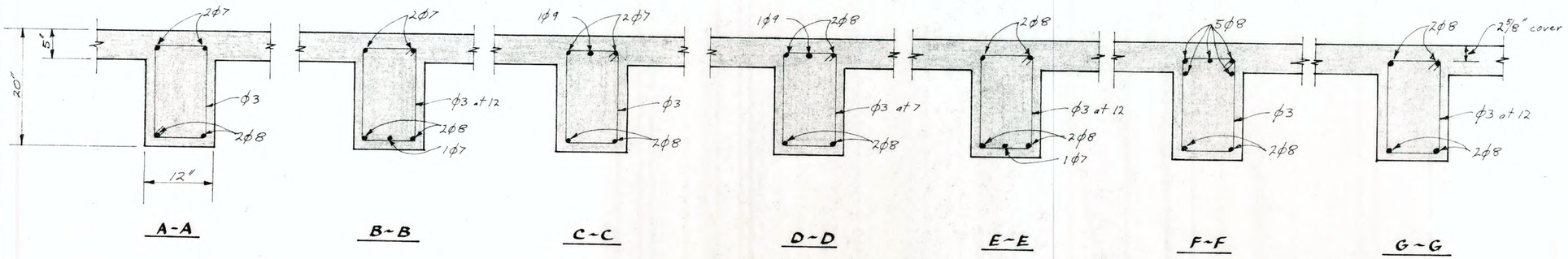
MAIN FRAMES "B"

DRAWING No. 1520/8A. DATE November, 1962

CHKD DRN J.B.Haney SCALES ¼" & 1" to 1'-0"

BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.





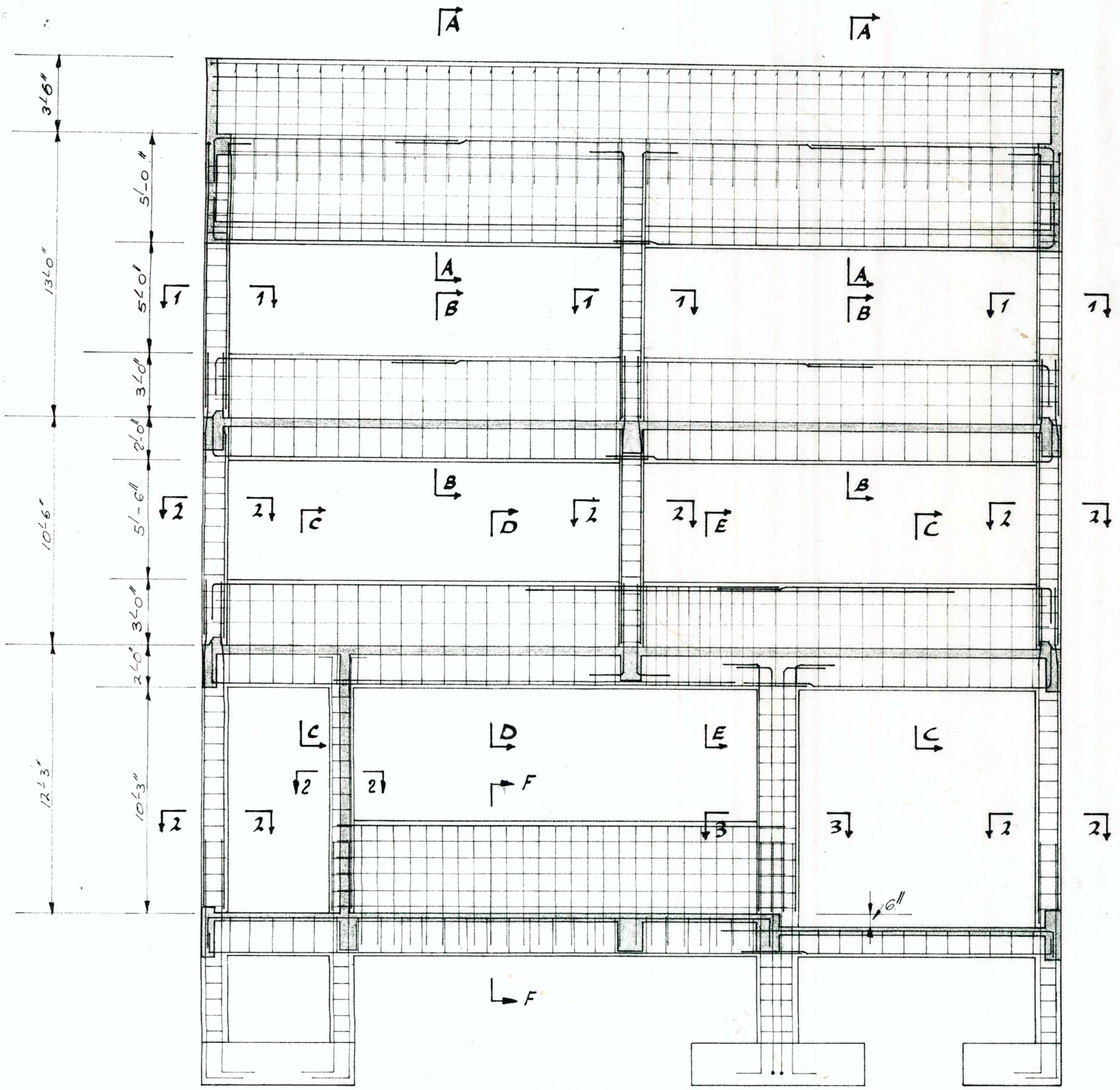
WAREHOUSE FOR WILLIAM H. LEE LTD.
6 NELSON STREET

LONGITUDINAL FRAME

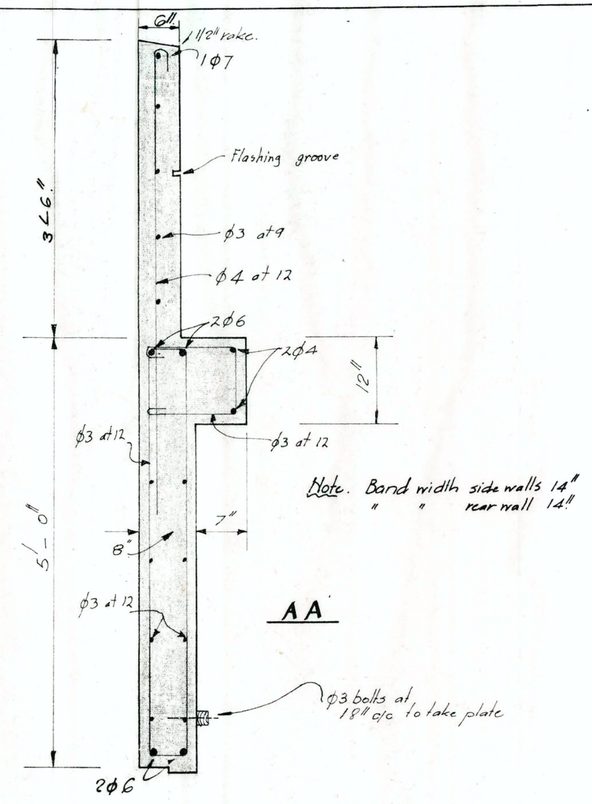
DRAWING No. J 520/ 9 A | DATE November, 1962

CHKD | DRN G.F.Hovey | SCALES ¼" & 1" to 1'-0"

BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.

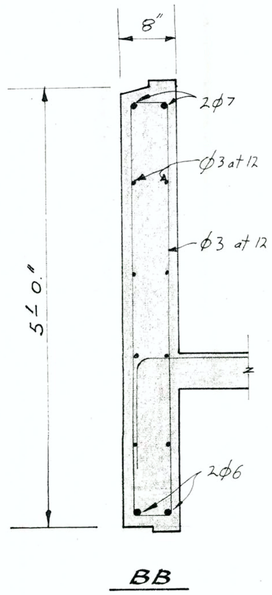


WEST WALL

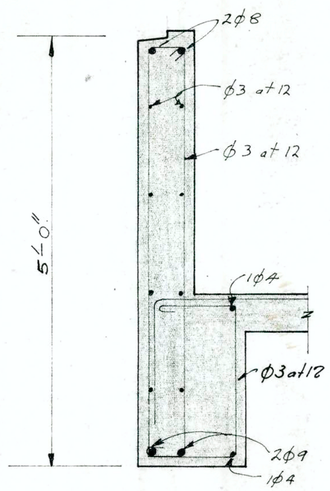


Note. Band width side walls 14"
" " rear wall 14"

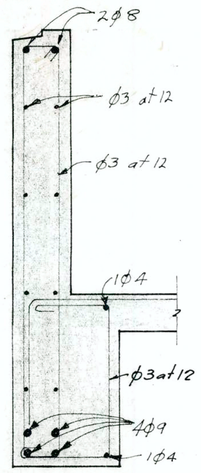
AA



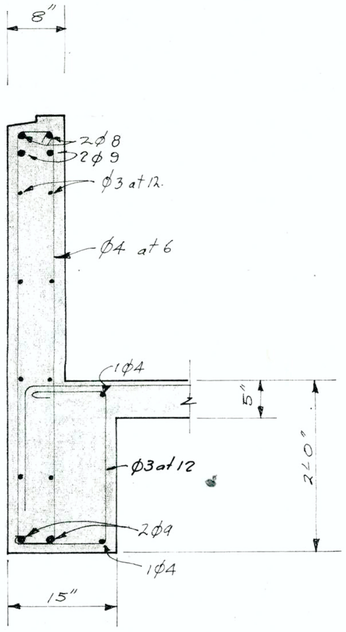
BB



CC



DD



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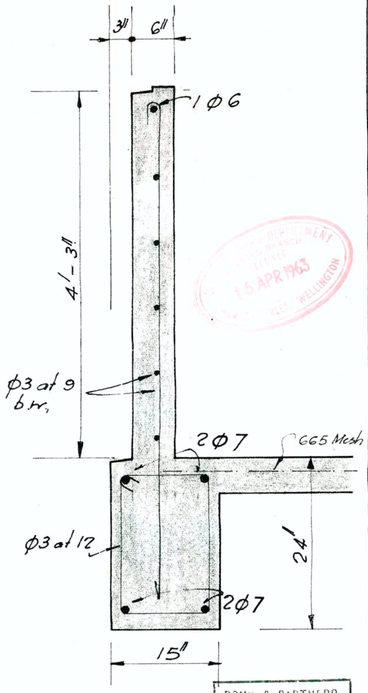

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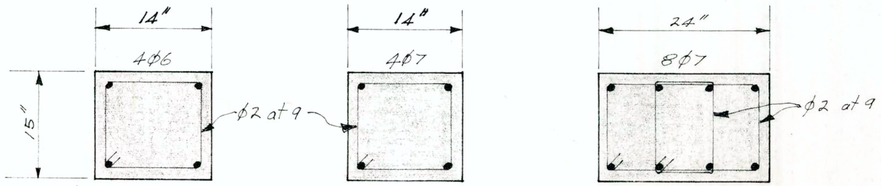
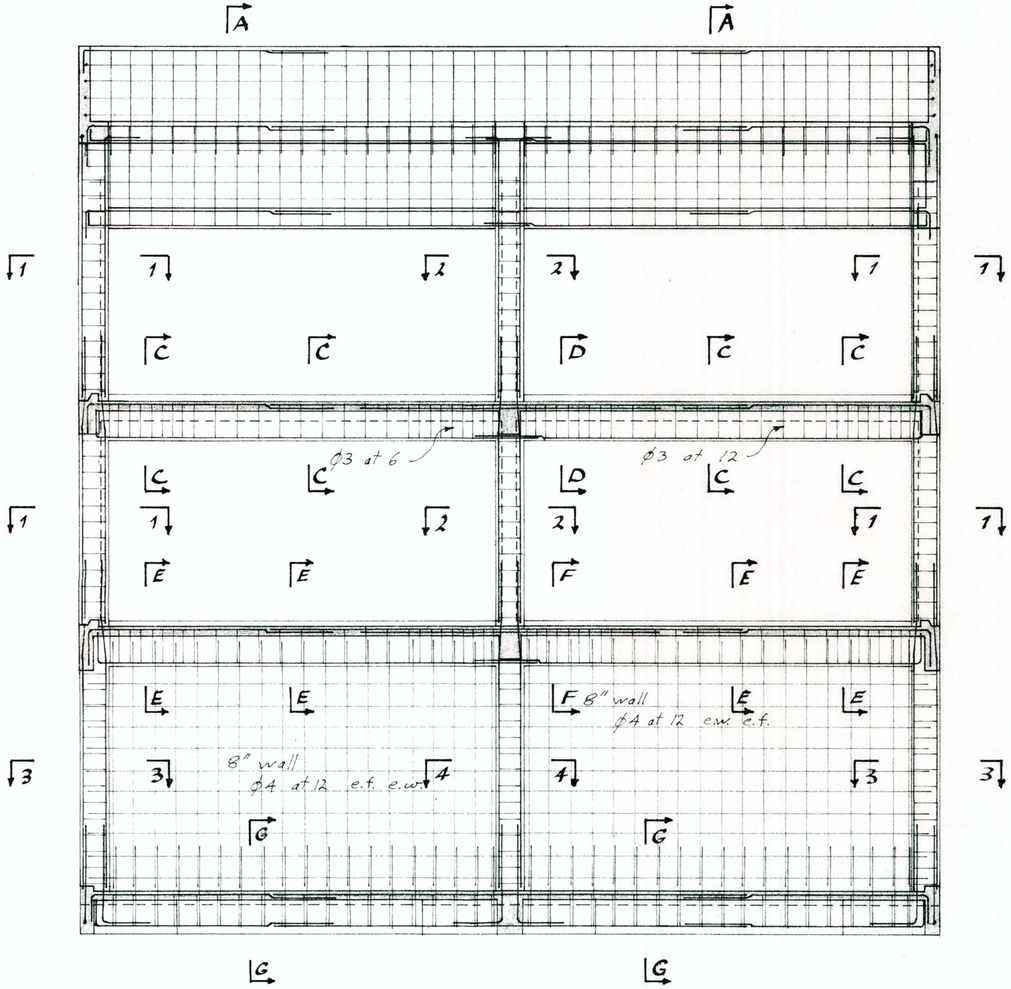


1-1 2-2 3-3

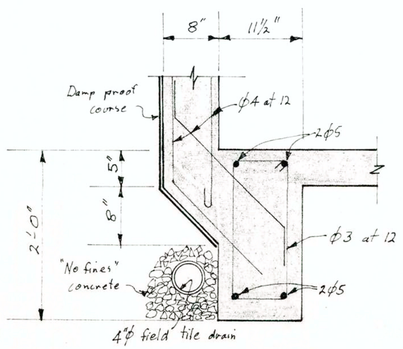
BOND & PARTNERS
CONSULTING ENGINEERS.
p.p. 10/4/63

WAREHOUSE FOR WILLIAM H. LEE. LTD. 6 NELSON STREET		
WEST WALL		
DRAWING No. J 520/10A.		DATE November, 1962
CHKD	DRN G.J.Bowey	SCALES 1/4" & 1" to 1'-0"

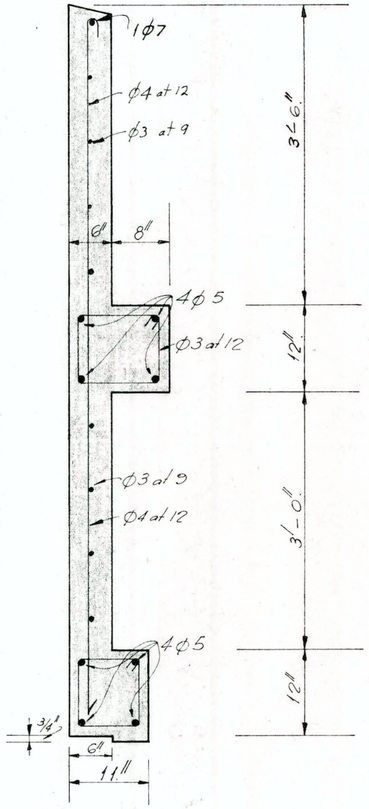
BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.



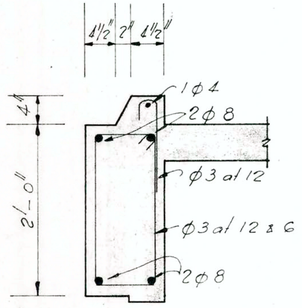
EAST WALL



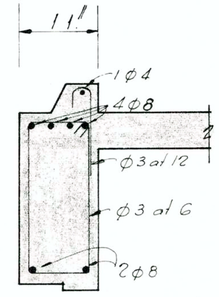
GG



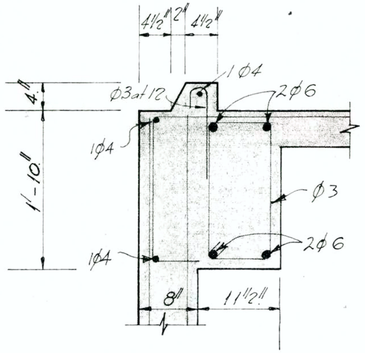
A - A



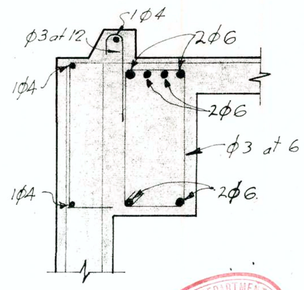
C - C



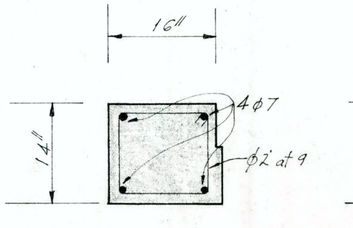
D - D.

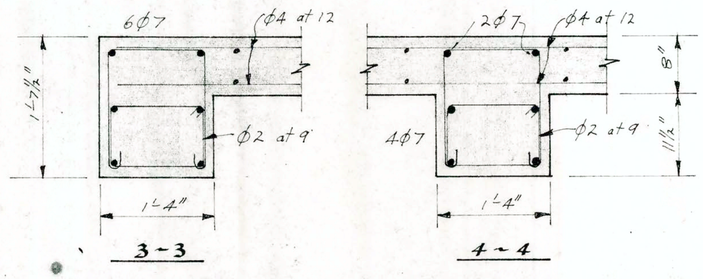


EE



FF

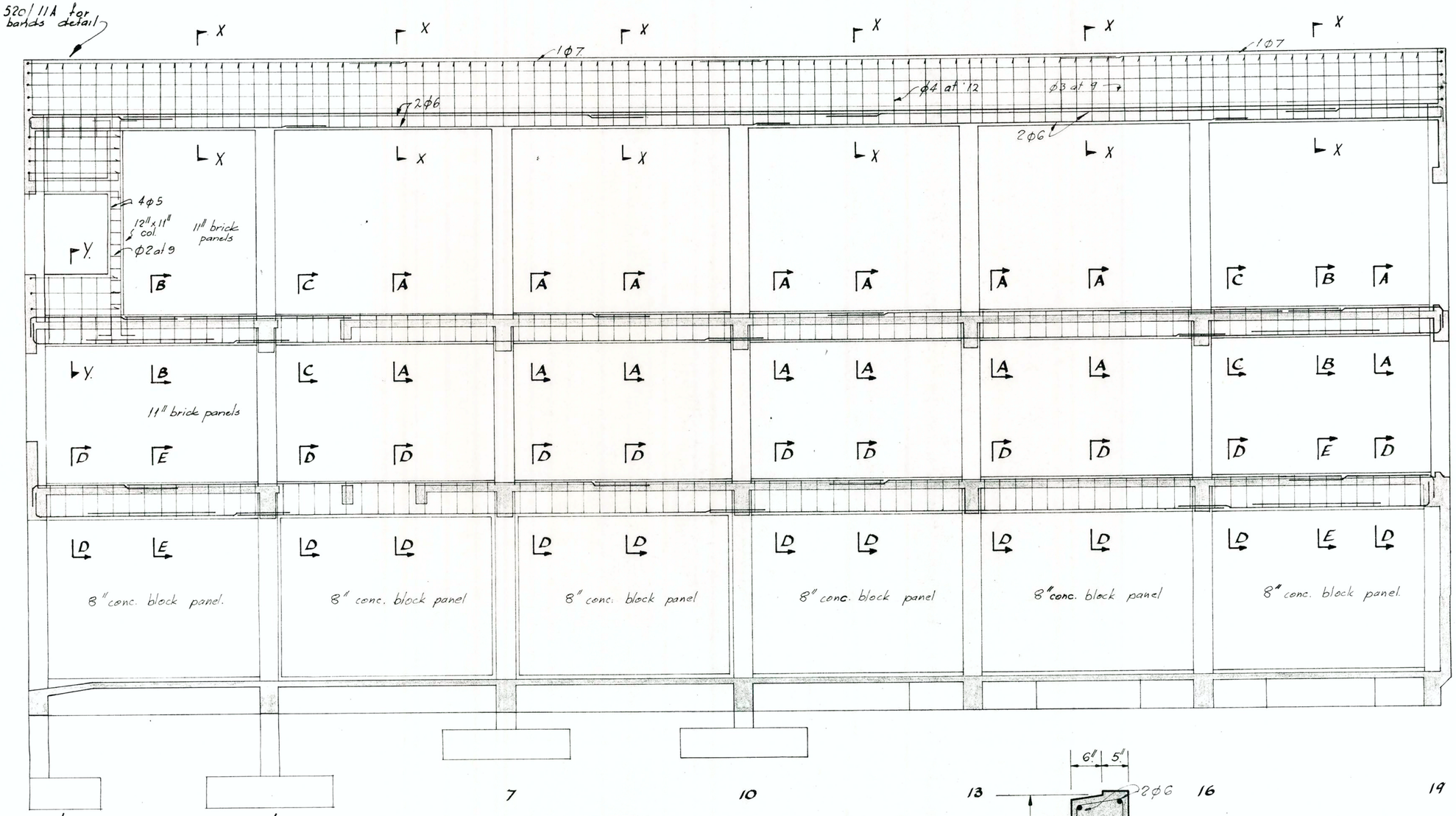


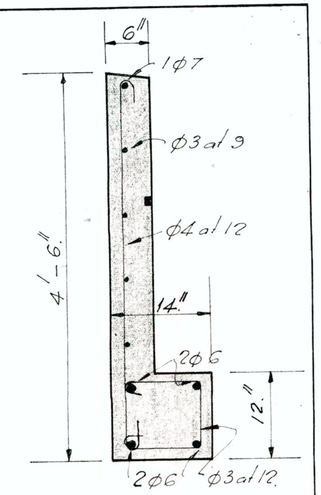


WAREHOUSE FOR WILLIAM H. LEE. LTD. 6 NELSON STREET		
EAST WALL		
DRAWING No. J 520 / 11A.		DATE November, 1962
CHKD	DRN J.D.Howey	SCALES ¼"= 1" to 1'-0".

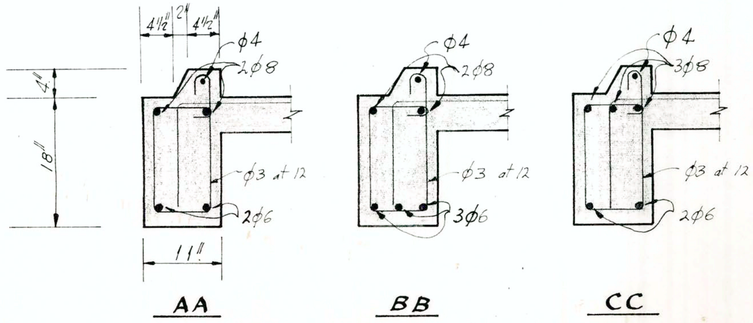
BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.

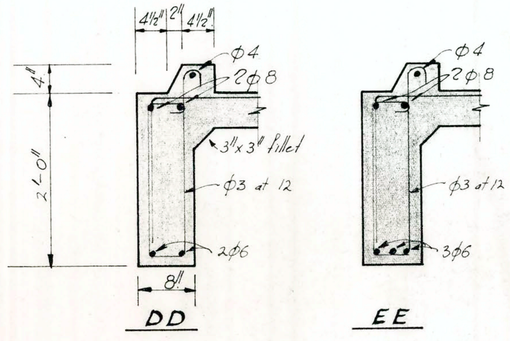
See Sheet 520/11A for parapet & bands detail

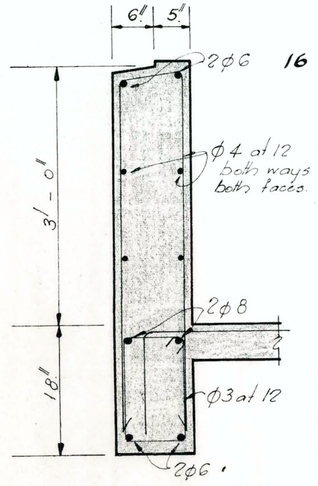




X-X.







Y-Y.

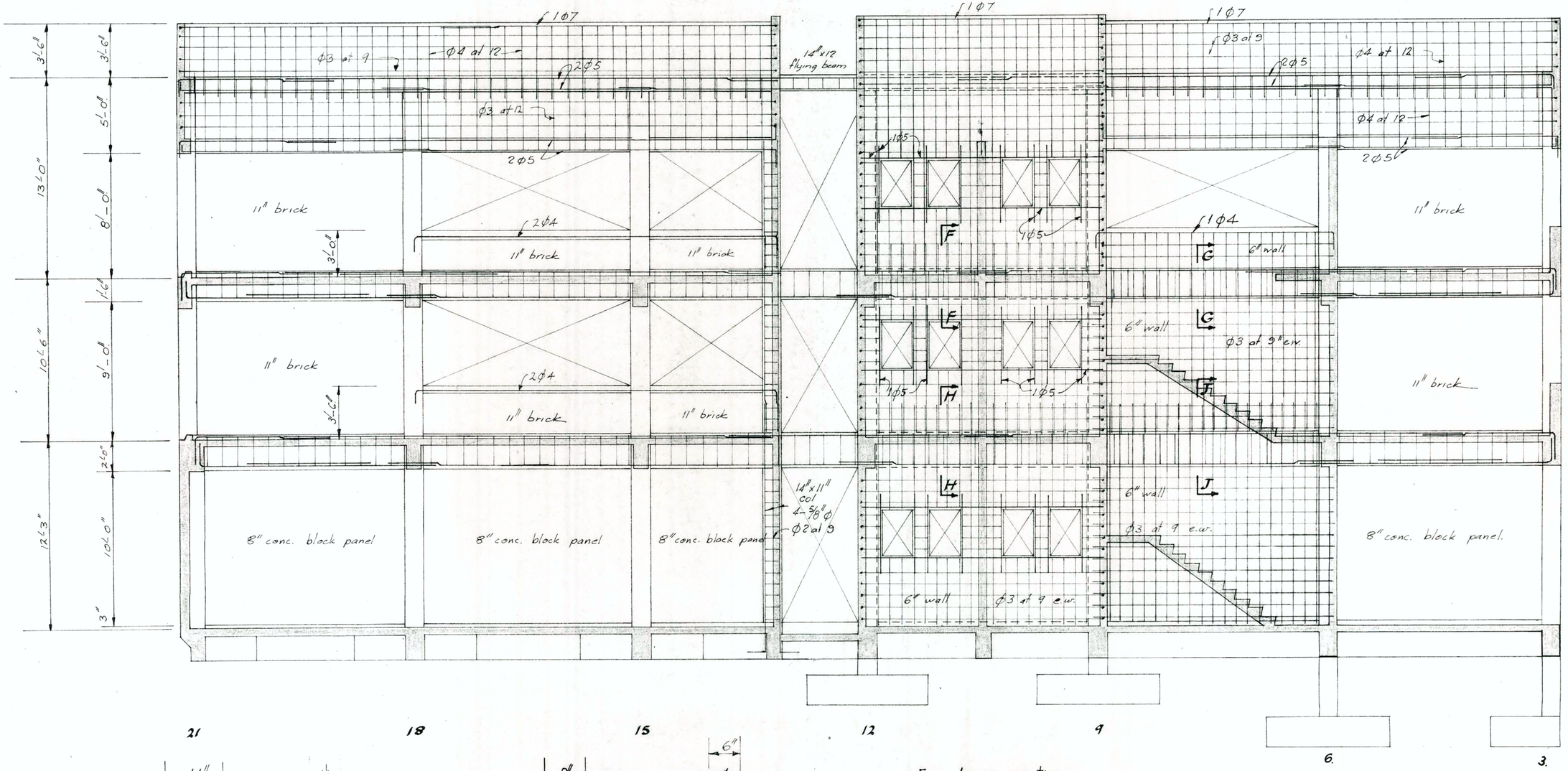
WAREHOUSE FOR WILLIAM H. LEE. LTD.
6 NELSON STREET

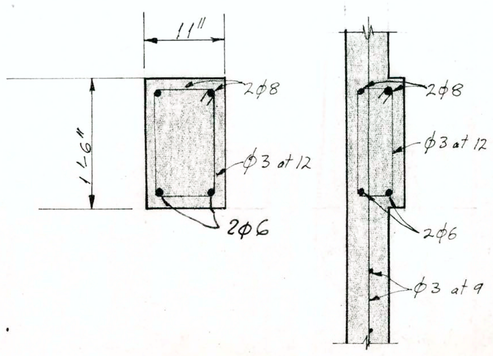
NORTH WALL

DRAWING No.: 520/12A		DATE November, 1962
CHKD	DRN	SCALES 1/4" & 1" to 1'-0"

BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.

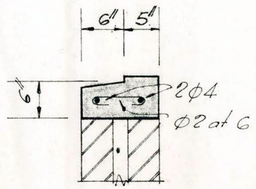
See Sheet 520/11A for sectional detail of parapet & bands. Set in lower band 5/8" ϕ bolts at 18" c/c to take timber plate for mezz. floor





SECTIONS F-F & H.H THROUGH OUTSIDE BEAM.

For beam sections see J520/12A.



CILL DETAIL ON BRICKWORK

WAREHOUSE FOR WILLIAM H. LEE. LTD.
6 NELSON STREET

SOUTH WALL

DRAWING No. J 520/13A. DATE November, 1962

CHKD DRN F.D.Harvey SCALES 1/4", 1" = 1'-0".

BOND & PARTNERS
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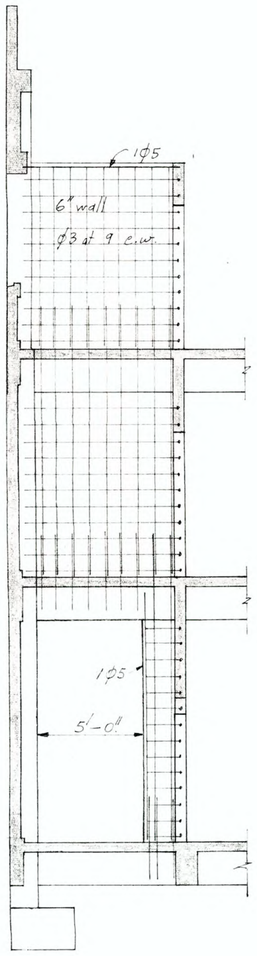
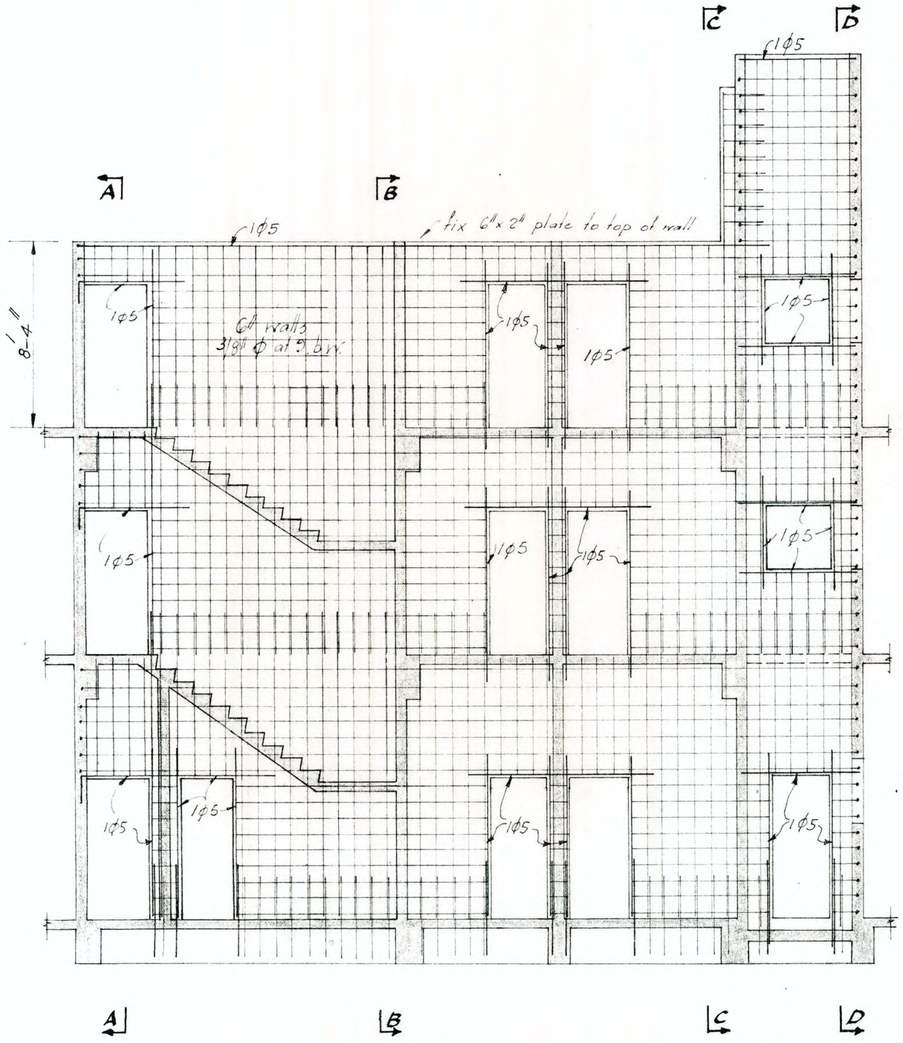
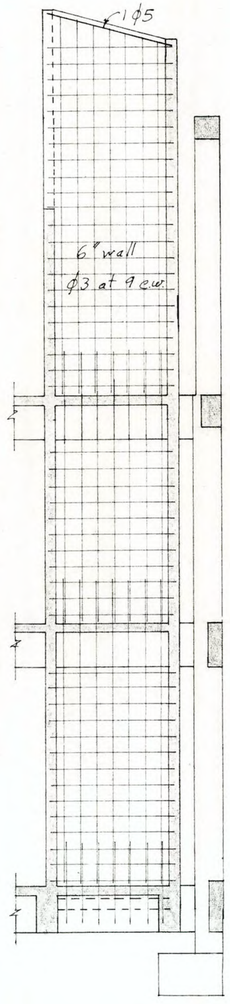


6

AA

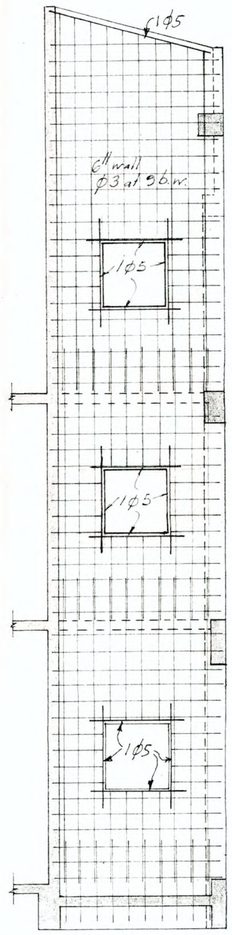


MAIN INTERNAL WALL TO STAIRCASE, TOILETS AND AREA.

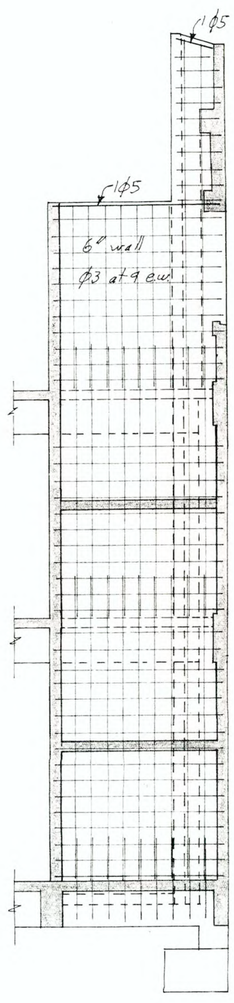


12

CC



DD



9

BB

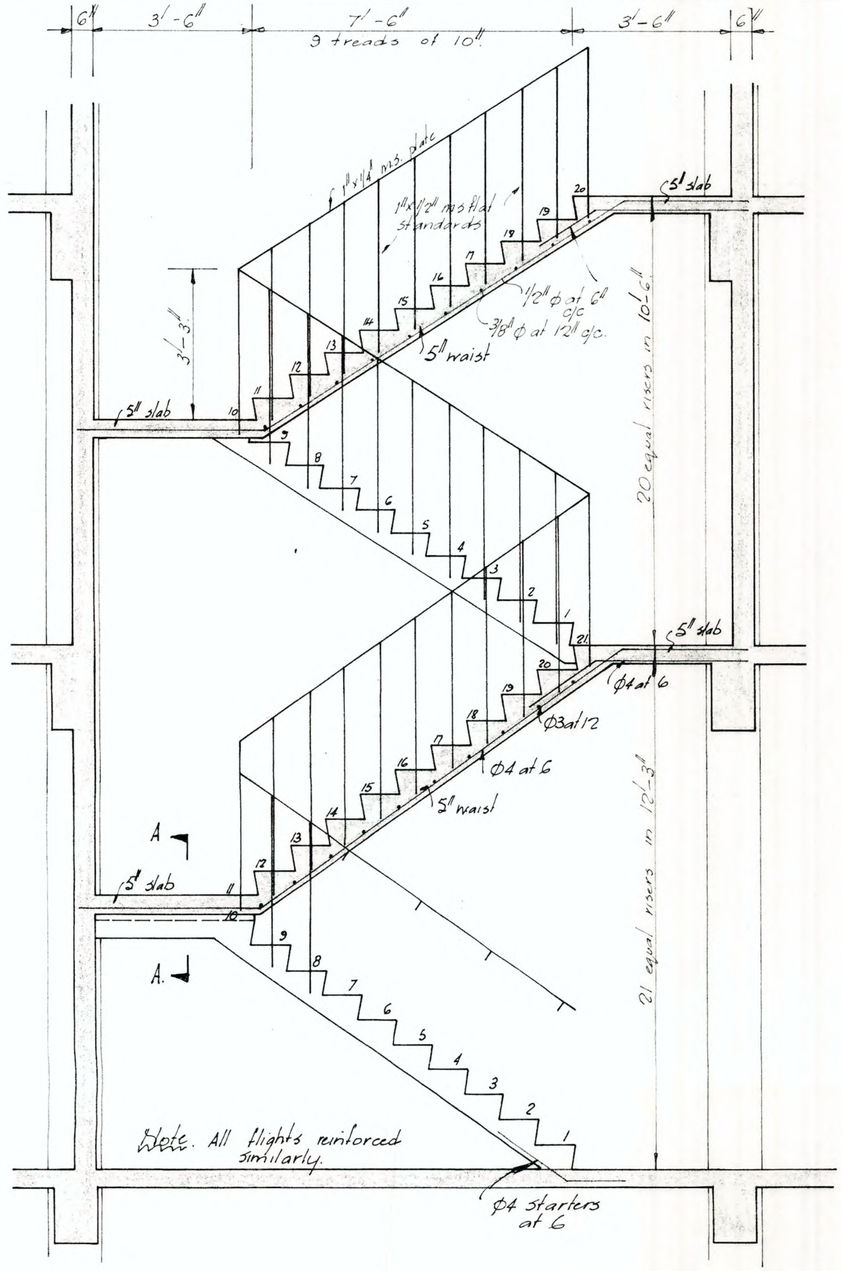
WAREHOUSE FOR WILLIAM H. LEE. LTD.
6 NELSON STREET

INTERNAL WALLS

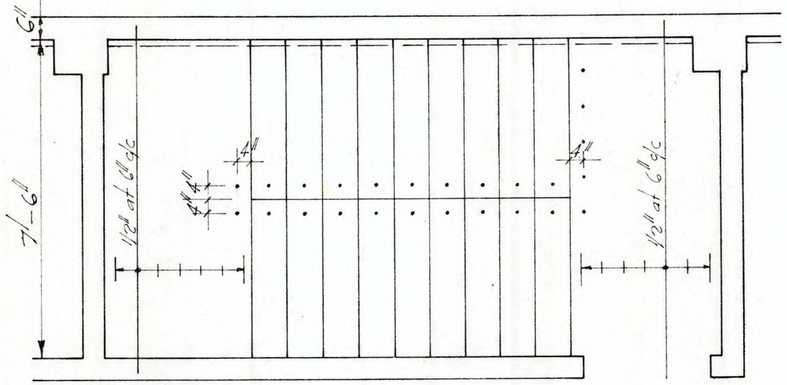
DRAWING No. J 520 / 14A. DATE November, 1962

CHKD DRN SCALES 1/4" to 1'-0"

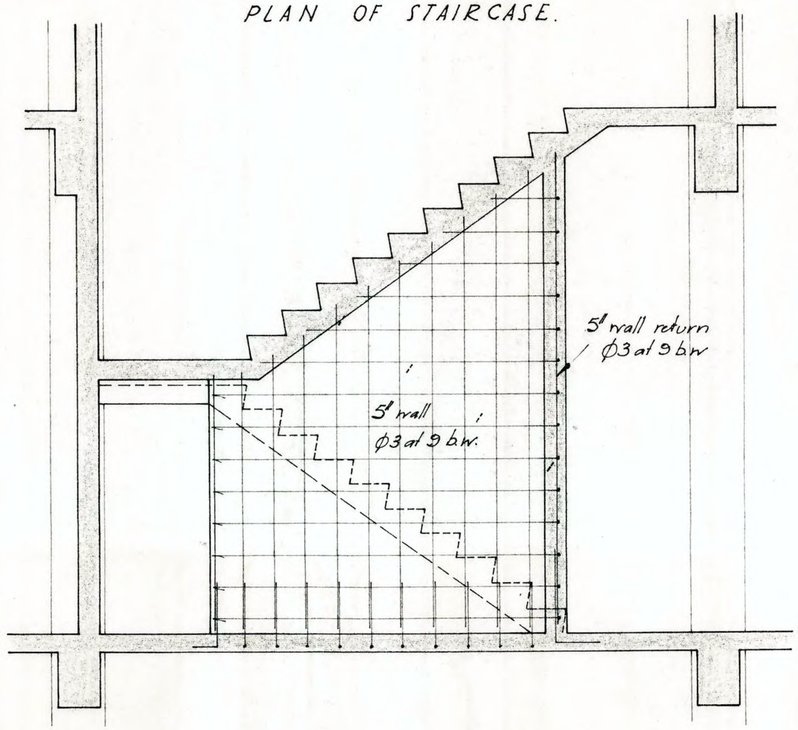
BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.



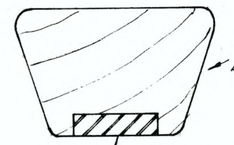
SECTION THROUGH STAIRCASE.



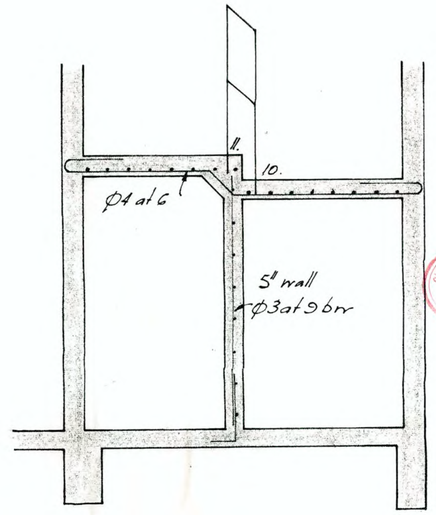
PLAN OF STAIRCASE.



INNER WALL TO SWITCHROOM.



HANDRAIL DETAIL
1" = 1'-0"



SECTION A.A.

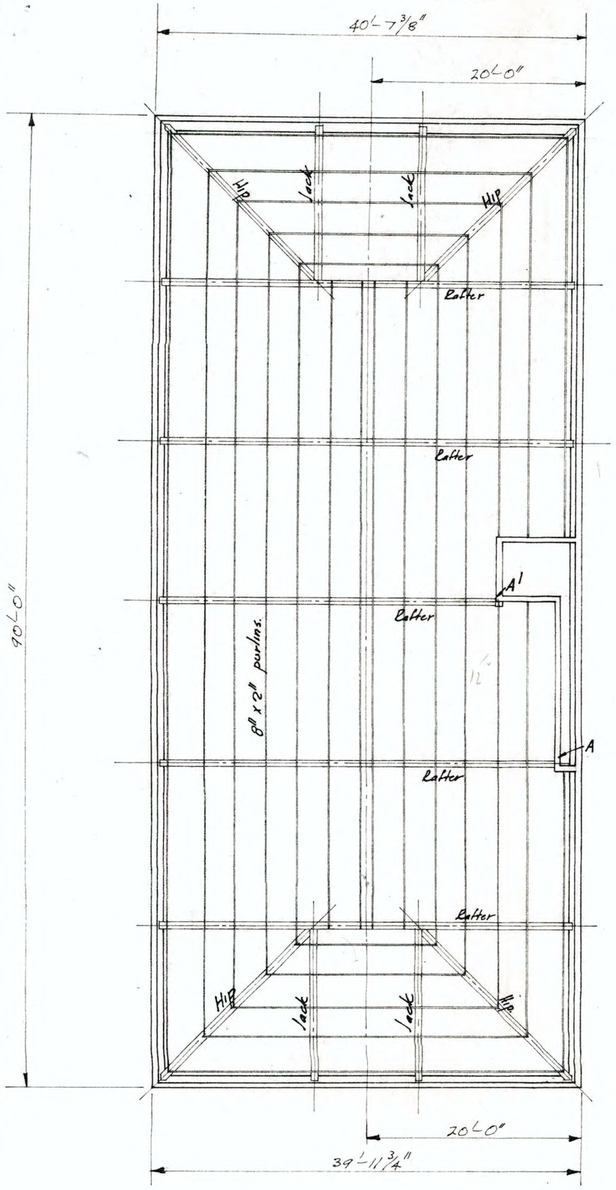
BOND & PARTNERS CONSULTING ENGINEERS. p.p. 10/4/63

WAREHOUSE FOR WILLIAM H. LEE LTD
6 NELSON STREET.

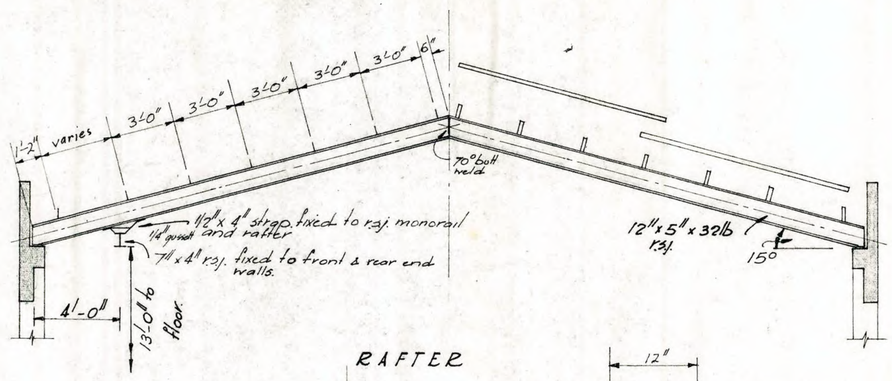
STAIRCASE DETAILS.

DRAWING No. J 520/15		DATE January 1962
CHKD	DRN. DeTerte	SCALES 1/2", 1" = 1'-0"

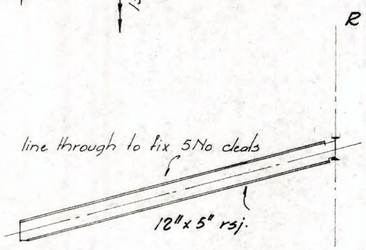
BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.



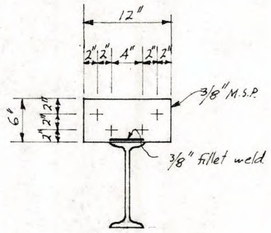
ROOF FRAMING PLAN
1/8" = 1'-0"



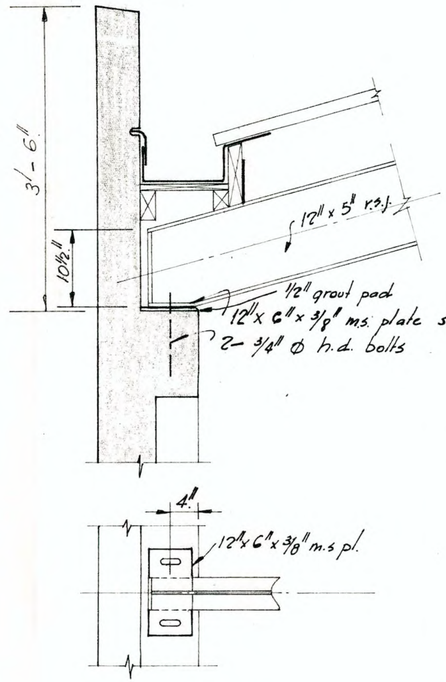
RAFTER



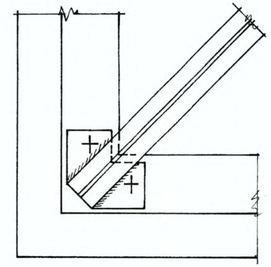
JACK



PURLIN CLEAT.

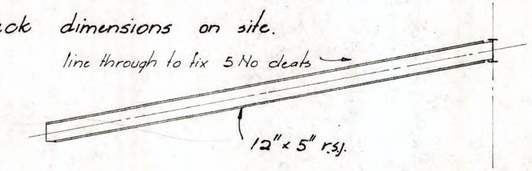


BASE PLATE.
RAFTERS & JACKS

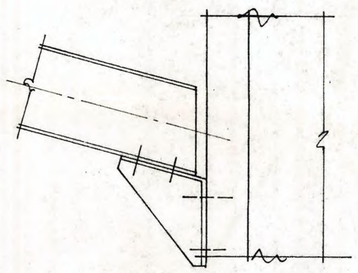


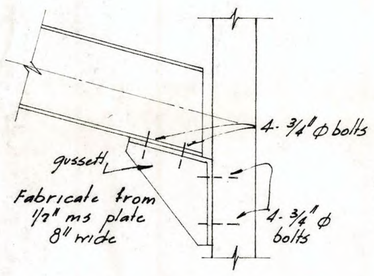
BASE PLATE
HIPS.

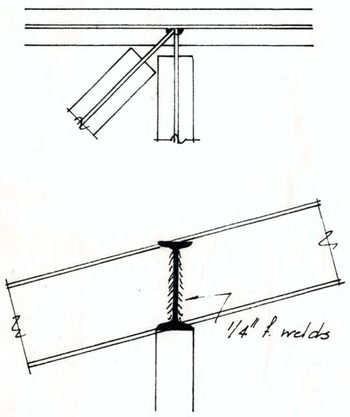
check dimensions on site.



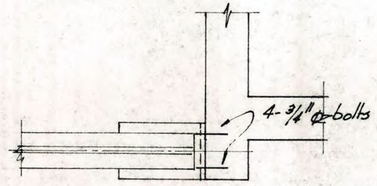
HIP



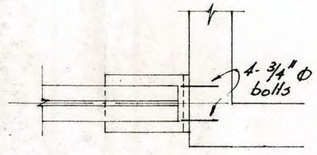




HIP & JACK CONNECTION



CORBEL AT "A1"



CORBEL AT "A"

BOND & PARTNERS CONSULTING ENGINEERS p.p. 10/4/63

WAREHOUSE FOR WILLIAM H. LEE. LTD.
6 NELSON STREET

ROOF DETAILS

DRAWING No. J 520 / 16.A.		DATE November, 1962
CHKD	DRN Details	SCALES 1/8" 1" = 1'-0"

BOND & PARTNERS
CONSULTING CIVIL & MECHANICAL ENGINEERS
20 FITZHERBERT TERRACE WELLINGTON, N.Z.

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