NEW ZEALAND'S FIRST POST-TENSIONED CONCRETE MASONRY HOUSE

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SUMMARY

Following two doctoral studies that have investigated the in-plane seismic response of post-tensioned concrete masonry, a consortium of product suppliers have collaborated with the University of Auckland and the not-for-profit organisation Habitat for Humanity to construct New Zealand's first post-tensioned concrete masonry house. A feature of this design is that all incorporated products are readily available in the market, with no proprietary products having been specifically developed for prestressed masonry. Consequently, it is hoped that this house will be a showcase for exposure of the technology in New Zealand. This paper discusses the post-tensioning details and comments on aspects of the design and construction.

INTRODUCTION

The primary objective of this research was to design and construct the first post-tensioned concrete masonry (PCM) house in New Zealand, a country of high seismicity. Previous research conducted at the University of Auckland focused on developing a PCM wall system using common construction materials and within the scope, or at least following the notion, of current New Zealand design practices [1]. In New Zealand, reinforced concrete masonry houses are currently designed using force-based methods and the masonry design standard NZS 4230 [2], or for simpler structures NZS 4229 [3], the non-specific masonry design standard. The PCM house was designed using a force-based design method, supported by research findings, guidance from NZS 4230 and adapting the notion of NZS 4229 to prestressed masonry. The 2004 edition of NZS 4230 contains a normative appendix on prestressed masonry that assisted in the design procedure. The design, detailing and construction of the house are reported in this paper.

Post-tensioned masonry walls gain their lateral strength and desirable seismic properties through the utilization of vertical unbonded post-tensioning. In-plane cyclic loading of an unbonded post-tensioned cantilever wall results in a horizontal crack forming at the wall-foundation interface. This leads to rocking behaviour that results in large drift capacity and reduced and localised wall damage, with the wall returning to its original vertical alignment at the conclusion of loading provided that sufficient residual prestress remains in the tendons. Laboratory testing investigating the in-plane cyclic and dynamic response of both concrete masonry and clay brick post-tensioned walls has demonstrated these characteristics, for example see [4-6].

PROJECT DETAILS

The PCM house was located in South Auckland, New Zealand and was the result of an alliance between Habitat for Humanity Manukau (HfHM), the University of Auckland and numerous consulting and material supply companies. Founded in 1976, Habitat for Humanity is a global non-profit non-denominational Christian housing organisation who assist low-income families into their own homes by providing a 'hand up' rather than a 'hand out' [7].

The single storey house comprised of a simple floor plan as shown in Figure 1, having outside dimensions of 15.6 m by 7 m. The exterior walls were constructed of PCM, while the interior partitions were triboard, a high density timber panel and the traditional material of choice for HfHM. Formblock® mortarless concrete masonry blocks [8] were used in the exterior wall construction. This was one of the first projects to use this block type, which had only recently appeared in the New Zealand market. The use of mortarless blocks is currently outside the scope of the non-specific masonry design standard, and therefore a specific design was required for this project. The floor consisted of a concrete raft design, typical of the floors used in residential masonry construction in this area of New Zealand. A simple timber truss roof with iron cladding enclosed the structure.

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DESIGN DETAILS

Floor

The plan view of the concrete raft floor design is shown in Figure 2, with the corresponding details 1 through 4 shown in Figure 3. The raft floor sat on the ground surface and comprised of an edge beam and internal ribs having a depth of 285 mm, with an 85 mm floor thickness, as shown in detail 1 (Figure 3). Polystyrene blocks were used to form the 100 mm wide internal ribs and provide additional insulation to the slab. The existing site had a sloping profile and two public drains running through the proposed house footprint, resulting in a more complex floor design. Additional ground beams and piles were required at both ends of the floor to provide bridging of a 900 mm diameter storm water pipe and a 150 mm diameter sewer connection. The ribs shown hatched depict the 100 mm wide internal bridging beams, consisting of two D16 (16 mm diameter) steel reinforcing bars and 6 mm diameter stirrups at 120 mm centres. A foundation retaining wall was required around the south and part of the east and west sides of the floor due to the existing site slope. This wall is indicated by a single diagonal hatch in Figure 2, with the corresponding detail given as detail 4 in Figure 3. The bridging and foundation wall design were consistent with common practices.

Walls

The exterior walls were constructed from Formblock® mortarless concrete masonry blocks providing a wall thickness of 190 mm. A plaster finish was applied to the exterior of the building, with insulation and lining installed on the interior face of the block walls. Figure 4(a) shows a Formblock® having a length of 398 mm and a height of 200 mm, and half height webs to enable the placing of horizontal bond beam steel reinforcement. The horizontal connection of adjacent blocks is shown in Figure 4(b), where plastic inserts lock the blocks together and locate the vertical reinforcement. The Formblock® geometry results in vertical reinforcement being located at the block ends as illustrated in Figure 4(b), or at the block centre.

The walls were fully grouted using a specially developed block fill concrete that exhibits low shrinkage and self-compaction properties and has a target 28 day compressive strength of 20 N/mm^2. The use of mortarless blocks required full grouting of the walls. The blocks were manufactured to meet the minimum requirements of NZS 4210 [9], which stipulates a minimum compressive strength of 12.5 N/mm^2. The tendons were installed at the locations illustrated in Figure 1, depicted as solid circles in the masonry walls. The layout was determined by ensuring that all panels contained at least one tendon and that tendons were distributed relatively evenly around the structure. The top tendon anchorage consisted simply of a steel plate and nut bearing directly on the masonry wall top, and therefore the timber top plate was discontinuous at these locations. Consequently, the tendon layout was influenced by the location of the connections between the roof trusses and top plate.

NZS 4229 stipulates that vertical steel reinforcement is to be spaced at a maximum of 800 mm centres, and in the cells at all wall ends and surrounding openings. The bars that trim openings assist in providing control against shrinkage. Anchoring the vertical reinforcement surrounding openings in the foundation would have restricted wall rocking and reduced the benefits of this wall system. An alternative detail was chosen where the vertical bars were terminated short of the floor slab and therefore only contributed to the control of shrinkage, illustrated in detail 2 in Figure 3. This was also expected to enhance construction as only the prestressing tendon anchorages were required to be installed in the foundation. The elevation of the east wall is depicted in Figure 5, where the prestressing tendons are shown as vertical lines projecting into the foundation beam, while the vertical reinforcing bars for shrinkage control terminate at the base of the masonry wall. The trimming of openings and the bond beam steel reinforcement design were consistent with current reinforced masonry construction practices. The thicker vertical line shown in Figure 5 and denoted 'CJ' depicts a shrinkage control joint, with the design of the steel reinforcement spanning the joint illustrated in detail 5 (Figure 3).

Wall – foundation connection

Detail 1 (Figure 3) depicts a cross-section of the masonry wall prestressed to a standard exterior foundation beam. The lower prestress anchorage consisted of a short length of prestressing tendon hooked around the foundation reinforcement and terminating with a coupler located at floor level. This enabled construction of the masonry wall followed by installation of the tendons by simply screwing the threaded bars into the previously installed couplers. The horizontal bond beam reinforcement consisted of a D16 bar in each of the top two block courses on alternate sides of the centrally located tendon. The tendon was enclosed within a PVC duct in the fully grouted wall.

Figure 3 shows details 3 and 4 for situations when the prestressed wall was located on a foundation beam providing pipe bridging or on a foundation retaining wall respectively. In terms of the wall and prestressing details, the only variation is the integration of the tendon anchorage and foundation steel.

Loading demand

Wind and seismic loading were determined using the New Zealand loadings code, NZS 4203:1992 [10], resulting in the consideration of loads at both the serviceability and ultimate limit states. The structure was located in a low seismic zone and on a category B site indicating intermediate soil type. The structural period was assumed to be less than 0.45 seconds, representing the period range of greatest seismic demand for this soil site. A structural ductility of 2 was assumed in design. The classic definition of ductility no longer applies to unbonded post-tensioned walls due to their rocking behaviour and large displacement capacity prior to tendon yielding. Based on the large displacement capacity observed during previous large-scale shake table testing of PCM walls [4], it was determined that a ductility of 2 was a conservative estimate of the readily achievable ductility level for this wall type.

Table 1 shows a summary of the calculated demand on the structure at the two limit states and for the two directions of loading, where along and across refer to loading in the north-south and east-west directions respectively. It is observed that loading due to earthquake governed in all but one case.

Prestress losses

Previous research at the University of Auckland demonstrated that considerable prestress losses can be expected for post-tensioned concrete masonry walls [11, 12]. Although creep and shrinkage testing was conducted on regular masonry units, the materials used in the mortarless blocks and grout were similar to those used in the creep and shrinkage study, and therefore the research findings could be applied to this design.

The tendons were 12 mm diameter threaded steel reinforcing bars, having a yield stress f_{py} of 500 N/mm^2 and an ultimate rupture stress f_{pu} of 575 N/mm^2. These bars had stress-strain characteristics resembling those of mild reinforcing steel, having a distinct yield plateau. This differed from the stress-strain curve for high tensile steel typically used in prestressed structures. As will be demonstrated, the 12 mm, 500 N/mm^2 bar provided sufficient strength for a structure of this size. This tendon type was therefore chosen for economic reasons, and it was ensured that tendon yielding would not occur under design level loading.

NZS 4230 stipulates the maximum stress allowed in the prestressing steel immediately after prestress transfer to be the lesser of $0.82f_{py}$ or $0.70f_{pu}$. Due to the stress-strain characteristics of mild reinforcing steel, it was important to ensure that the tendons remained in the elastic range during loading at all limit states. Given that relatively small wall displacements and tendon elongations were expected at the ultimate limit, and that prestress losses were expected to be significant, the maximum allowable initial tendon stress was specified, providing an initial stress f_{pi} of 403 N/mm^2 ($0.70f_{pu}$). Prestress losses due to steel relaxation were assumed to be small compared with those due to creep and shrinkage of the fully grouted masonry and were therefore ignored.

Based on the recommendations published previously, a creep coefficient C_c of 3.0 and maximum shrinkage strain ε_{sh} of 1000 $\mu\varepsilon$ were assumed in calculating losses [11, 12]. The loss in prestress Δf_{pl} has components due to creep Δf_{cr} and shrinkage Δf_{sh} and was calculated using Equation 1, where E_m was taken as 15 GPa as stipulated in NZS 4230, and the masonry stress f_{mi} was calculated for the shortest wall panel representing the worst case, giving a value of 0.30 N/mm^2. The modulus of elasticity of the prestressing steel was assumed to equal 200,000 N/mm^2. The resulting effective tendon stress after losses f_{se} was found to equal 191 N/mm^2 or 21.6 kN ($0.38f_{py}$). The significance of prestress losses is clearly evident, with losses resulting in a reduction in tendon stress of 53%. The high shrinkage strain that can be expected due to fully grouting the masonry, and the relatively low tendon yield stress due to the steel type chosen, contributed to this level of anticipated prestress loss.

$$\Delta f_{pl} = \Delta f_{cr} + \Delta f_{sh} = C_c \frac{f_{mi}}{E_m} E_{ps} + \varepsilon_{sh} E_{ps} \qquad (1)$$

Bracing capacity

The bracing capacity of the house was determined following the notion of the bracing approach outlined in NZS 4229. Each of the exterior masonry walls formed a bracing line, as shown in Figure 1, where lines in the north-south and east-west directions are denoted with numbers and letters respectively. The capacity of a bracing line was found assuming individual bracing panels where the height of each cantilever was measured from the base of the wall to the bottom of the bond beam. The elevation of line 2 is illustrated in Figure 6 and shows the bracing panels depicted hatched.

The bracing capacity of the structure was checked at both the serviceability and ultimate limit states to ensure that it exceeded the demand listed in Table 1. Limit states for PCM walls have been discussed in other publications, for example see [13], and have been reproduced in Figure 7 for completeness. It was decided to design the structure allowing no wall uplift at the serviceability limit, therefore calculating wall strength at first cracking. At ultimate limit state loading, the walls were assumed to have reached the nominal strength limit, with capacity calculated assuming a rectangular stress block as shown in Figure 7(b), where α and f'_m were taken from NZS 4230 and had values of 0.85 and 12 N/mm^2 respectively.

NZS 4230 provides an expression for calculating tendon stress f_{ps} at nominal wall strength. At the time of house design, the accuracy of this code expression when applied to PCM walls was unknown. Subsequent research has resulted in the development of a more appropriate expression based on finite element modelling [13]. Consequently, a conservative approach was adopted in this design where it was assumed that this increase equals zero, and therefore the tendon stress at the nominal limit state was assumed to equal the effective stress after losses, f_{se}. The resulting bracing capacities of the four exterior walls are listed in Table 2 and are seen to exceed the base shear demand. The two values shown for each wall line at the ultimate limit state are a consequence of the non-symmetric distribution of tendons, resulting in differing wall strengths in the two directions of loading. Strength reduction factors ϕ of 1.0 and 0.85 have been applied to the capacity values calculated at the serviceability and ultimate limit states respectively. Additional guidance on calculating PCM wall strength can be found in [1, 13].

Other design checks and considerations

A check of the shear capacity of the longest wall in each direction of loading demonstrated that additional horizontal shear reinforcement was not required. The reinforced concrete foundation beam was checked for shear at the ultimate limit state, which assumes wall rocking and therefore a lever arm between the tendon anchorage and the centre of the compression stress block. The standard exterior foundation beam design was shown to provide sufficient shear strength for the given loads.

Sliding between the walls and foundation was checked, recognising that the normal force comprises of a component due to wall and roof self-weight and the prestressing force in the tendons. A conservative value of 0.7 was taken for the coefficient of static friction μ_s following the recommendations of Paulay and Priestley [14] and assuming that the masonry was placed on a smooth concrete floor. The sliding resistance was shown to be considerably greater than the calculated base shear.

Corrosion protection of the tendons was not specified as it was decided that the tendons would be sufficiently separated from the environment, enclosed within plastic ducts inside fully grouted masonry. The bottom anchorage was fully enclosed in the foundation beam and the top anchorage was located within the roof space.

CONSTRUCTION

Although complicated by the required pipe bridging and foundation retaining, construction of the raft floor was conducted following standard New Zealand practices. A photo showing the integration of a bottom tendon anchorage in the standard exterior beam is shown in Figure 8(a). Figure 8(b) shows the first course of blocks for the PCM walls being laid using traditional methods, with a tendon coupler shown protruding from the floor. Above this masonry course, wall erection speed was greatly enhanced through the use of mortarless blocks. The tendons were installed after the walls had reached their full height and following the installation of the mild reinforcing steel around the openings and in the bond beam. Washouts were provided in the first course of masonry at each tendon location to assist when screwing the tendons into the couplers, as illustrated in Figure 8(c). PVC ducting was then installed to ensure the tendon remained unbonded. A temporary top tendon anchorage was installed, enabling a small prestressing force to be applied to the tendon using a spanner. This small force was estimated to be only a few kilonewtons, and helped provide additional stability to the mortarless block walls prior to grouting.

The walls were prestressed one week after grouting using simple stressing equipment, consisting of a small hollow core jack and jacking chair, and a hand pump, as shown in Figures 8(d) and (e) respectively. The tendons were stressed to the desired level (403 N/mm^2 or $0.70f_{pu}$), which was determined using the calibrated pressure gauge seen in Figure 8(e). A load cell and digital meter were also used for verification, though it is noted that the use of a load cell is redundant provided the pressure gauge is reliable. Upon reaching the desired level of prestress, the top anchorage nut was hand tightened through access in the jacking chair, and the stressing equipment removed.

It is difficult to make a direct cost comparison with traditional reinforced concrete masonry construction in New Zealand due to the nature of this project. Most of the product and labour provided for the construction of the floor and walls was donated. This project has demonstrated that a post-tensioned wall system can be simply integrated into existing floor and foundation design with minimal effort. The use of mortarless blocks significantly enhanced the speed of wall construction, and therefore shows considerable promise for the future. The reduction in wall steel quantities was minimal due to the need to provide shrinkage control around all openings. If future research can demonstrate the ability of fibre reinforced grout to provide shrinkage control, then steel quantities could be significantly reduced with a corresponding time and labour saving. Post-tensioning of the walls is a relatively simple procedure using readily available equipment and requiring moderately skilled labour.

CONCLUSIONS

A consortium of designers and product manufacturers has led to the design and construction of the first post-tensioned concrete masonry house in New Zealand. An account of the design and detailing of the house has been presented. The house was constructed using generic materials and represents one of the first applications of mortarless blocks in New Zealand. This structure was designed to satisfy New Zealand seismic loading requirements, and required a specific design as the use of mortarless blocks is currently outside the scope of the masonry standard. Integration of the prestressing tendons resulted in minimal changes to the design of the floor and roof, demonstrating that the use of PCM does not require a large departure from traditional masonry practices. Speed of construction was considerably enhanced with the use of mortarless masonry blocks.

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Table 1 – Design loads

Limit state	Direction	Demand	Loading type
Serviceability	Across	22	Wind
	Along	11	Earthquake
Ultimate	Across	70	Earthquake
	Along	70	Earthquake
		kN	

Table 2 – Design loads

Limit state	Direction	Bracing line	Base shear Capacity		Base shear Demand
Serviceability	Across	A	19.0		
		B	9.9		
			28.9	>	22
	Along	1	34.6		
		2	24.7		
			59.3	>	11
Ultimate	Across	A	46.9 / 48.7		
		B	25.0 / 25.0		
			71.9 / 73.7	>	70
	Along	1	92.8 / 81.7		
		2	63.1 / 61.3		
			155.9 / 143.0	>	70
			kN		kN

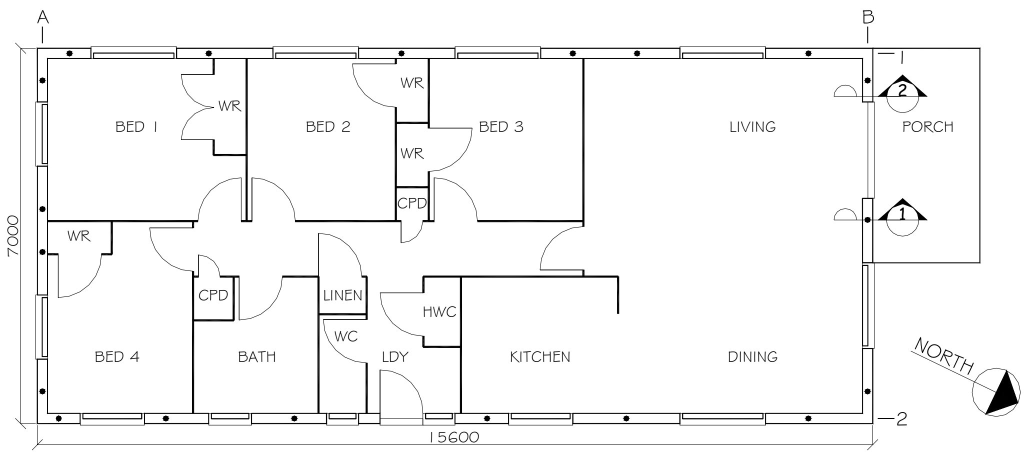


Figure 1 – Plan layout of house

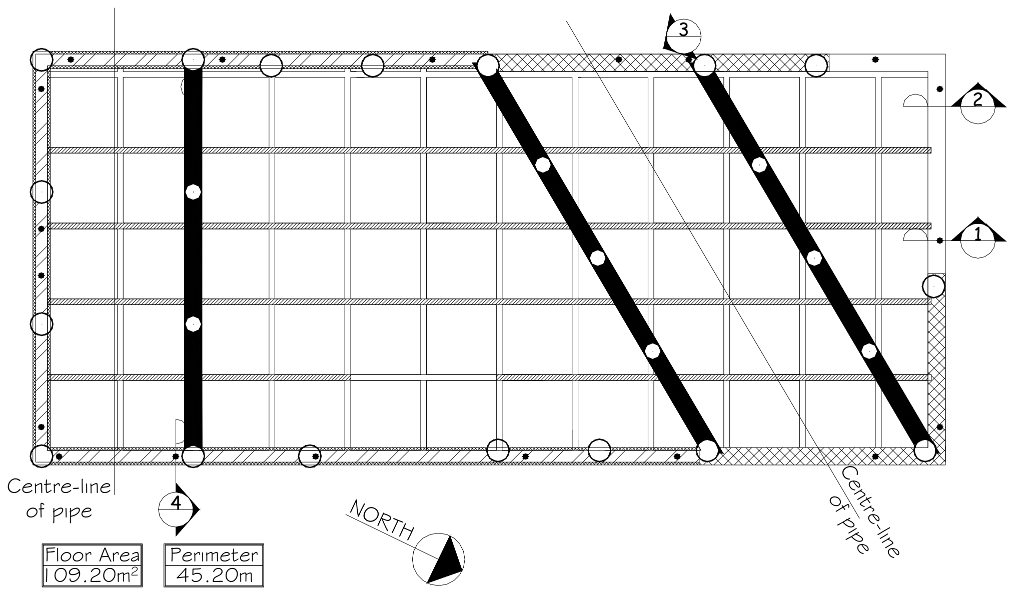


Figure 2 – Raft floor design

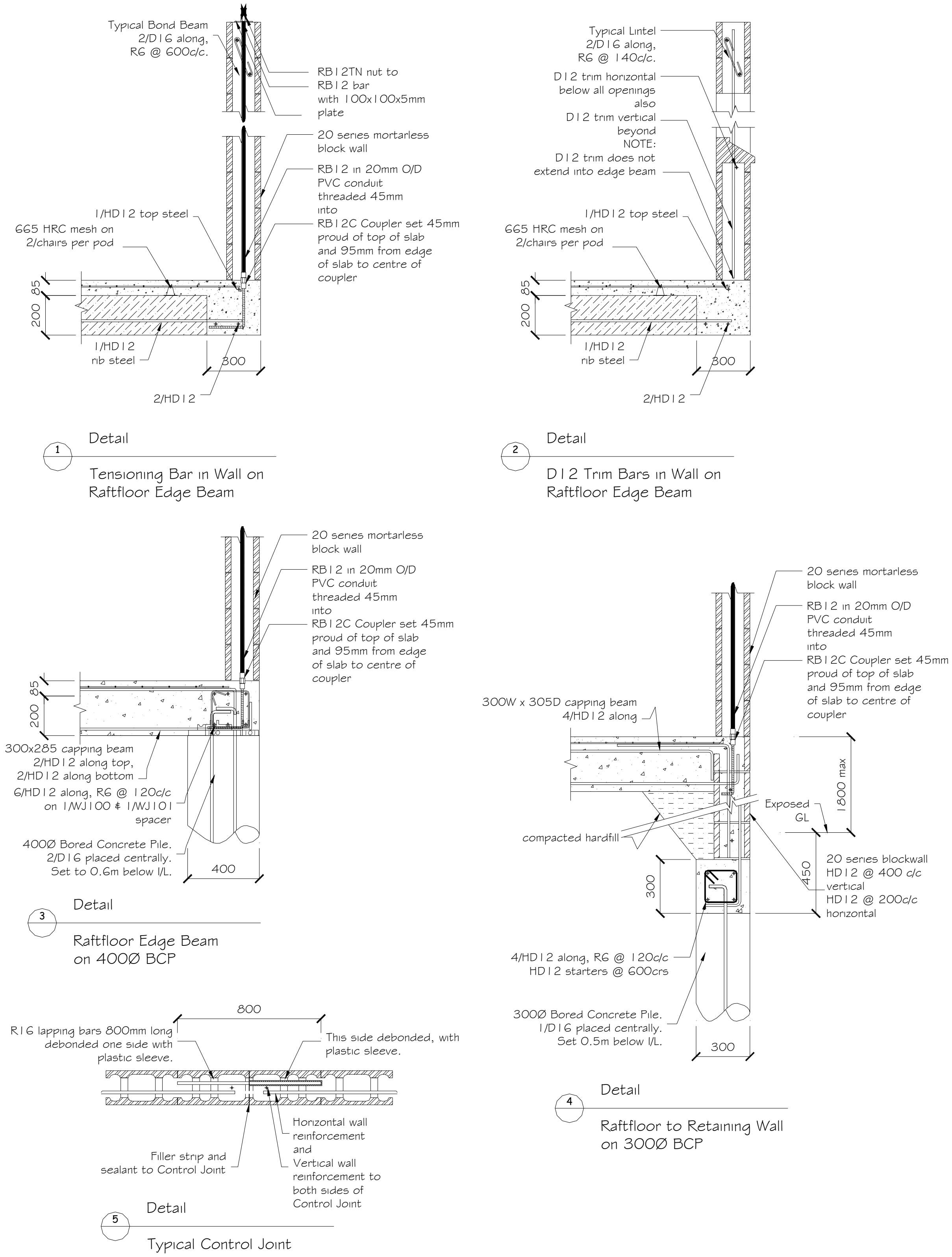
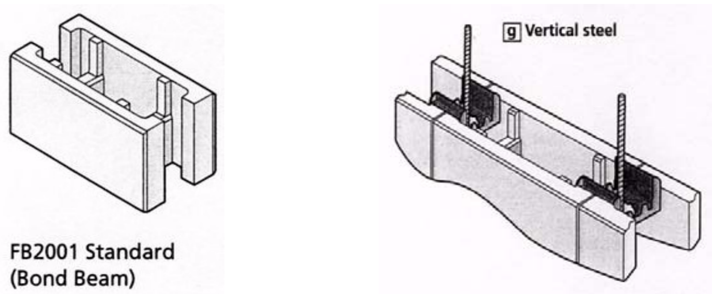


Figure 3 – Design details



(a) Single masonry block (b) Horizontal connection and steel location

Figure 4 – Formblock [8]

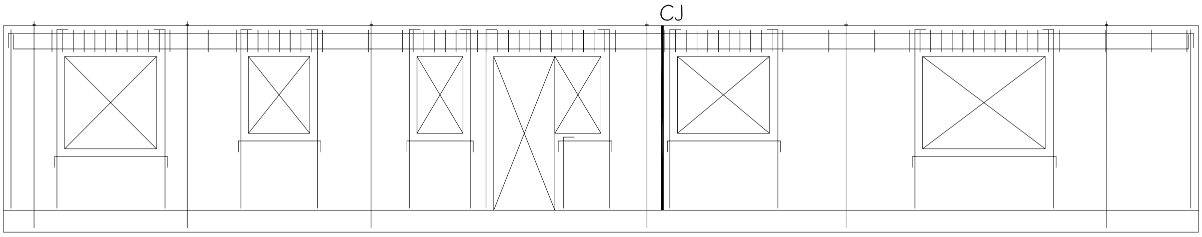


Figure 5 – Elevation of east wall showing steel layout

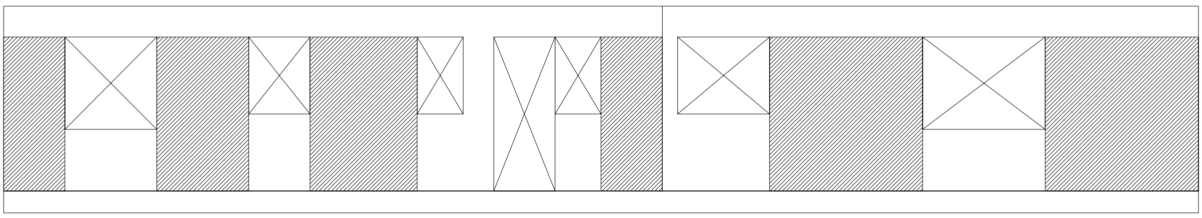


Figure 6 – Bracing panels in wall line 2 (east wall)

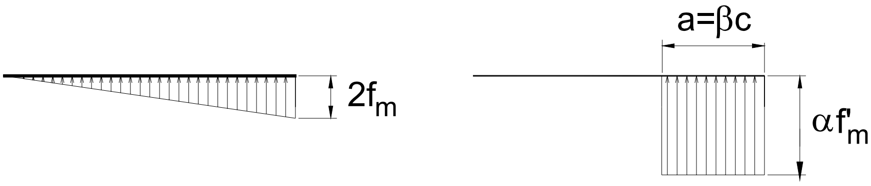


Figure 7 – Design limit states; (a) Serviceability, (b) Ultimate


(a) Tendon anchorage in exterior beam

(b) Laying the first course of blocks


(c) Tendon coupler at floor height prior to grouting

(d) Post-tensioning equipment setup

(e) Hand pump and pressure gauge

Figure 8 – Construction photos