NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING
2019 Pacific Conference on Earthquake Engineering
TURNING HAZARD AWARENESS INTO RISK MITIGATION
4 – 6 April | SkyCity, Auckland | New Zealand

Capacity of diaphragm strengthened with FRP: comparison between ACI 440.2R and in-situ tests

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ABSTRACT

Seismic codes are frequently revised and updated, especially after large earthquakes. These updates entail that existing structures must be assessed to meet the new requirements and, very often, retrofitted. In the specific case of concrete slabs, the assessment often concludes that strengthening is required to increase the tension capacity of the 'tie' members of the diaphragm. Recently, Fibre-Reinforced Polymer materials (FRP) have been extensively used to strengthen existing buildings due to its light weight, fast installation and less invasive approach in comparison with more conventional materials. Design codes have been developed specifically for strengthening buildings using FRP, with ACI440.2R probably being the most widely used standard for the design of FRP strengthening of concrete buildings. However, equations for strengthening concrete diaphragms are not provided in ACI440.2R and, therefore, the equations given for flexural or shear strengthening of other structural members have to be adopted for concrete diaphragms. An example of a FRP strengthening of a hollowcore diaphragm of an existing supermarket building located in Wellington is presented in this study. The Equivalent Static Method (ESM) was used to obtain the seismic demand acting on the diaphragm and the analysis of the diaphragm was carried out using a grillage model. Due to the lack of guidance in ACI440.2R for strengthening of floor diaphragms, the equations to determine the effective stress of FRP for flexural and shear strengthening were used, and the calculated results were compared with the results from in-situ tests of FRP specimens

installed on the actual diaphragm. The results show that the equations provided in ACI440.2R significantly underestimate the effective stress of FRP (up to 50%) applied to concrete diaphragms.

1 INTRODUCTION

Seismic codes are usually revised and updated after large earthquakes. Depending on the year of construction of existing buildings, these structures may not fulfil the requirements of the current design codes (Priestley 1996). This is one of the reason for existing structures to often require strengthening after being assessed by current seismic codes (INN 2003; NZS 1170.5 2004). In the specific case of reinforced concrete (RC) structures, concepts as capacity design and ductility were mostly unknown by engineers before 1975 when these concepts were adopted in the design (Priestley 1996). Columns, beams, walls and slabs of existing pre-1975 RC buildings are required to be strengthened in most of the cases (MBIE 2016a).

Recently, Fibre-Reinforced Polymer (FRP) has been extensively used to strengthen existing buildings due to its high strength-to-weight ratio, fast installation, high corrosion resistance and less invasive approach in comparison with more conventional materials (Teng et al. 2003). First studies on FRP focused on beam strengthening as an alternative to steel plate bonding (Teng et al. 2003) and numerous investigations have been carried out (Ritchie et al. 1991; Chajes et al. 1994; Ross et al. 1999; Daouadji et al. 2016). In this case, FRP sheets or plates are bonded to the underside of the beam to resist the tensile forces due to bending.

FRP has been also utilised for shear strengthening of RC beams (Triantafillou 1998; Zhang and Hsu 2005; Ozden et al. 2014), and different methods have been used to bond FRP sheets or plates to RC beams. These include bonding FRP on the sides of RC beams only, bonding U-jackets to cover both sides and the underside of the beam, and wrapping FRP around the cross section.

Strengthening of RC columns with external wrapping of FRP has been extensively reported (Demers and Neale 1999; Benzaid and Mesbah 2013; Krishna, Jacob, and Saravana Raja Mohan 2018). In this case, FRP sheets are orientated in the hoop direction enhancing the shear and ductility capacity of RC columns (Seible et al. 1997) which is fundamental for seismic retrofit. Anchorage of RC columns using FRP anchors has been also investigated (del Rey Castillo, Griffith, and Ingham 2019; del Rey Castillo et al. 2019).

Flexural strengthening of RC slabs using FRP sheets or plates has been worldwide studied and implemented (Mosallam and Mosalam 2003; Teng et al. 2000; Seim et al. 2001). The strengthening in this case is very similar to flexural strengthening of RC beams i.e. the FRP is bonded to the underside of the slab.

Design codes have been developed for strengthening buildings using FRP covering all the aspects mentioned above (ACI 2017; CEN 2005; CNR 2013). However, to the best of the authors' knowledge, a procedure for strengthening RC slabs for diaphragm action has never been reported in the literature. Seismic loading was usually not considered for designing RC slabs in existing structures (MBIE 2016a), which implies that the in-plane capacity of existing RC slabs is likely to be inadequate to develop diaphragm action in the slab.

This paper presents a retrofit of a hollowcore concrete slab using FRP strips of an existing supermarket building located in Wellington. Equations given in ACI440.2R (ACI 2017) to determine the effective stress of FRP for pure tension and shear strengthening are compared with in-situ tests of FRP specimens installed on the actual slab, in addition to the methodology described in CNR-DT 200 for shear tear capacity of FRP sheets. The main objective of the work reported here was to determine the accuracy between the results obtain with equations given in ACI440.2R (ACI 2017) and the experimental results obtained on the floor diaphragm. An innovative design at the perimeter of the diaphragm is proposed to achieve higher tensile capacity from the FRP strips. A good understanding of how to do FRP strengthening of RC diaphragm floors will allow to better design these structural elements and, therefore, will ensure that seismic loads are transferred and distributed to the lateral resisting elements.

2 SUPERMARKET BUILDING, WELLINGTON

2.1 The building

The building was designed in 1981 and is located in Central Wellington. The building consists of three different areas, the supermarket, the carpark and a single storey street front mall. The supermarket is in the ground floor toward the rear portion of the site, while the carpark is located at the first floor over the supermarket. The mall is the single-storey part of the building located at the front).

The supermarket was built with a concrete slab on-grade at the ground floor level, precast RC walls, precast RC columns, U-shell beams, RCM walls and a hollow core concrete slab at the first-floor level. The carpark consists of steel portal frames supporting light weight roofing. The mall consists of a concrete slab on-grade, precast RC columns, steel beams and timber partitions. Figure 1 shows a plan view of the first floor of the building.

EQ Struc ltd was engaged to undertake a seismic strengthening design of the building to achieve 67%NBS as defined in Part A of the MBIE guidelines (MBIE 2016b). The %NBS compares the building's Ultimate Limit State (ULS) earthquake resistance with the current New Zealand Building Code requirements for a new building constructed on the site. The seismic rating of the building is then expressed as a percentage of New Building Standard (%NBS).

2.2 Analysis of the building

The design earthquake demand on the structural elements of the building were derived using the Equivalent Static Method (ESM) defined in NZS 1170.5 (NZS 1170.5 2004). ESM was deemed to be appropriate as the building has a fundamental period lower than 0.4 s. The assessment was completed using conventional calculations and 3D computer aided analysis. Earthquake induced forces acting in the orthogonal directions parallel to the main axes of the structural system and the combination of these two orthogonal directions, as per Clause 5.3.1.2 of NZS 1170.5 (NZS 1170.5 2004) were considered. Table 1 shows the parameters considered to obtain the seismic demand.

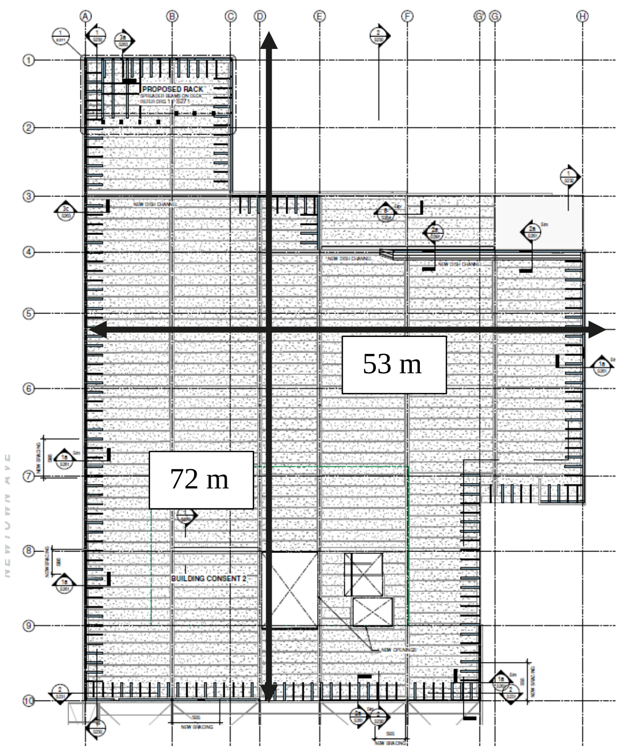

Figure 1: First Floor Plan View

2.3 Analysis of the diaphragm

The concrete slab (diaphragm) has a total area of 3000 m^2 and it was analysed using the "Diaphragm Grillage Modelling Methodology" provided in Appendix C5E of the MBIE guidelines by EQ Assess (MBIE 2016a) for the seismic assessment of existing buildings –The Seismic Assessment of Existing Buildings Section 5. This methodology is a more sophisticated analysis than a simple strut and tie analysis and is recommended for diaphragms with complicated geometries.

The thickness, width and spacing of the grillage members were determined using the procedure provided in Appendix C5E of the MBIE guidelines (MBIE 2016a).

Table 1: General seismic factors.

Seismic Factor Coefficient	**NZS 1170.5 Clause**	**Selected Value**
Ductility Factor (μ)	4.3	1.0
Structural Performance Factor (S_p)	4.4	1.0
Soil Type	3.1.3	C
Hazard Factor (Z)	3.3	0.4
Earthquake combination factor	4.2	0.3
Near-Fault Factor N(T,D)	3.16	1.0
Return Period Factor (R_u)	3.15	1.0

As mentioned, the first floor of the building consists of a 200 mm precast hollowcore slab. The slab has a 50mm concrete topping reinforced with 665 mesh and is supported by U-shell beams and precast columns. Starters bars connect the topping to the perimeter precast walls. The detailing of the wall-to-floor connection can be seen in Figure 2, while the dimensions of the hollow core unit are reported in Figure 3.

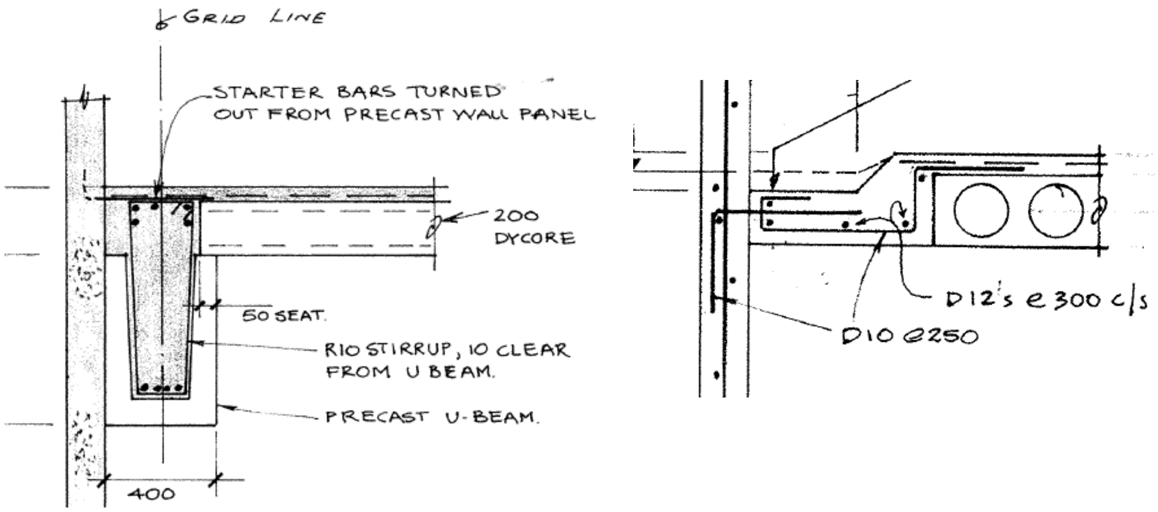

Figure 2: Hollowcore slab details. Connection to walls perpendicular (left) and parallel (right) to span direction of precast units

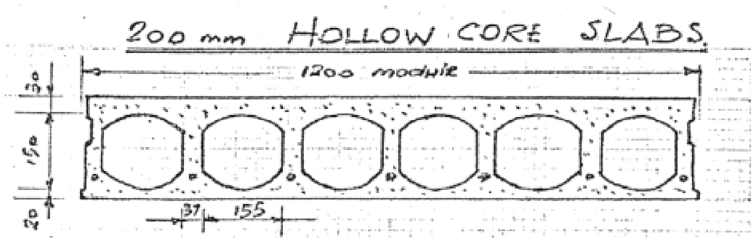

Figure 3: Dimensions of 200mm Hollowcore slab

The spacing of the orthogonal members of the grillage was selected as 1.0m, which determines the width of the grillage members. Orthogonal members have a width equal to 0.75·spacing = 750mm and diagonal members have a width equal to 0.53·spacing = 530mm. The thickness of the diagonal members and the orthogonal members perpendicular to precast units is the average thickness of the combined area of the concrete topping plus the top flange of the units (80mm). The thickness of the orthogonal members parallel to the precast units is the average thickness of the combined area of the concrete topping and the precast units.

Diagonal members resist only compressive forces, therefore, a limit for tensile forces equal to zero was applied to these members in the computer model. The diaphragm was analysed separately from the main structure. Inertial forces and the reactions from the vertical elements (walls and columns) were manually applied to the diaphragm model.

2.4 Diaphragm results and strengthening

The compressive capacity of the diaphragm was adequate to resist the seismic forces, but the tensile capacity of the diaphragm only achieved 5%NBS. Figure 5 shows screenshots of the tensile forces for the grillage members. The tensile capacity of the diaphragm was provided entirely by the reinforcement mesh, which was calculated as 77 kN/m. This value was exceeded in almost all the members of the grillage, which infers that a global strengthening was required. The areas with the highest tensile forces, shown in yellow in Figure 4, correspond to the areas of the diaphragm connected to a concrete ramp used to access the carpark. Externally bonded CFRP sheets are used to strengthen the concrete diaphragm. CFRP was selected instead of other FRP materials because its high tensile strength, without adding any significant seismic weight to the structure (Teng et al. 2003).

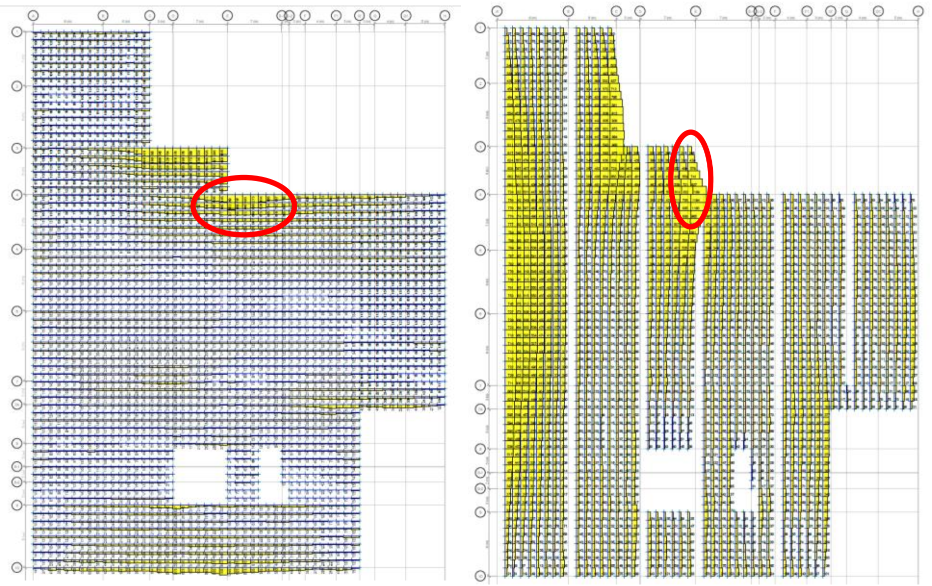

Figure 4: Tensile forces in the orthogonal members of the grillage. Members parallel (left) and perpendicular (right) to the precast units

3 CALCULATIONS

The debond capacity of the FRP sheet in a shear-tear configuration was calculated following two of the most commonly used standards, the American ACI440 and the European CNR-DT200

3.1 ACI 440.2R-17

ACI440.2R (ACI 2017) is a guide by the American Concrete Institute for the design of externally bonded FRP for strengthening concrete structures. As mentioned before, this document does not provide a procedure or equations for the design of diaphragm strengthening. It is then up to the engineer's judgement to decide what equations and procedure to utilise for the design. Since the strengthening is required to increase the tensile capacity of the diaphragm, the procedure given for elements in pure axial tension, as per Clause 12.4, is used in the design.

The tensile capacity provided by the FRP sheets is typically limited by the FRP-to-concrete bond strength. ACI440.2R provides an equation to determine the effective strain of the FRP when premature FRP-to-concrete debonding occurs. Equations 1-5 (ACI 2017) give the parameters to determine the effective strain.

$$\varepsilon_{fe} = k_v \cdot \varepsilon_{fu} \leq 0.004 \tag{1}$$

$$k_v = \frac{k_1 \cdot k_2 \cdot L_e}{11{,}900\varepsilon_{fu}} \leq 0.75 \tag{2}$$

$$L_e = \frac{23{,}300}{(n \cdot t_f \cdot E_f)^{0.58}} \tag{3}$$

$$k_1 = \left(\frac{f_c'}{27}\right)^{\frac{2}{3}} \tag{4}$$

$$k_2 = 1 \tag{5}$$

where ε_{fe} is the effective strain of the FRP; k_v is the bond reduction coefficient; ε_{fu} is the ultimate strain of the FRP; L_e is the active bond length; n is the number of layers of FRP; t_f is the thickness of the FRP; E_f is the modulus of elasticity of the FRP and f'_c is the concrete compressive strength. The tensile stress in the FRP is given in Equation 6 below.

$$f_{fe} = E_f \cdot \varepsilon_{fe} \tag{6}$$

Table 2 shows the values of the effective strain, tensile stress and tensile capacity provided by 1 and 2 layers of 200 mm wide strips of CFRP. The concrete strength considered is 37.5 MPa which corresponds to the probable compressive strength of a concrete with a nominal compressive strength equal to 25 MPa.

Table 2: Tensile capacity of FRP according to ACI440.2R.

Number of layers	**1 layer**	**2 layers**
Thickness of one layer (t_f)	1.0mm	1.0mm
Modulus of elasticity (E_f)	65087 MPa	65087 MPa
Ultimate strain (ε_{fu})	0.98%	0.98%
Width (b)	200mm	200mm
Effective strain (ε_{fe})	0.394%	0.263%
Tensile stress (f_{fe})	256 MPa	171 MPa
Tensile capacity	51 kN	69 kN

3.2 CNR-DT 200

CNR-DT 200 (CNR 2013) is an Italian-developed standard widely used in Europe and provides an approach based on fracture energy to calculate the debond strength of the CFRP strips in a shear-tear configuration. Equations 7 and 8 define the parameters to determine this value.

$$F_{max} = b_f \sqrt{2 \cdot E_f \cdot t_f \cdot \Gamma_F} \quad (7)$$

$$\Gamma_F = k_b k_G \sqrt{f_{cm} \cdot f_{ctm}} \quad (8)$$

where F_{max} is the tensile strength of the CFRP strips; b_f is the width of the strips; E_f is the modulus of elasticity of the CFRP; t_f is the thickness of the CFRP; Γ_F is the specific fracture energy; $k_b = 1.18$, is the geometrical corrective factor; k_G= 0.077 mm, is experimental corrective factor; f_{cm} is the concrete compressive strength and f_{ctm} is the concrete tensile strength. The tensile strength obtained using Equation 7 for a 200 mm wide CFRP strip is 70.8 kN.

4 IN-SITU TESTS

In-situ shear-tear tests were carried out in the actual concrete slab to verify the results obtained from ACI440.2R. FRP strips with a width of 200 mm and a thickness of 1.0mm were bonded to the slab, after the surface was prepared following the instructions from the supplier. Three different bond lengths were tested to investigate their effects on the results. Figures 5 and 6 show the experimental setup and the tests, respectively. Table 3 shows a summary of the experimental results. It is noticeable from Table 3 that there is an improvement in the tensile capacity of the strips when the bond length is increased from 500 mm to 800 mm. However, the tensile capacity did not increase when the bond length is 1200 mm with respect to the results obtained for a bond length of 800 mm.

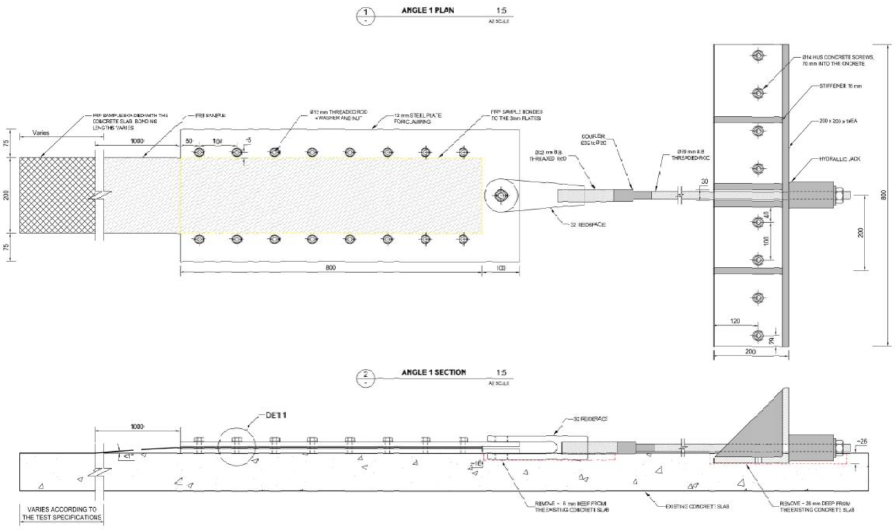

Figure 5: Experimental Setup. Plan view (top) and elevation (bottom)

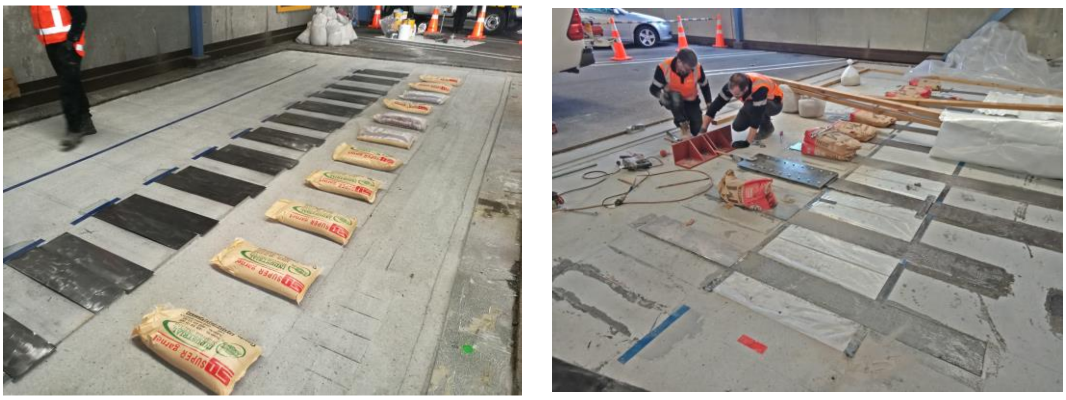

Figure 6: CFRP sheets and epoxy used in the tests (left). Pull-out test (right)

Table 3: Test results – Tensile capacity

Bond length	500 mm	800 mm	1200 mm
Batch 1 – first test	67.7 kN	75.0 kN	76.0 kN
Batch 1 – second test	67.7 kN	79.0 kN	-
Batch 2 – first test	68.2 kN	84.9 kN	85.4 kN
Batch 2 – second test	71.1 kN	81.4 kN	-

ACI440.2R provides an equation to calculate the development length required for the FRP to achieve the effective stress. Refer to Equation 9 below.

$$l_{df} = \sqrt{\frac{n \cdot t_f \cdot E_f}{\sqrt{f'_c}}} \tag{9}$$

For a 200 mm wide and 1 mm thick strip (same as the tests), the development length according to Equation 9 is equal to 103 mm. This result shows the benefits of providing a larger bond length. While the development length defined in Equation 9 is to ensure that the FRP strips achieve the effective stress, providing a larger bond length allows the FRP strips to achieve higher stresses and, therefore, higher tensile capacity. For this reason, a minimum development length equal to 1000 mm was adopted in the design.

Regarding the effective strain of the FRP, the test results show that the FRP has an average effective strain equal to 0.615% when the bond length is equal or greater than 800 mm. This is about 1.5 times the value obtained using the equations from ACI440.2R for elements in pure axial tension. For design purposes, a design strain equal to 0.615% was adopted.

When the tensile strength value calculated using Equation 7 from CNR-DT 200 is compared to the value obtained from the in-situ tests, it can be noticed that this value is very close to those obtained in the in-situ tests for a bond length equal to 500 mm (97% accuracy).

CNR-DT 200 (CNR 2013) also defines an optimal bond length, which is the length, if exceeded, has no increase in the force transferred between concrete and FRP. Equation 10 defines the optimal bond length.

$$l_{ed} = \frac{s_u}{2 \cdot \gamma_{Rd} \cdot \Gamma_F} \sqrt{\frac{\pi^2 \cdot E_f \cdot t_f \cdot \Gamma_F}{2}} \geq 200mm \tag{10}$$

where s_u= 0.25 mm and γ_{Rd} = 1.25 is the corrective factor. The optimal bond length obtained from Equation 10 is equal to 58 mm which much lower than the value obtained from the in-situ tests (800 mm).

Figures 7 and 8 show the FRP strengthening proposed for the diaphragm. 635 mm wide strips at 2.0 m centres was determined to be the most cost-effective configuration for the FRP strengthening. The width of 635 mm is the width of the rolls of the FRP sheets and, therefore, any other width for the FRP strips will involve extra work. Most of the strips are only 1 layer of FRP. However, strips with 2 layers of FRP were required in areas with high tensile demand. CFRP bands of different widths were required along the perimeter of the slab and around the openings. Up to 7.0 m wide CFRP bands (2 layers) were required at the areas close to the ramp. A comparison with the preliminary FRP strengthening design carried out using the equations from ACI440.2R showed that the amount of FRP decreased by 50%

As mentioned above, the bond length provided is 1000 mm. To achieve that, an innovative design is proposed. Steel plates with lengths of 1.0 m and 2.0 m long, running parallel to the CFRP strips, are provided along the perimeter of the diaphragm. These steel plates have a double function. Firstly, they provide the required unloaded length for the CFRP strips to develop the tensile capacity achieved in the in-situ tests. Secondly, the steel plates resist the tensile forces at the perimeter which are beyond the capacity of the FRP strips. Mechanical anchorage was also investigated as an alternative to provide the required tensile capacity to the CFRP strips at the ends instead of using the steel plates. However, anchorage systems are not effective for diaphragms due to a strain compatibility issue. Debonding will occur first before the anchorage system is engaged which does not occur with the steel plates solution. Figure 9 shows a typical arrangement and cross section of the steel plates at the perimeter.

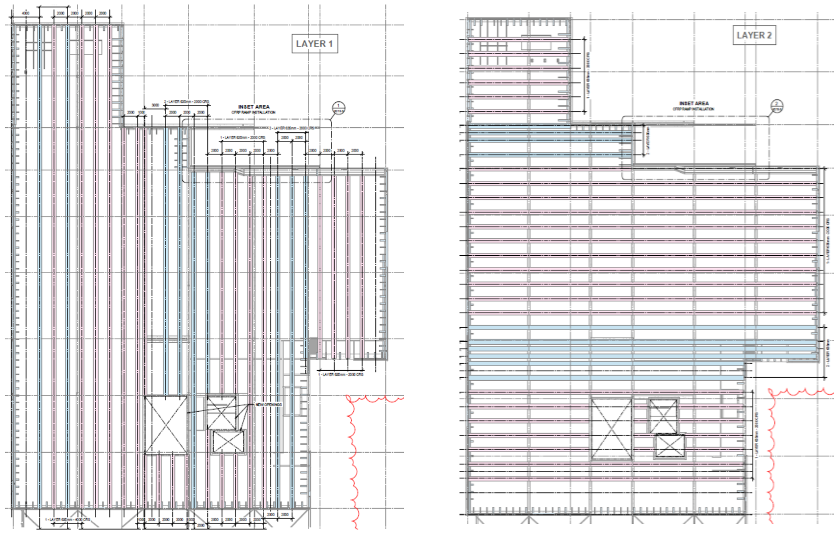

Figure 7: CFRP strips. 1 layer (red) and 2 layers (blue)

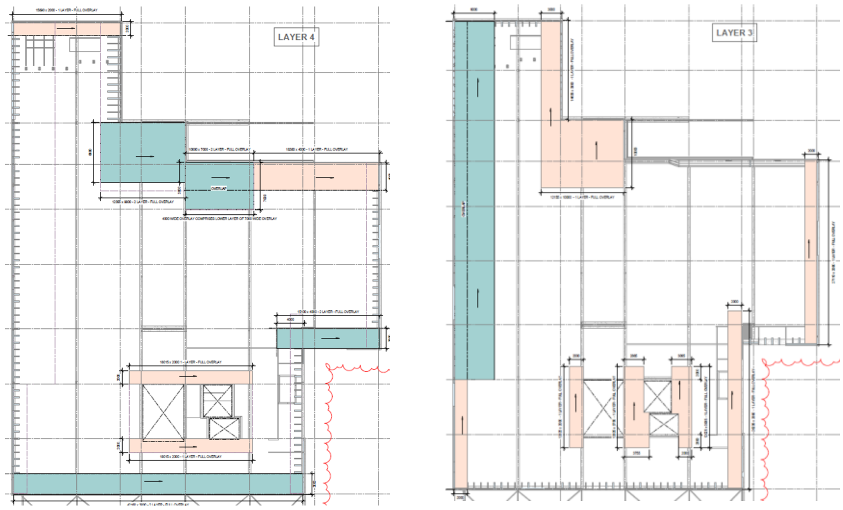

Figure 8: CFRP bands. 1 layer (orange) and 2 layers (green)

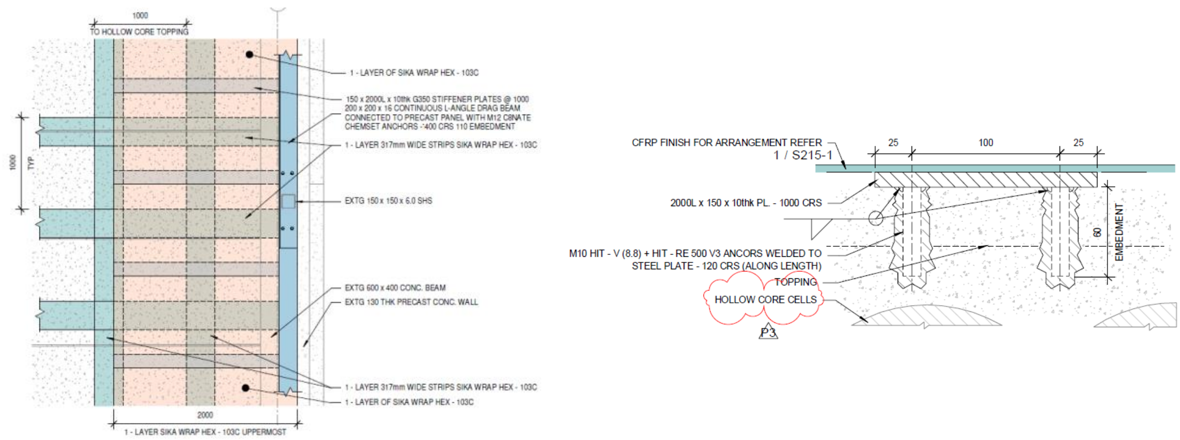

Figure 9: Steel plates at the perimeter. Plan view (left) and cross section (right)

5 CONCLUSIONS

A seismic strengthening design of a 3000 m^2 concrete slab has been undertaken. The aim of strengthening design was to provide a score of 67%NBS to the slab. CFRP strips were design to provide the required tensile capacity to the diaphragm. The procedure for members in pure tension given in ACI440.2R (ACI 2017) was utilised for the FRP design, in addition to the methodology described in CNR-DT 200 for shear tear capacity of FRP sheets. In-situ tests were carried out to verify the design. An innovative design for providing longer bond lengths to the CFRP strips is proposed. The main objective of this work was to verify the accuracy of the theoretical equations given in the document and prove the benefits of providing longer bond lengths to FRP strips.

The investigations revealed:

- The effective tensile strain obtained in the in-situ tests is about 1.5 times greater than value calculated using ACI440.2R.
- Increasing the bond length beyond the value of the development length calculated using ACI440.2R allows the CFRP strips to achieve higher tensile strength.
- The tensile strength of CFRP strips calculated using CNR-DT 200 is very close (97% accuracy) to the average value obtained from the in-situ tests for a bond length equal to 500 mm. However, the optimal bond length is underestimated by a significant margin (14 times less).

Therefore, it is recommended to carry out in-situ tests to verify the equations given for FRP design in ACI 440, although the methodology used in CNR gave accurate results. It is also required to develop a specific procedure for diaphragm strengthening, providing specific equations to obtain the tensile capacity of FRP.

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