DIAPHRAGM ACTION IN DRAMIX^{TM} STEEL FIBRE REINFORCED CONCRETE ON PROFILED SHEET METAL HYBRID FLOOR SYSTEMS

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Abstract

This paper presents a design procedure covering the diaphragm action of a hybrid floor slab comprising of Dramix steel fibre reinforced concrete (SFRC) topping on profiled metal sheeting. The SFRC alternative is shown to be an efficient and structurally adequate equivalent to conventional bar reinforced diaphragm.

It also demonstrates that Dramix SFRC topping on profiled metal sheeting satisfies the requirements of Section 8 “Stress Development, Detailing and Splicing of Reinforcement and Tendons”, Section 13 “Design of Diaphragms” and Appendix A “Strut-and-Tie Models” of NZS3101:2006 [1,2].

A design example is presented which shows the analysis and design calculations carried out for a typical 110mm thick 5.0m x 10.0m Hi-Bond slab supported along its four sides on Universal Beams which are supported in turn on four corner Universal Columns. The slab has a 2m x 2m opening located 1.0m from its east side and central to its long axis.

Two methods of analysis are presented to determine the stresses and forces induced in the slab due to the application of a seismic lateral in-plane force. These include a Finite Element Analysis (FEA) method and a Strut-and-Tie method. Results of the two methods provide enough evidence that the hybrid slab constructed with Dramix SFRC topping on Hi-Bond sheet metal is capable to transfer the lateral load to the supporting elements adequately through the diaphragm action.

A design methodology then follows to determine the strength of concrete, size of trimmer reinforcing bars, starter bars and the steel fibre dosage required for the slab to resist the calculated stresses and forces in accordance with the provisions of NZS3101:2006 [1,2].

1. BASIS OF DIAPHRAGM ACTION

Lateral loads from wind and earthquake actions on buildings are usually transmitted to the lateral force-resisting structure through the floors and roof acting as diaphragms. Lateral force-resisting structures can be in the form of shear walls, moment resisting frames, braced frames, etc. Floors and roofs incorporating hybrid floor systems constructed using profiled metal sheets with cast-in-situ concrete layer can act simultaneously as floors subjected to gravity loads and as horizontal diaphragms to transfer in-plane actions due to lateral loads.

The diaphragm can be analyzed by considering the floor or roof as a horizontal beam [3]. The lateral force-resisting structural system forms the supports for this beam to which the lateral loads are transmitted. As a beam tension and compression are induced in the chords of the diaphragm and the perimeter frame must be capable of carrying the induced forces.

When the ribs of a hybrid floor or roof span parallel to the supporting element (shear wall or moment resisting frame) the shear in the diaphragm beam must be transferred between the adjacent ribs and also to the supporting structure. The web shear must also be transferred to the chord elements. Thus the design of a diaphragm is essentially a connection design problem.

Where earthquake loading is a major consideration special attention needs to be given to the robustness of the system and details. This includes checking that vertical support for the floor is not lost due to the elongation of the supporting beams at plastic hinges resulting in the collapse of the floor.

In hybrid floor/roof systems with a composite topping the topping itself can act as a diaphragm if it is adequately reinforced. Reinforcing requirements can be determined by shear-friction. The cord forces in the perimeter frames should be derived on the basis of a strut-and-tie action in deep beams. The starter bars holding the floor to the perimeter beams are designed on the basis of shear-friction

2. BASICS OF STEEL FIBRE REINFORCED CONCRETE

Design properties of Dramix SFRC are dependant on its post-cracking toughness which is influenced by the concrete compressive strength and also by fibre properties of aspect ratio, ultimate tensile strength and fibre geometry which controls its anchorage to the concrete matrix. Different fibre properties will result in different fibre dosages to meet specific design requirements. Hence SFRC designs must be based on either reliable test data provided by the manufacturers and backed up with a CE label and Certificate of Conformity to steel fibre manufacturing standard EN14889-1, or confirmed by project specific testing.

The Dramix SFRC design methodology presented in this paper is based on Clause 5.5 - Properties of Steel Fibre Reinforced Concrete" and Appendix A to the Commentary of Section 5 of NZS3101:2006 [2] in addition to Dramix Guidelines - Design of Concrete Structures, Steel Wire Fibre Reinforced Concrete Structures with or without Ordinary Reinforcement" Nr. 4 – 1995 [4] and Bekaert Design Guidelines [5]. The beam test used in this document to determine the SFRC properties is similar to the test method described in NZS3101, C5A [2].

The design method calculates the ultimate axial tension capacity of Dramix SFRC section based on the assumptions stated in Clause C5.A4.1 of NZS3101:2006 Part 2 [2] and its ultimate shear capacity in accordance with the provisions of C5.A4.2 [2].

The mean equivalent flexural strength of Dramix SFRC and the design value of the increase in shear strength due to the presence of steel fibres are based on the ratios R_{em} and R_t, respectively as provided by Dramix Guidelines Nr. 4 – 1995 [4]. These ratios are expressed as functions of the fibre aspect ratio, dosage and diameter. A constant factor "C" is used in determining the values of these two ratios, which relates to the shape of the fibre and its protective coating. For ordinary (uncoated) Dramix steel fibres with duo form ends the constant factor "C" is considered as 20.

3. THE FLOOR DIAPHRAGM EXAMPLE

The floor slab example considered in this report is a typical 110mm thick 5.0m x 10.0m Hi-Bond slab supported externally along its four sides on Universal Beams which are in turn supported by four corner Universal Columns. The slab is also supported internally on secondary Universal Beams (shown as yellow) and has a 2m x 2m opening located 1.0m from its east side and central to its long axis as shown in the sketch of Figure (1) below:

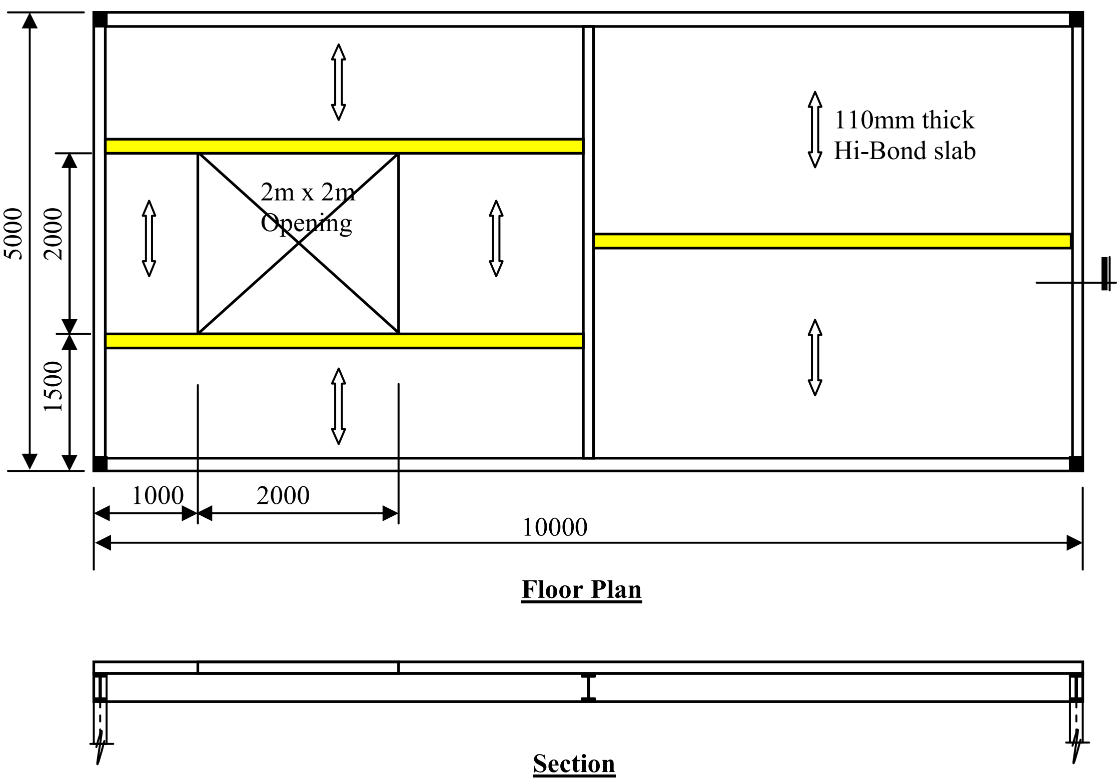

Figure (1) General arrangement of Hi-Bond slab design example

The hybrid slab is composed of 0.75 mm thick Dimond^{TM} Hi-Bond decking with 110mm thick concrete topping. The slab is designed to support a live load of 1.5 kPa plus a SDL of 0.85kPa, the sum of which is less than its maximum medium and long-term superimposed load capacity of 8.2 kPa as obtained from tables 10, 11, 12 and 13 of Dimond's "Hi-Bond Design Manual No.7" [6]. Also based on these tables above the slab requires a minimum negative steel reinforcement area of 175 mm^2/m above the interior supports in the direction parallel to its ribs. From geometry of Hi-Bond sheet the maximum height of the sheet section is 55mm. This gives a minimum diaphragm thickness of 55 mm on which the analysis and design of Dramix SFRC slab example are based.

4. TEMPERATURE AND SHRINKAGE REINFORCEMENT

According to Clause 8.8.1 of NZS3101:2006 [1] a minimum ratio of reinforcement area to gross concrete area of $0.7/f_y$, but not less than 0.0014, is required to resist temperature and shrinkage stresses in the slab.

Hence for an "h" mm thick slab with a concrete strength f'_c (or f_{fck}) of 25 MPa and reinforced conventionally with a welded wire mesh of yield strength f_y of 485 MPa the minimum required temperature and shrinkage reinforcement area A_{smin} is:

$$\rho_{\min} = \frac{0.7}{f_y} = \frac{0.7}{485} = 0.00144 > 0.0014 \qquad O.K.$$
$$A_{s\min} = 0.00144 \times h \times 10^3 = 1.44\,h \;\; mm^2/m \tag{1}$$

resulting a tensile force F_{ymin} provided by A_{smin} as:

$$F_{y\min} = A_{s\min} f_y = \frac{1.44\,h \times 485}{10^3} = 0.7\,h \;\; kN/m \tag{2}$$

In order to replace this tensile resistance of conventionally reinforced slab by an alternative Dramix SFRC topping a minimum characteristic axial tensile strength of SFRC $f_{fctk,ax}$ as calculated below is required:

$$f_{fctk,ax} = \frac{F_{y\min}}{A_g} = \frac{0.7\,h \times 10^3}{h \times 10^3} = 0.7\ MPa \tag{3}$$

It can be noted that the value of $f_{fct,ax}$ above is independent of the slab thickness and hence it is valid for any thickness. Based on the stress block given in Appendix A to the Commentary of Section 5 of NZS3101:2006 [2] the equivalent mean flexural tensile strength of the alternative Dramix SFRC $f_{fctm,eq}$ is:

$$f_{fctm,eq} = \frac{f_{fctk,ax}}{0.37} = \frac{0.7}{0.37} = 1.89\ MPa \tag{4}$$

For f_{fck} of 25 MPa the mean flexural tensile strength $f_{fctm,fl}$ is:

$$f_{fctm,fl} = \frac{0.3(f_{fck})^{2/3}}{0.6} = 0.5 \times (25)^{2/3} = 4.27\ MPa \tag{5}$$

The equivalent mean flexural strength ratio R_{em} is then calculated as:

$$R_{em} = \frac{100 \times f_{fctm,eq}}{f_{fctm,fl}} = \frac{100 \times 1.89}{4.27} = 44.26 \tag{6}$$

Based on energy absorption of the standard toughness index test of the 450mm long Dramix SFRC specimen, R_{em} can be evaluated for specimen deflection limits of L/300 = 1.5mm and L/150 = 3.0mm and is therefore designated as $R_{em,150}$ and $R_{em,300}$ respectively. If Dramix RC80/60BN steel fibre is used then an aspect ration λ_f of 80, fibre length L_f of 60, fibre diameter d_f of 0.75mm and a constant factor C of 20 will be substituted in R_{em} formulae provided in Dramix Guidelines Nr. 4–1995 [4] and Bekaert Design Guidelines [5] as follows:

$$R_{em,300} = \frac{180\, W_f\, \lambda_f}{180\,C + W_f\, \lambda_f} = \frac{180 \times 80 \times W_f}{180 \times 20 + 80 \times W_f} \tag{7}$$

$$R_{em,150} = \frac{180\, W_f\, \lambda_f\, (d_f)^{1/3}}{180\,C + W_f\, \lambda_f (d_f)^{1/3}} = \frac{180 \times 80 \times (0.75)^{1/3}\, W_f}{180 \times 20 + 80 \times (0.75)^{1/3}\, W_f} \tag{8}$$

Setting the above two ratios equal to 44.26 as calculated above will result values of fibre dosage W_f of 14.7 kg/m^3 and 16.1 kg/m^3 corresponding to $R_{em,300}$ and $R_{em,150}$ respectively. Hence higher fibre dosage W_f of 16.1 kg/m^3 could be specified to meet the minimum temperature and shrinkage stress control requirements of . On the basis of the calculations above Table (1) below is prepared to provide fibre dosages required to satisfy the minimum temperature and shrinkage stress control requirements for Dramix RC80/60BN covering concrete strengths ranging between 20 MPa and 50 MPa.

Table (1): Minimum fibre dosages required for temperature and shrinkage stress control

Concrete Strength $f_c = f_{fck}$ (MPa)	Dramix RC80/60BN	
	$W_{f,300}$ (kg / m^3)	$W_{f,150}$ (kg / m^3)
20	18.0	19.8
25	14.7	16.1
30	12.5	13.8
35	11.0	12.1
40	9.9	10.9
45	9.0	9.9
50	8.2	9.1

5. AXIAL TENSILE STRENGTH OF DRAMIX SFRC DIAPHRAGM

The design axial tensile strength of SFRC can be calculated using the stress block derived in Clause C5.A7 of NZS3101:2006 Part 2 [2].

For a concrete strength f'_c (or f_{fck}) of 25MPa and a dosage W_f of 20 kg/m^3 Dramix RC80/60BN steel fibre of an aspect ration λ_f of 80, fibre length L_f of 60, fibre diameter d_f of 0.75mm and a constant factor C of 20, the equivalent mean flexural strength ratio R_{em} can be calculated using the two formulae provided in Dramix Guidelines Nr. 4 – 1995 [4] and Bekaert Design Guidelines [5] for the two energy absorption limits of L/300 and L/150 of a standard toughness index Dramix SFRC specimen. These are:

$$R_{em,300} = \frac{180\,W_f\,\lambda_f}{180\,C + W_f\,\lambda_f} = \frac{180\times 20\times 80}{180\times 20 + 20\times 80} = 55.38 \tag{9}$$

$$R_{em,150} = \frac{180\,W_f\,\lambda_f\,(d_f)^{1/3}}{180\,C + W_f\,\lambda_f\,(d_f)^{1/3}} = \frac{180\times 20\times 80\times (0.75)^{1/3}}{180\times 20 + 20\times 80\times (0.75)^{1/3}} = 51.78 \tag{10}$$

The higher R_{em} of 55.38 should be considered. The equivalent mean flexural tensile strength of Dramix SFRC $f_{fctm,eq}$ can be determined as:

$$f_{fctm,eq} = \frac{R_{em}\times 0.5\,(f_{fck})^{2/3}}{100} = \frac{55.38\times 0.5\,(25)^{2/3}}{100} = 2.37\ MPa \tag{11}$$

The design axial tensile strength of Dramix SFRC $f_{fct,ax}$ in MPa (or in kN per 1.0m width per 1.0mm thickness of the slab) can then be calculated as:

$$f_{fct,ax} = 0.37\,f_{fctk,eq} = 0.37\times 2.37 = 0.88\ MPa\ (or\ kN/m/mm\ thickness\ of\ slab) \tag{12}$$

For a 110mm thick slab the design axial tensile strength is therefore 0.88x110 ≈ 96 kN/m.

On the basis of the calculations above Table (2) below is prepared to provide the mean axial tensile strengths provided by Dramix RC-80/60-BN in concrete strengths ranging between 20 MPa and 50 MPa for fibre dosages of 20, 25, 30, 35 and 40 kg/m^3.

Table (2): Mean axial tensile strengths of SFRC with Dramix RC-80/60-BN fibres

Concrete Strength $f'_c = f_{fck}$ (MPa)	Slab Thickness (mm)	Dramix RC80/60BN Dosage W_f (kg/m^3)				
		20	25	30	35	40
		Axial tensile strength (kN/m.l)				
20	50	38	44	49	54	58
	60	45	53	59	64	69
	70	53	61	69	75	81
	80	60	70	79	86	92
	90	68	79	88	97	104
	100	75	88	98	107	115
	110	83	96	108	118	127
	120	91	105	118	129	139
	130	98	114	128	140	150
	140	106	123	137	150	162
	150	113	131	147	161	173
25	50	44	51	57	62	67
	60	53	61	68	75	80
	70	61	71	80	87	94
	80	70	81	91	100	107
	90	79	92	102	112	121
	100	88	102	114	125	134
	110	96	112	125	137	147
	120	105	122	137	149	161
	130	114	132	148	162	174
	140	123	142	159	174	188
	150	131	153	171	187	201
30	50	49	57	64	70	76
	60	59	69	77	84	91
	70	69	80	90	98	106
	80	79	92	103	113	121
	90	89	103	116	127	136
	100	99	115	129	141	151
	110	109	126	141	155	166
	120	119	138	154	169	182
	130	129	149	167	183	197
	140	138	161	180	197	212
	150	148	172	193	211	227

Table (2) Continued: Mean axial tensile strengths of SFRC with Dramix RC-80/60-BN fibres

Concrete Strength $f'_c = f_{fck}$ (MPa)	Slab Thickness (mm)	Dramix RC80/60BN Dosage W_f (kg/m^3)				
		20	25	30	35	40
		Axial tensile strength (kN/m.l)				
35	50	55	64	71	78	84
	60	66	76	86	94	101
	70	77	89	100	109	117
	80	88	102	114	125	134
	90	99	115	128	140	151
	100	110	127	143	156	168
	110	121	140	157	171	184
	120	132	153	171	187	201
	130	143	165	185	203	218
	140	153	178	200	218	235
	150	164	191	214	234	252
40	50	60	70	78	85	92
	60	72	83	93	102	110
	70	84	97	109	119	128
	80	96	111	125	136	147
	90	108	125	140	153	165
	100	120	139	156	170	183
	110	132	153	171	187	202
	120	144	167	187	204	220
	130	156	181	203	222	238
	140	168	195	218	239	257
	150	180	209	234	256	275
50	50	70	81	90	99	106
	60	83	97	108	119	128
	70	97	113	127	138	149
	80	111	129	145	158	170
	90	125	145	163	178	191
	100	139	161	181	198	213
	110	153	178	199	218	234
	120	167	194	217	237	255
	130	181	210	235	257	276
	140	195	226	253	277	298
	150	209	242	271	297	319

6. SHEAR STRENGTH OF DRAMIX SFRC DIAPHRAGM

Equation (C5A-8) of Clause C5.A4.2.1 of Appendix A to C5 of NZS3101:2006 Part 2 [2] determines the design shear strength $V_{rd,3}$ for the Dramix SFRC as the sum of shear resistance of the section without shear reinforcement V_{cd} as given by equation (C5A-9) plus the contribution of steel fibre shear reinforcement V_{fd} as given by equation (C5A-11). Dramix Guidelines Nr. 4 – 1995 [4] and Bekaert Design Guidelines [5] provide a limiting condition that in no case $V_{rd,3}$ should exceed twice V_{fd}. This implies that the design shear strength $V_{rd,3}$ for the Dramix SFRC slab example in this report is determined as follows:

Based on the same assumed values stated in Section (5) of this paper regarding concrete strength and Dramix steel fibre dosage and properties, and neglecting bar reinforcement in the slab, V_{cd} can be calculated as:

$$V_{cd} = 0.08\sqrt{f_{fck}}\, b_w\, d = \frac{0.08\times\sqrt{25}\times 1\times 10^3}{1000} = 0.4\ kN/m\ width/mm\ thickness\ of\ slab \qquad (13)$$

Considering γ_c =1.5, n = 0.0 and k_1= 1.0, V_{fd} can be calculated as follows:

$$f_{fctk,ax} = 0.7\, f_{fctm,ax} = 0.7\times 0.3(f_{fck})^{2/3} = 0.21\times(25)^{2/3} = 1.8\ MPa \qquad (14)$$

$$R_t = \frac{1.1 W_f\, \lambda_f}{180C + W_f\, \lambda_f} = \frac{1.1\times 20\times 80}{180\times 20 + 20\times 80} = 0.34 \qquad (15)$$

$$\tau_{fd} = 0.54\, f_{fctk,ax}\, \frac{R_t}{\gamma_c} = 0.54\times 1.8\times\frac{0.34}{1.5} = 0.22\ MPa \qquad (16)$$

$$V_{fd} = k_1\, \tau_{fd}\, b_w\, d = \frac{1.0\times 0.22\times 1\times 10^3}{1000} = 0.22\ kN/m\ width/mm\ thickness\ of\ slab \qquad (17)$$

$V_{rd,3}$ for the Dramix SFRC slab is the smaller of:

1. $V_{rd,3} = V_{cd} + V_{fd} = 0.4 + 0.22 = 0.62\ kN/m\ width/mm\ thickness\ of\ slab$
2. $V_{rd,3} = 2V_{fd} = 2\times 0.22 = 0.44\ kN/m\ width/mm\ thickness\ of\ slab$

and for a 110mm thick slab the design shear strength is 0.44 x 110 ≈ 48 kN/m width.

Therefore, based on the calculations above Table (3) below is prepared to provide the design shear strengths of Dramix RC80/60BN in concrete strengths varying from 20 MPa to 50 MPa for fibre dosages of 20, 25, 30, 35 and 40 kg/m^3.

Table (3): Design shear strengths of SFRC with Dramix RC-80/60-BN fibres

Concrete Strength $f'_c = f_{fck}$ (MPa)	Slab Thickness (mm)	Dramix RC80/60BN Dosage W_f (kg/m^3)				
		20	25	30	35	40
		Shear strength (kN/m.l)				
20	50	19	22	25	27	29
	60	23	26	29	32	35
	70	26	31	34	38	40
	80	30	35	39	43	46
	90	34	39	44	48	52
	100	38	44	49	54	58
	110	41	48	54	59	63
	120	45	53	59	64	69
	130	49	57	64	70	75
	140	53	61	69	75	81
	150	57	66	74	80	87
25	50	22	25	28	31	33
	60	26	30	34	37	40
	70	31	36	40	44	47
	80	35	41	46	50	54
	90	39	46	51	56	60
	100	44	51	57	62	67
	110	48	56	63	68	74
	120	53	61	68	75	80
	130	57	66	74	81	87
	140	61	71	80	87	94
	150	66	76	85	93	100
30	50	25	29	32	35	38
	60	30	34	39	42	45
	70	35	40	45	49	53
	80	40	46	51	56	60
	90	44	52	58	63	68
	100	49	57	64	70	76
	110	54	63	71	77	83
	120	59	69	77	84	91
	130	64	75	83	91	98
	140	69	80	90	98	106
	150	74	86	96	105	113

Table (3) Continued: Design shear strengths of SFRC with Dramix RC-80/60-BN fibres

Concrete Strength $f'_c = f_{fck}$ (MPa)	Slab Thickness (mm)	Dramix RC80/60BN Dosage W_f (kg/m^3)				
		20	25	30	35	40
		Shear strength (kN/m.l)				
35	50	27	32	36	39	42
	60	33	38	43	47	50
	70	38	44	50	54	59
	80	44	51	57	62	67
	90	49	57	64	70	75
	100	55	64	71	78	84
	110	60	70	78	86	92
	120	66	76	85	93	100
	130	71	83	93	101	109
	140	77	89	100	109	117
	150	82	95	107	117	126
40	50	30	35	39	43	46
	60	36	42	47	51	55
	70	42	49	54	60	64
	80	48	56	62	68	73
	90	54	63	70	77	82
	100	60	69	78	85	92
	110	66	76	86	94	101
	120	72	83	93	102	110
	130	78	90	101	111	119
	140	84	97	109	119	128
	150	90	104	117	128	137
50	50	35	40	45	49	53
	60	42	48	54	59	64
	70	49	56	63	69	74
	80	56	64	72	79	85
	90	63	73	81	89	96
	100	69	81	90	99	106
	110	76	89	99	109	117
	120	83	97	108	119	127
	130	90	105	117	128	138
	140	97	113	126	138	149
	150	104	121	135	148	159

7. SEISMIC LOADING

The seismic actions applied on the floor slab example described in section (3) are based on the following parameters in accordance with provisions of AS/NZS 1170.0:2004 Structural Design Actions, Part 0: General Principles [7] and Part 5: Earthquake actions – NZ [8]:

- Site Subsoil Category: C - Shallow Soils (Clause 3.1.3.4)
- Fundamental Translational Period, T = 0.4 seconds
- Basic Seismic Hazard Acceleration Coefficient, $C_h(T)$ = 2.36 (Table3.1)
- Hazard Factor, Z = 0.13 "Auckland" (Table 3.3)
- Importance Level = 2 (Table 3.2 – AS/NZS 1170.0:2004)
- Design Life = 50 years
- Annual Probability of Exceedance =1/500 (Table 3.3 – AS/NZS 1170.0:2004)
- Return Period Factor for ULS, R_u = 1.0 (Table 3.5)
- Near-Fault Factor, N(T,D) = 1.0 (Clause 3.1.6)
- Elastic Site Hazard Spectrum Horizontal Loading C(T) = 0.307 (Equation 3.1)
- Ductility Factor μ = 1.25 "Nominally Ductile Structure" (Clause 2.2.3)
- Structural Performance Factor, S_p = 0.925 (Clause 4.4.2)
- K_μ = 1.14 (Clause 5.2.1.1)

The ultimate limit state horizontal design action coefficient, $C_d(T_1)$ can then be calculated as per Equations 5.2(1 and 2) of Clause 5.2.1.1 [8] as follows:

$$C_d(T_1) = \frac{C(T_1)\, S_p}{k_\mu} = \frac{0.307 \times 0.925}{1.14} = 0.249 \tag{18}$$

Based on Table 6 of Dimond's Hi-Bond Design Manual No.7 [6], a 110mm thick slab weighs 1.99 kN/m^2. The total seismic load on the floor slab can be calculated as follows considering a SDL of 0.85 kPa and a live load of 1.5 kPa:

$$\begin{aligned}
&Slab\ self\ \ weight = 1.99 \times [(10 \times 5) - (2 \times 2)] = 91.5\ kN \\
&Slab\ SDL = 0.85 \times [(10 \times 5) - (2 \times 2)] = 39.1\ kN \\
&Slab\ LL = 0.4 \times 1.5 \times [(10 \times 5) - (2 \times 2)] = 27.6\ kN \\
&Total\ slab\ seismic\ load, W_t = 158.2\ kN
\end{aligned}$$

The live load factor Ψ_u for seismic ULS load combination is taken as 0.4, whereas the live load area reduction factor Ψ_a is conservatively taken as 1.0. The equivalent static lateral force F_i at ULS acting on the slab is considered to be equal to the total horizontal seismic shear, V and can be determined in accordance with Equation 6.2(1) of AS/NZS 1170.5:2004 [8] as follows:

$$F_i = V = C_d(T_1)\, W_t = 0.249 \times 158.2 = 39.37\ kN \tag{19}$$

8. FINITE ELEMENT ANALYSIS (FEA) OF DIAPHRAGM

The FEA of the example floor slab is based on a computer model of a diaphragm comprising of quadrilateral plate elements representing the concrete floor. The material properties attributed to concrete elements are: a modulus of elasticity of 25 GPa, a density of 25 kN/m^3 and a Poisson's ratio of 0.2. An element thickness of 55 mm is considered, which reflects the thickness of the continuous solid concrete diaphragm above the top of the profiled sheet metal ribs. The maximum size of quadrilateral elements is 1.0 x 1.0m; however element sizes and dimensions are automatically adjusted to demonstrate the peak stress at locations immediately surrounding the slab opening.

In order to include the torsional shear effects resulting from the accidental eccentricity provided in Clause 5.3.2 of AS/NZS 1170.5:2004 [8] and the effect of non-symmetrical location of the 2.0 x 2.0m opening on offsetting both mass and rigidity centres from the geometrical centre of rectangular area, it is considered more appropriate to apply the ultimate seismic lateral force on the diaphragm by means of a uniform load of a conservative value of 50 kN distributed along the 10 m long side of the diaphragm. As the most critical direction of the seismic force is taken in the north-south direction, a vertical seismic ultimate UDL of 5kN/m is hence applied on the northern side with the diaphragm being simply supported at its southern two corners as shown in Figure (2).

Elastic FEA is carried out for the diaphragm model with total numbers of 71 nodes and 52 elements automatically generated for the model mesh. Once the FEA is complete a comparison of the deformed shape to the original is presented as shown in Figure (3). Figures (4 to 6) below show the FEA output.

It is clear from Figures (4 and 5) showing the maximum and minimum principal stress contours that most of the diaphragm area is either under compression or under tensile stresses of magnitudes less than 0.6 MPa, except for the bottom side (cord) of diaphragm and the north-east/south-west corners of the opening. Maximum principal tensile stresses of 0.6-2.06 MPa and 0.6-1.40 MPa across the element width are computed along the bottom cord and corners of the opening, respectively. Tension stresses in both places are aligned to the east-west direction as shown in the stress vector diagram.

Considering concrete strength of 25 MPa with a dosage of 20 kg/m^3 of Dramix RC80/60BN steel fibres, and referring to the Table (2) presented in section (5), an interpolated axial tensile strength of 48 kN/m.l can be obtained. This corresponds to tensile capacity of 0.876 MPa, which is greater than the tensile stress of 0.6 MPa present in most of the diaphragm area. Also the dosage of 20 kg/m^3 of Dramix RC65/60BN steel fibres is greater than 16.1 kg/m^3 required as a minimum dosage for shrinkage/temperature stress control.

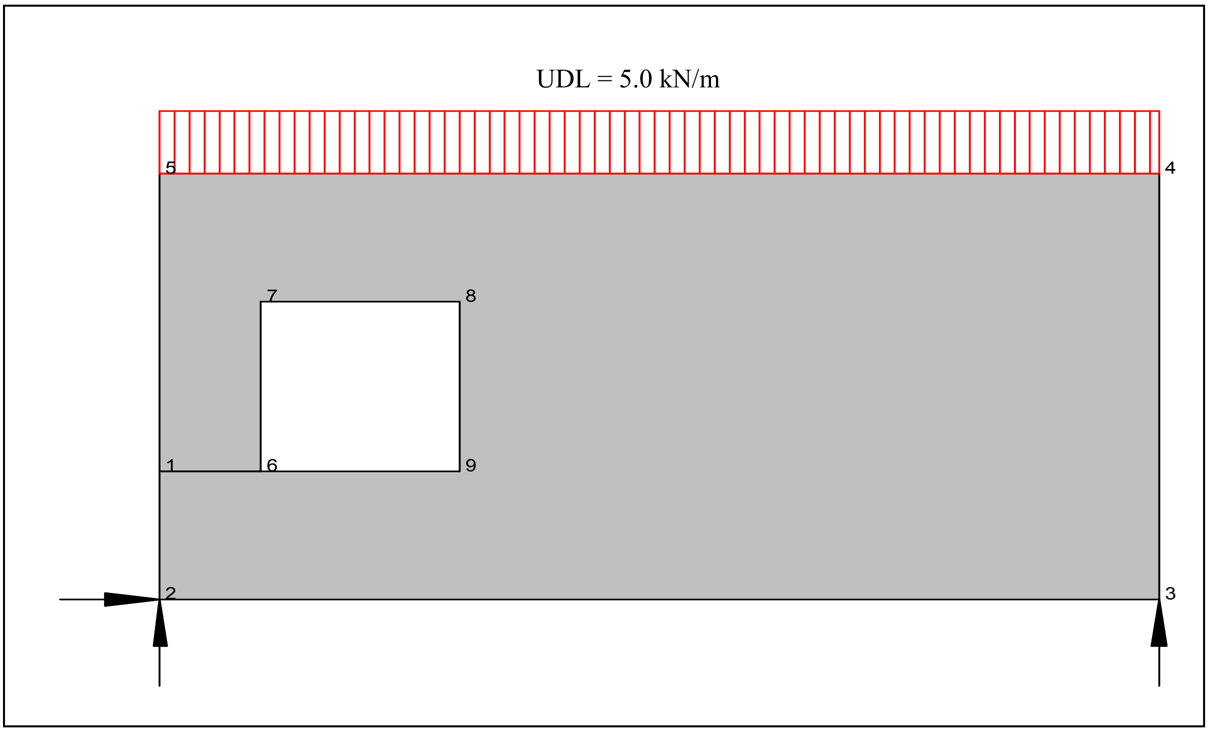

Figure (2) Diaphragm geometry with supports and loading

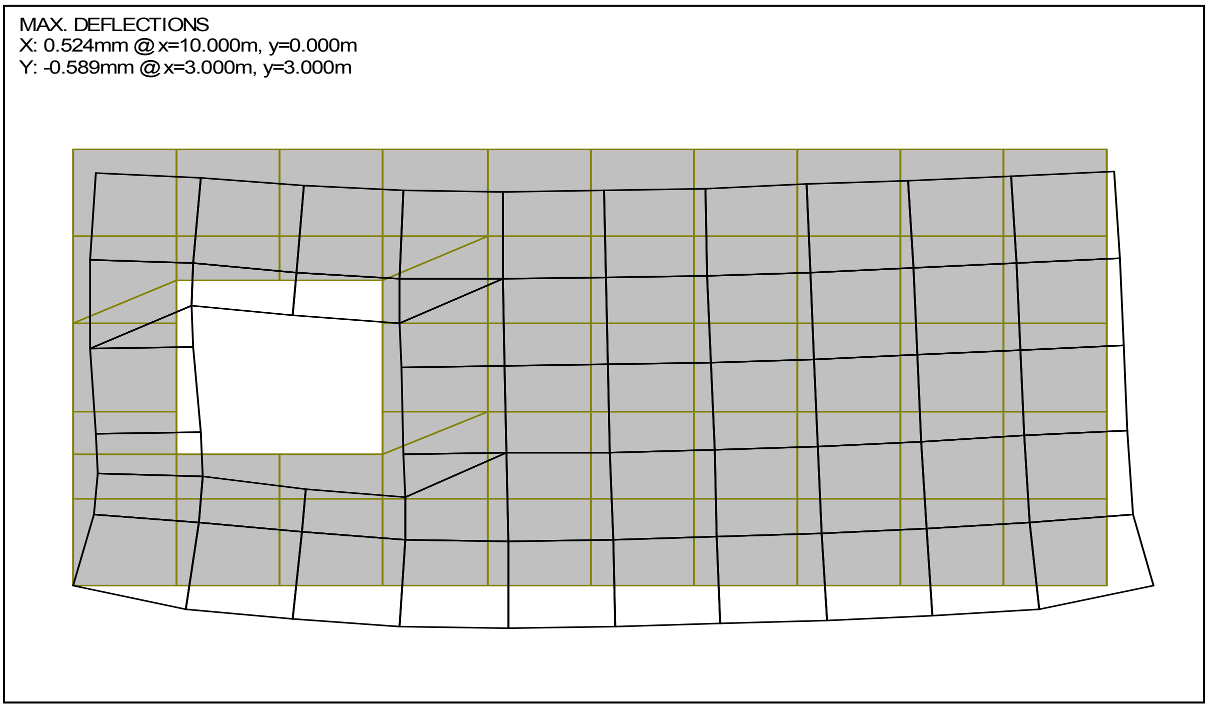

Figure (3) Deformed/original shapes showing the FEA meshes of both cases

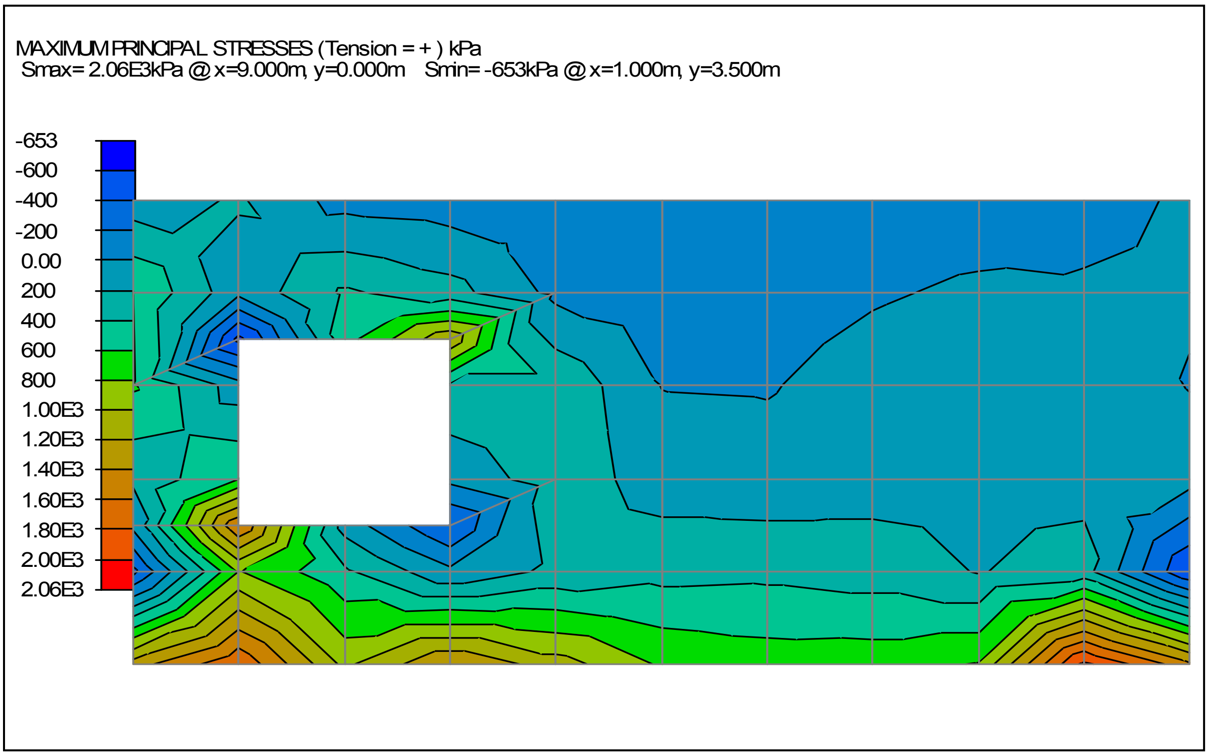

Figure (4) Maximum principal stress contours

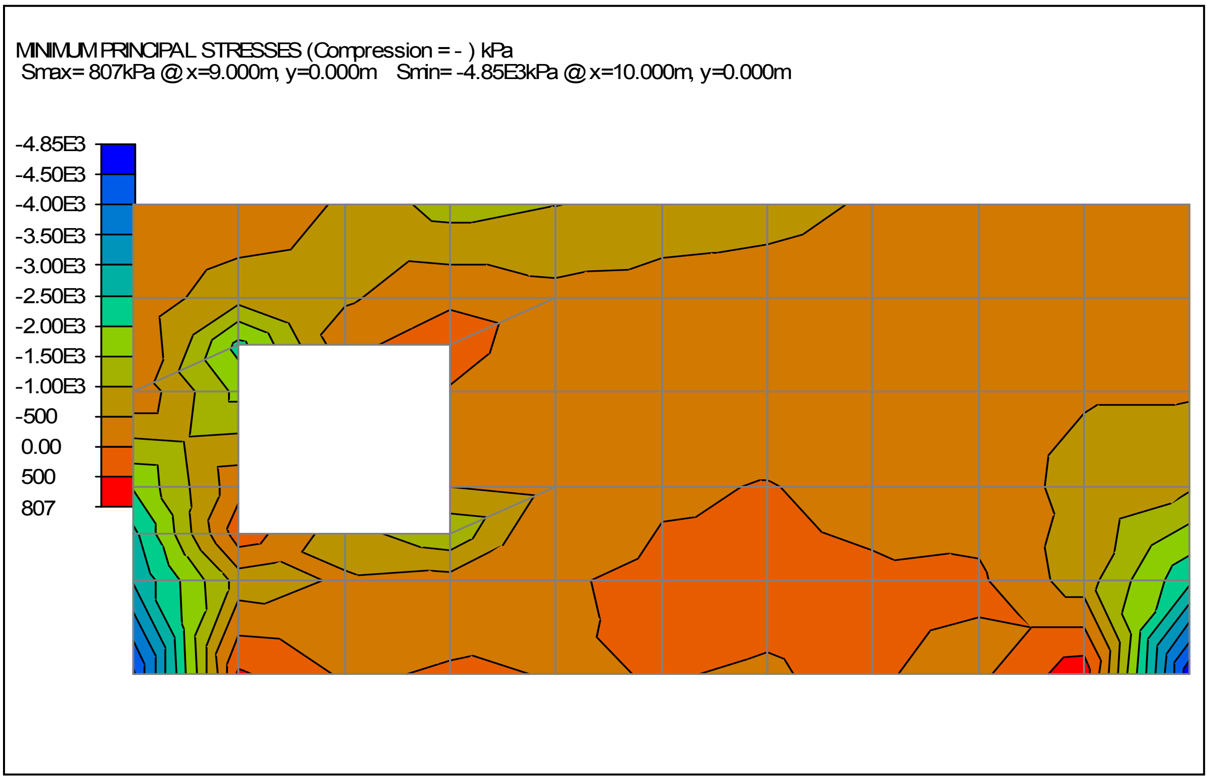

Figure (5) Minimum principal stress contours

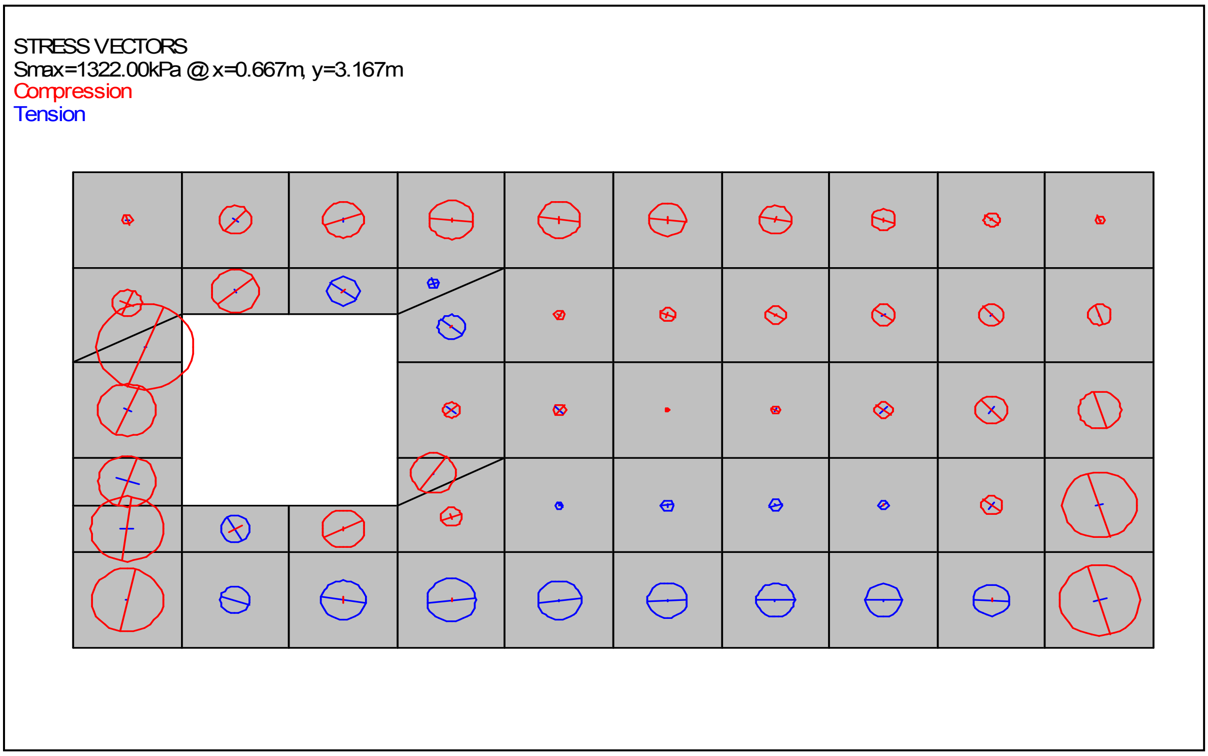

Figure (6) Stress vectors for tensile and compressive stress directions

For the bottom cord of slab and top/bottom sides of the opening additional tensile strength is required, which can be supplied by providing trimmer conventional reinforcing bars at these locations as follows:

$$Max\ net\ tensile\ stress = Average\ tensile\ stress - SFRC\ tensile\ strength$$

$$Max\ net\ tensile\ stress = \frac{2.06 + 0.6}{2} - 0.876 = 0.46\ MPa \tag{20}$$

$$Max\ net\ tensile\ force\ per\ element\ width = 0.46 \times 55 \times 1000 = 25{,}300\ Newtons$$

Assuming a grade f_y = 500 MPa reinforcing trimmer bars, and a strength reduction factor for tension in plain concrete, ϕ of 0.6 as provided in Clause 2.3.2.2 (g) of NZS 3101:2006 [1] the reinforcement area required can be determined as:

$$A_s = \frac{F_t}{\phi\ f_y} = \frac{25{,}300}{0.6 \times 500} = 84.34\ mm^2 \tag{21}$$

Therefore HD12 trimmer bar with A_s of $113mm^2$ will be adequate. Given the seismic load is of a reversible nature in both orthogonal directions, thus HD12 trimmer bars are required at four sides of the diaphragm and at the perimeter around the slab opening. All trimmer bars should be adequately anchored to the requirements of Clause 13.3 of NZS 3101:2006 [1] as shown in Figure (7) below:

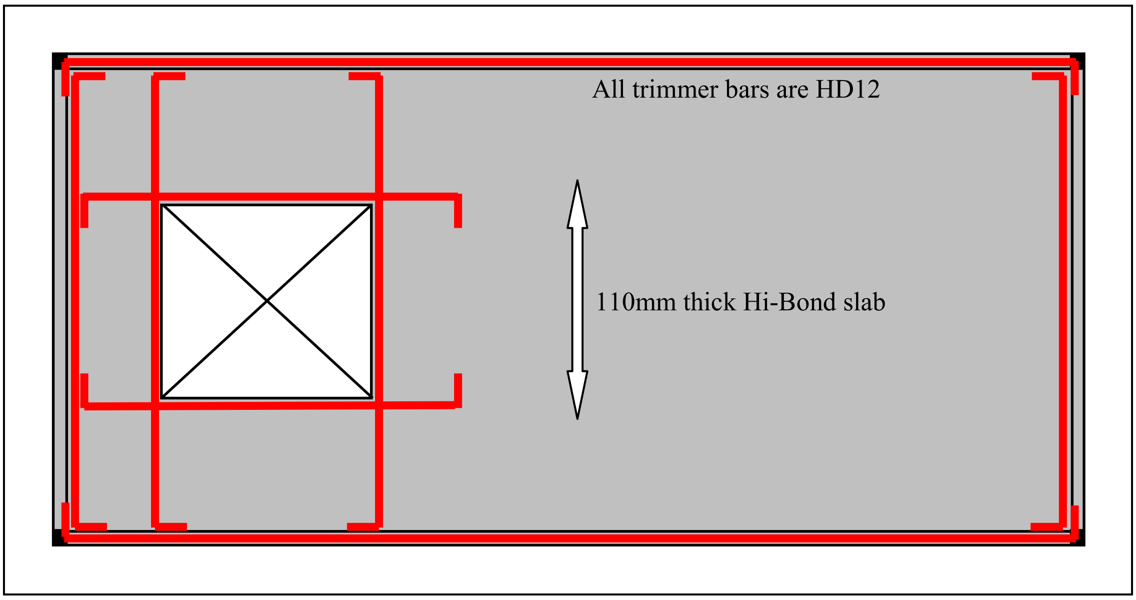

Figure (7) Trimmer reinforcing bar details after FEA method

Alternatively, a conservative approach would be to neglect the contribution from the Dramix SFRC and size the trimmer bars based on the capacity of the steel on its own. The maximum shear stress of the diaphragm with the supporting lateral load resisting frame beam can be determined along the shortest (5.0m long) side as follows:

$$v_u = \frac{5 \times 10000}{2 \times 5000 \times 55} = 0.091\ MPa \tag{22}$$

For the selected SFRC concrete of 25 MPa strength with 20 kg/m^3 of Dramix RC80/60BN steel fibres, and referring to the Table (3) presented in section (6), an interpolated shear strength value of 24 kN/m.l. can be obtained. This corresponds to a shear capacity 0.437MPa, which is greater than v_u of 0.091 MPa. Hence no shear-friction reinforcing bars are required to tie the diaphragm to the perimeter beams.

9. STRUT-AND-TIE MODEL OF DIAPHRAGM

The strut-and-tie model of the example diaphragm is generated in accordance with the provisions of Appendices A and CA – Strut-and-tie models of NZS 3101:2006 Parts 1 and 2 [1,2].

Initially a truss model is idealized using struts and ties joined at nodes to establish an admissible path of internal forces in equilibrium with ULS seismic load and end reaction. The seismic load is considered as more appropriate to be the 39.37 kN concentrated ULS point load applied at the geometrical centre point location of the rectangular area of the diaphragm in the north-south. This location is selected so that it will produce the smallest possible angle of 26.5° between bottom cord and strut at node 2 as shown in Figure (8). Such angle is greater than the minimum allowable angle of 25° provided in Clause A 4.5.

The truss model is solved to determine the member forces which are assigned to each strut as "S" and tie as "T". Struts consisting primarily of concrete are assigned compression forces and ties consisting of trimmer reinforcing bars around the slab and opening sides, are the tension members. Figure (8) below shows the idealized truss with five struts and three ties joined at six nodes. It is essential that the geometry of truss shown complies with requirements of Appendix A of NZS3101:2006 [1].

The thickness of struts is limited by the diaphragm thickness of 55 mm, whereas the critical widths are controlled by the strength of the strut cross sections and the nodal zone surfaces to which the struts and/or ties are joined as provided in Clause A7. The nominal compressive strength of each strut is determined in accordance with Clause A5 of NZS 3101:2006 [1] whereas that of a nodal zone is calculated in accordance with Clauses A7.1 and A7.2.

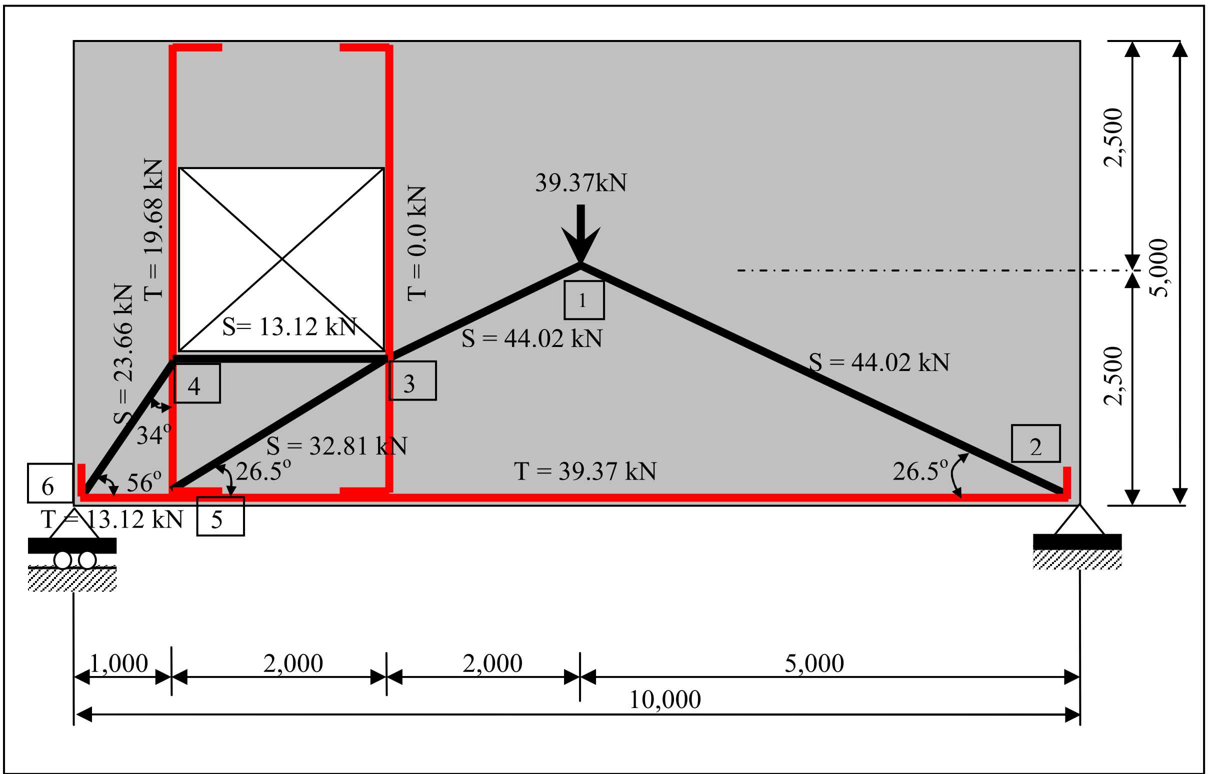

Figure (8) Strut-and-Tie truss model for downward north/south earthquake load

A 25 MPa concrete strength is considered with 20 kg/m^3 dosage of Dramix RC80/60BN steel fibres, which is greater than 16.1 kg/m^3 required for shrinkage and temperature stress control. Consequently, the widths of all struts and nodal zone surfaces are obtained and shown in Figure (9).

It is determined from the analyzed truss above that the maximum tensile force occurs at the bottom cord (tie) of the truss and is equal to 39.37 KN. Assuming a Grade f_y = 500 MPa reinforcing trimmer bars, and a strength reduction factor for strut-and-tie models, ϕ of 0.75 as provided in Clause 2.3.2.2 (h) of NZS 3101:2006 [1] the reinforcement area required can be determined as:

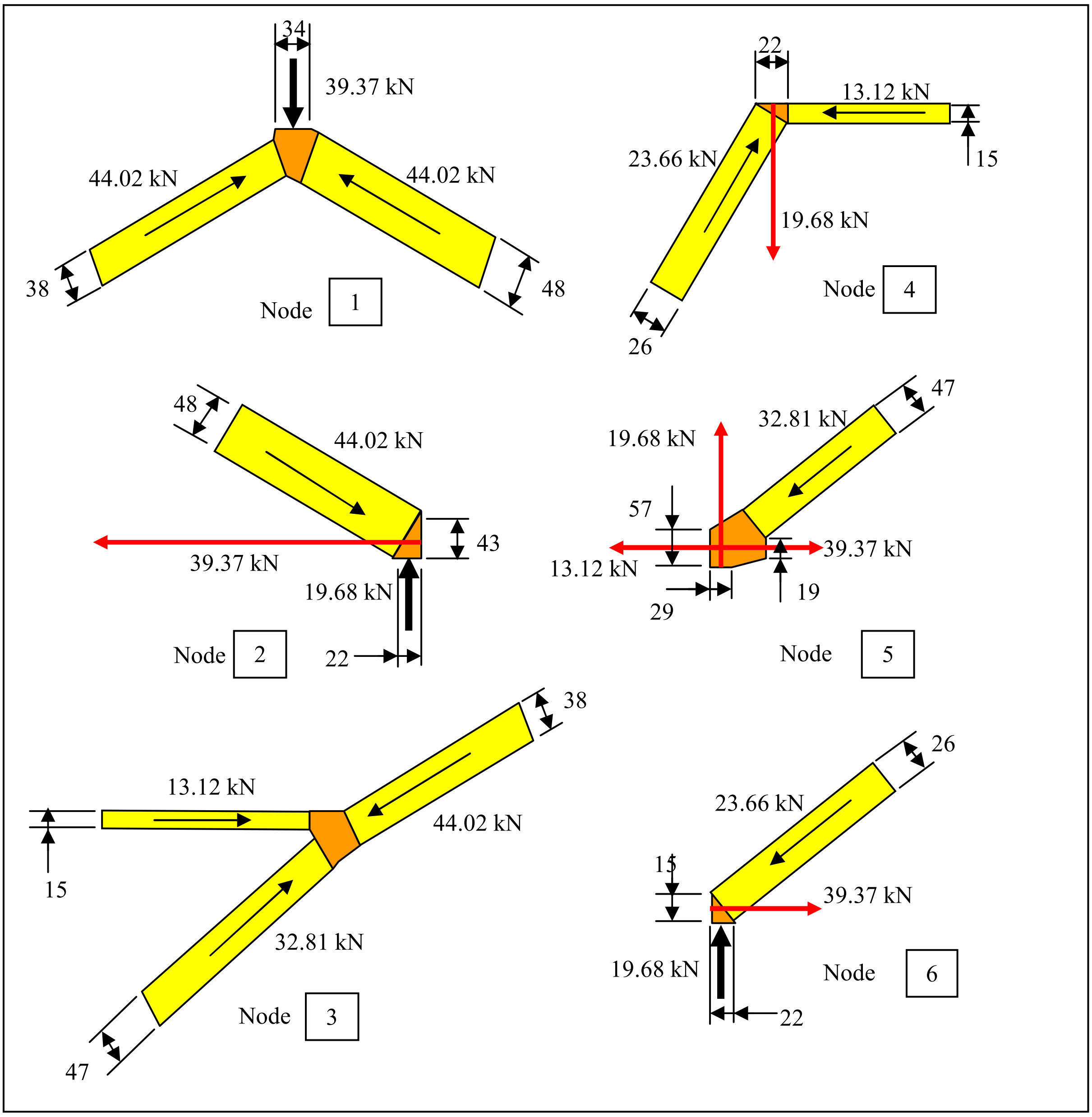

Figure (9) : Truss nodal forces/reactions with widths of struts and nodal zone surfaces

$$A_s = \frac{F_t}{\phi \ f_y} = \frac{39.37 \times 1000}{0.75 \times 500} = 105 \ mm^2 \qquad (23)$$

Therefore HD12 trimmer bar with A_s of 113 mm^2 will be adequate. Given the seismic load is of a reversible nature in both orthogonal directions, thus HD12 trimmer bars are required at four sides of the diaphragm and at the perimeter around the slab opening.

All trimmer bars should be adequately anchored to the requirements of Clause 13.3 of NZS 3101:2006 [1] as shown in Figure (10).

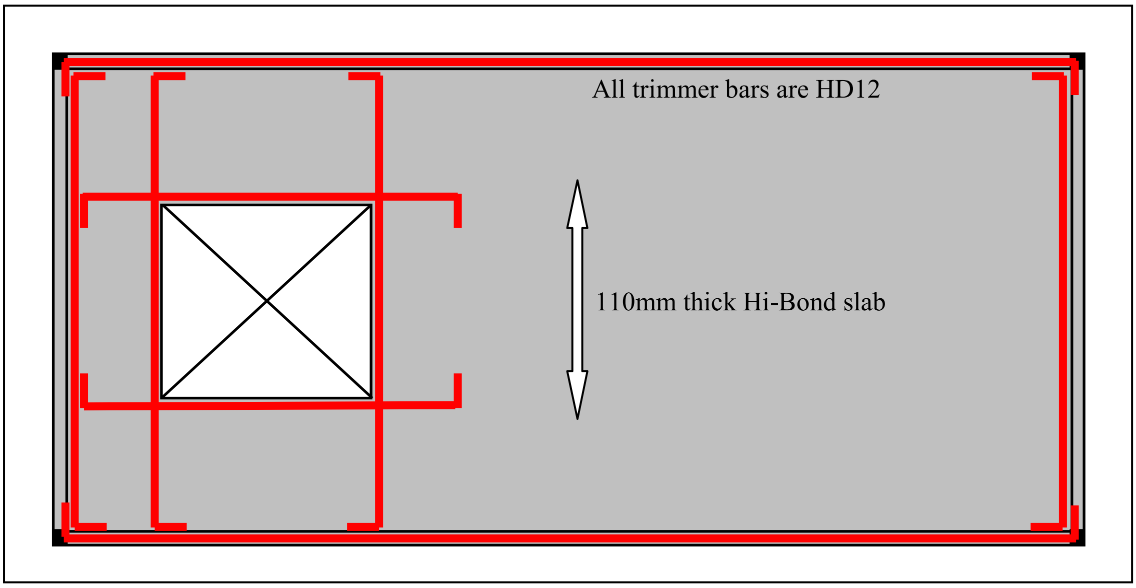

Figure (10) Trimmer reinforcing bar details after strut-and-tie method

The maximum shear stress of the diaphragm with the supporting lateral load resisting frame beam can be determined as follows:

$$v_u = \frac{39370}{2 \times 5000 \times 55} \times 1.25 = 0.09\ MPa \tag{24}$$

The factor of 1.25 is used to allow for the effects of non-symmetrical location of the opening and accidental eccentricity on the shear stress at the supporting beams. For the selected

SFRC concrete of 25 MPa strength and 20 kg/m^3 of Dramix RC80/60BN steel fibres, and referring to the Table (3) presented in section (6), interpolated shear strength of 0.437 MPa can be obtained, which is greater than v_u above. Hence no shear-friction reinforcing bars are required to tie the diaphragm to the perimeter beams.

10. CONCLUSIONS

— SFRC diaphragm alternative is shown to be an efficient and structurally adequate equivalent to conventional bar reinforced diaphragm.

— Dramix SFRC topping on profiled metal sheeting satisfies the requirements of Section 8 “Stress Development, Detailing and Splicing of Reinforcement and Tendons”, Section 13 “Design of Diaphragms” and Appendix A “Strut-and-Tie Models” of NZS3101:2006.

— Results of FEA and Strut-and-Tie models demonstrate that hybrid slab comprising of Dramix SFRC topping on Hi-Bond sheet metal is capable to transfer the lateral load to the supporting elements adequately through the diaphragm action.

— A design methodology is developed to determine the strength of concrete, size of trimmer reinforcing bars, starter bars and the steel fibre dosage required for a hybrid slab to resist in-plane diaphragm stresses calculated in accordance with the provisions of NZS3101:2006.

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