Austral Deck
Design for Construction Loading

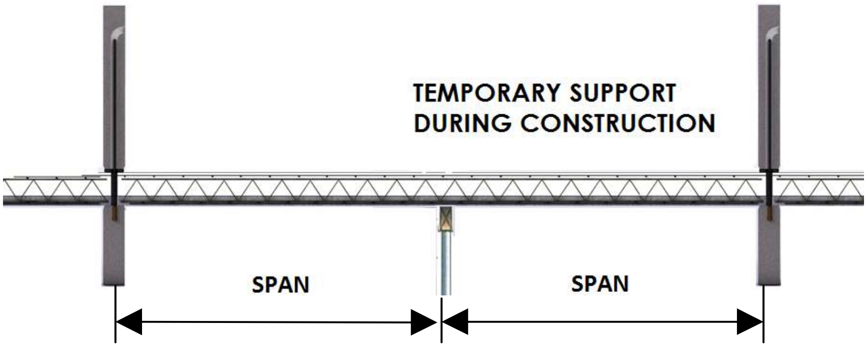
Permanent formwork
and
Span capability

Introduction

The purpose of this document is to demonstrate the process of designing Austral Deck as formwork complying with the stability, strength and serviceability limit state criteria specified in AS 3610-1995 Formwork for Concrete and AS3610.1-2010 Formwork for concrete – Part 1 Documentation and surface finish.

The process is used to determine the maximum spanning capacity of Austral Deck based on the structural properties and construction loads in order to define the temporary propping requirement of a concrete slab constructed using Austral Deck.

The specific calculations in this document apply to a simply supported double span of an Austral Deck panel for the variable structural properties and construction loads listed in page 4. However the calculations also provide guidance for other situations.



Graphs in Appendix A can be used as a guide for Austral Deck as formwork under construction loads for various slab thickness and steel lattice girder reinforcement configurations.

Austral Deck – Construction Load Analysis software can be used to determine prop spacing for a variety of structural properties and construction loads. The software can be downloaded via the Austral Precast website www.australprecast.com.au

The information contained in this document is provided as a general guide only and does not replace the need for the specification of the steel and concrete requirements from a qualified structural design engineer.

Austral Precast has dedicated engineering services available for project assistance. We are able to provide design support for engineers to determine the best way to specify and document Austral Deck. Our technical experts can identify the most efficient panel geometry meeting project requirements, specifications and installation process.

Austral Precast offers fully detailed shop drawings as part of your Austral Deck supply. It is the client's designer who is responsible to design and certify the concrete slab, the entire structure and certify the panel design for temporary propping and construction loads.

Austral Deck Design principles

In general, the design of the Austral Deck system consists of two design stages

- 1st Stage : Design as formwork

Must be designed for
  - Lifting during manufacturing,
  - Transport,
  - Lifting during installation
  - Construction loads during use as formwork

- 2nd Stage: Final in-use slab design
  - Must be designed for strength and serviceability for in-service situation as per conventional reinforcement concrete suspended slab

This document only deals with the first stage, design as formwork. However, the final slab can be designed using conventional reinforced concrete methods based on the assumption that it acts as a monolithic reinforced concrete element. It is the designer's responsibility to satisfy himself/herself that the rough surface of the precast concrete and the presence of the embedded diagonal web reinforcement is sufficient to assume a monolithic concrete element.

Stage one: Design as formwork

The precast element of Austral Deck must be designed as formwork for construction loads since it is utilised as permanent formwork. The strength of the precast element required to span between temporary and permanent supports during construction is gained from the lattice girder type reinforcement that is partially embedded in the precast concrete.

The top chord of the lattice girder type reinforcement is not embedded in the precast concrete so Austral Deck must be designed as formwork using a combination of the Australian Standard for Concrete Structures (AS3600-2009) and Steel Structures (AS4100-1998) along with the Australian standard for loading (AS1170) and formwork (AS3610-2013).

It is recommended that the design engineer uses strict deflection criteria when designing as formwork for construction loads to alleviate any significant cracking.

Design criteria and specifications

Strength
The Panel must resist the bending and shear action effects from all the appropriate load combinations. In the case of a simply supported panel the following load combinations are appropriate:

Stage I – prior to placement of concrete

$$1.25G+1.5Q_{uv}+1.5M_1 \qquad (1)$$

Stage II – during placement of concrete.

$$1.25G+1.25G_C+1.5Q_{uv}+1.5M_2 \qquad (2)$$

$$1.25G+1.25G_C+Q_C \qquad (3)$$

Stage III – after placement of concrete.

$$1.25G+1.25G_C+1.5Q_{uv}+1.5M_3 \qquad (4)$$

If the panel is considered a primary member as per AS 3610 – 1995, then these loads must be multiplied by a factor of 1.3. For example:

Stage I – prior to placement of concrete.

$$1.3(1.25G+1.5Q_{uv}+1.5M_1) \qquad (5)$$

Stiffness
The panel stiffness must be such that the deformation under the appropriate load combination does not exceed chosen limits, using either the limits specified in AS 3610.1 – 2012, Table 3.3.2 (Form face deflection) or otherwise chosen. In the case of a simply supported panel the following load combinations are appropriate:

Stage II – during placement of concrete.

$$G+G_C+Q_{uv} \qquad (6)$$

Stage III – after placement of concrete.

$$G+G_C+Q_{uv}+M_3 \qquad (7)$$

While AS 3610 – 1995 does not require the addition of Q_{uv}, it has been added to be in line with the intentions of AS 1170.0 – 2002.

Surface Finish
The surface finish of the panel soffit conforms with the physical quality of a “Class 2” surface finish as specified in AS 3610.1 – 2010. The surface class chosen for the in situ slab is also used to define the maximum allowable deflection limits.
Panel Capacity

The strength and stiffness of the panel is dependent on the truss, panel size and geometry. During construction the applied loads are resisted by the action of the truss members and panel concrete. The resistance provided by any mesh or additional reinforcement bars is ignored.

The following structural checks are performed:

Strength	a) Top Chord Compression b) Top Chord Tension c) Bottom Chord Compression d) Bottom Chord Tension e) Concrete Panel Compression f) Diagonal Compression g) Concrete tensile strength
Service	a) Deflection b) Cracking

The maximum allowable span is calculated for each case.

Maximum Span

The maximum span is selected on the basis that the design action, calculated from the factored load combinations, does not exceed the capacity of the panel.

A summary of the calculations showing the maximum span for each action is given in the table below:

Design Action		Max. Span (m)
Positive Bending	Top Chord Compression Bottom Chord Tension	3.41 3.37
Negative Bending	Top Chord Tension Bottom Chord Compression Concrete Panel Compression	3.95 1.33 5.59
Shear	Diagonal Lacing Compression	5.13
Cracking	Concrete Tensile Strength Flexural cracking	2.23 3.96
Deflection	Serviceability deflection	3.05

The maximum span for the given configuration is therefore:

Maximum Span	3.05	m

The bottom chord compression limit can be ignored if the bottom chord is completely embedded in the panel concrete for its full length. Control for concrete tension can also be ignored as it is based on the uniaxial tensile strength of the concrete panel rather than the flexural tensile strength. This implies that the concrete is allowed to crack but is controlled by the limits set out in section 9.4.1 of AS3600-2009.

Stacked Materials

The maximum span loadings include factors M_1, M_2 and M_3 for the live loadings of stacked materials. AS 3610 – 1995 gives these values as 4.0 kPa for before and after the placement of concrete (M_1 and M_3), and 0 kPa for during the placement of concrete (M_2).

The maximum span may be increased by decreasing these loads. In such a case, this lowered load limit must be clearly indicated in the formwork documentation and construction control put in place to ensure it is not exceeded.

Assumptions

1) Vertical and horizontal action effects from environmental loads have been ignored (e.g. wind uplift, river currents). If relevant, appropriate strength and service loads should be calculated with reference to AS 3610 – 1995, Table 4.5.2.
2) The value for stacked materials during Stage I (M_1) applies also to Stage III (M_3) and during Stage II the value for stacked materials (M_2) is 0 kPa.
3) The effects of form face deflection and construction tolerances can be ignored.
4) The deviations specified for form face deflection, in AS 3610.1 – 2010, Table 3.3.2, will be interpreted as the deflection criteria for the panel as per the following extract:

Surface Class	Deflection Limit
1	Lesser of 2mm or span/360
2	Lesser of 3mm or span/270
3	Greater of 3mm or span/270
4	Greater of 3mm or span/270
5	N/A

5) The welds connecting diagonal wires to the top and bottom chord of the truss are capable of transmitting the full design action effects.

6) Truss geometry is as per the following table:

Truss Type	Wire Size (mm)			
	Top	Bottom	Diagonal	Height
T80/10	9.5	6.3	6.3	82
T110/10	9.5	6.3	6.3	111
T150/10	9.5	6.3	6.3	154
T190/10	9.5	6.3	6.3	191
T110/12	11.9	6.3	6.3	112
T150/12	11.9	6.3	6.3	155
T190/12	11.9	6.3	6.3	192

Panel Properties

Overall slab thickness, d	250	mm
Minimum Cover to Bottom Reinforcement	20	mm
Concrete Density, r	2400	kg/m^3
Concrete Strength at Loading, f_{cm}	20	MPa
Concrete Modulus of Elasticity, E_{cj}	22610	MPa
Panel Width, b	2500	mm
Precast Panel Thickness, t_p	75	mm
Number of Truss per Panel, n_t	10	
Number of Voids, n_v	0	
Void Width, b_v	NA	mm
Void Thickness, t_v	NA	mm
Class of Surface finish	2	
Number of spans (based on supports)	2	

The panel concrete properties (fcm, Ecj) are at the time of lifting from casting beds.

Construction Loads

Panel Dead Load, G	1.87	kPa
In-situ Slab Dead Load, G_C	4.29	kPa
Construction Live Load, Q_{uv}	1	kPa
Concrete Mounding Load, Q_C	3	kPa
Stacked Materials, M_1	4	kPa
Stacked Materials, M_2	0	kPa
Stacked Materials, M_3	4	kPa

Load Combinations

Stage	Load Combination	Load	Unit	Equation
	Strength			
I	$1.3(1.25G + 1.5Q_{uv} + 1.5M_1)$ **	12.78	kPa	(5)
II	$1.3(1.25G + 1.25G_c + 1.5Q_{uv} + 1.5M_2)$ **	11.96	kPa	
II	$1.3(1.25G + 1.25G_c + Q_c)$ *	15.86	kPa	
III	$1.3(1.25G + 1.25G_c + 1.5Q_{uv} + 1.5M_3)$ **	19.76	kPa	
	Stiffness			
II	$G + G_c + Q_{uv}$	7.16	kPa	(6)
III	$G + G_c + Q_{uv} + M_3$ **	11.16	kPa	(7)

* Although AS 3610 – 1995 states that the concentrated load Q_C will apply over an area of 1.6 x 1.6 m, it has been applied over the full area of the panel.

** The loads from stacked materials, M, may apply to one span only.

Design Load

Therefore the design loads are as follows:

Strength, w*	19.76	kPa
Service, w_s	11.16	kPa

Truss Properties

Austral Deck Truss Type, T	T190/12	
Average Truss Spacing, T_s	250	mm
Truss Height, T_h	192	mm
Truss Bar Yield Strength, f_{syt}	500	MPa

Top Chord

Bar Diameter, d_t	11.9	mm
Area, A_t	1112.2	mm^2
Strut Length, L_t	200	mm
Effective Length, l_t	180	mm
Radius of Gyration, r_t	2.98	mm

Bottom Chord

Bar Diameter, d_b	6.3	mm
Area, A_b	623.4	mm^2
Strut Length, L_b	200	mm
Effective Length, l_b	200	mm
Radius of Gyration, r_b	1.58	mm

Diagonal Lacing

Bar Diameter, d_w	6.3	mm
Area, A_w	623.4	mm^2
Angle of Web, q	62.5	degrees
Strut Length, L_w	216.5	mm
Effective Length, l_w	151.5	mm
Radius of Gyration, r_w	1.58	mm

Mesh

Mesh Size	SL72	
Wire Diameter, d_m	6.8	mm
Area, A_m	448	mm^2

Capacity Calculations

The number of spans (based on the number of temporary supports) affects the coefficients used when calculating the maximum spans:

$$M^* = jw^*L^2 \rightarrow L = \sqrt{\frac{M^*}{jW^*}}$$

No. of spans	j_1, positive moment	j_2, negative moment
1	0.125	NA
2	0.096	0.125
≥3	0.101	0.121

Top Chord Compression

In accordance with AS 4100 – 1998 Steel Structures, Clause 6.1

$N^* \leq \phi\alpha_c N_s$		
Where Φ	0.9	
Form factor, k_f	1	
Section capacity, $N_s = k_f A_n f_y$	556.1	kN
λ_n	85.57	
α_a	18.77	
α_b	0.50	
λ	94.95	
η	0.27	
ξ	1.07	
α_c	0.58	
Limit state capacity, $\Phi\alpha_c N_s$	288	kN
Truss height, T_h	192	mm
Limit state moment capacity, M^*_{tc}	55.28	kN.m
Maximum span based on moment capacity of top chord in compression, L_{tc}	3.41	m

Top Chord Tension

In accordance with AS 4100 – 1998, Clause 7.1

$N^* \leq \phi A_g f_y$		
Where Φ	0.9	
Limit state capacity, $\Phi A_g f_y$	500	kN
Truss height, T_h	192	mm
Limit state moment capacity, M^*_{tt}	96.09	kN.m
Maximum span based on moment capacity of top chord in tension, L_{tt}	3.95	m

Bottom Chord Compression

In accordance with AS 4100 – 1998, Clause 6.1

$N^* \leq \phi\alpha_c N_s$		
Where Φ	0.9	
Form factor, k_f	1	
Section capacity, $N_s = k_f A_n f_y$	311.7	kN
λ_n	179.6	
α_a	11.05	
α_b	0.50	
λ	185.11	
η	0.56	
ξ	0.68	
α_c	0.20	
Limit state capacity, $\Phi\alpha_c N_s$	56.88	kN
Truss height, T_h	192	mm

Limit state moment capacity, M^*_{bc}	10.92	kN.m
Maximum span based on moment capacity of bottom chord in compression, L_{bc}	1.33	m

Bottom Chord Tension

In accordance with AS 4100 – 1998, Clause 7.1

$$N^* \leq \phi A_g f_y$$

Where Φ	0.9	
Limit state capacity, $\Phi A_g f_y$	281	kN
Truss height, T_h	192	mm
Limit state moment capacity, M^*_{bt}	53.87	kN.m
Maximum span based on moment capacity of top chord in tension, L_{bt}	3.37	m

Diagonal Lacing Compression

Where Φ	0.90	
Form factor, k_f	1.00	
Section capacity, $N_s = k_f A_n f_y$	311.72	kN
λ_n	136.07	
α_a	13.93	
α_b	0.50	
λ	143.03	
η	0.42	
ξ	0.78	
α_c	0.32	
Limit state capacity, $\Phi \alpha_c N_s$	89.21	kN
Maximum span based on shear capacity of diagonal lacing, L_{ds}	5.13	m

The maximum span is calculated from the greatest shear at any point along the slab:

$$V^* = j_3 w^* L \rightarrow L = \frac{V^*}{j_3 w}$$

Where j_3 is a constant that depends on the number of spans.

No. of spans	j_3
1	0.5
2	0.625
≥3	0.6

Concrete Panel Compression

Transformed Section

For serviceability limit state the panel is analysed as an uncracked section using the Transformed Area method to determine the stresses in the steel and concrete.

Steel Elastic Modulus, E_s	200,000	MPa
Concrete Elastic Modulus, E_{cj}	22,610	MPa
Modular Ratio, n	8.85	
Distance from Soffit to Top Chord	221.9	mm
Transformed Top Chord Area	9,838	mm^2
Distance from Soffit to Bottom Chord	29.9	mm
Transformed Bottom Chord Area	4,891	mm^2
Panel Concrete Area	187,500	mm^2
Distance to the Neutral Axis, y_g	46.29	mm
Second Moment of Inertia, I_g	4.07E+08	mm^4

In accordance with AS 3600 – 2009 Concrete Structures, Clause 8.1.3

Concrete strength at loading, f'_c	20	MPa
Concrete strength factor, α_2	0.85	
Compressive area factor, γ	0.85	
Φ	0.60	
Effective cross section depth (equal to panel thickness t_p), d	75	mm
y_g	46.29	mm
k_u	0.62	
Limit state capacity, ΦN_c	1003	kN
Truss height, T_h	192	mm
Limit state moment capacity, M^*_{pc}	192.6	kN.m
Maximum span based on moment capacity of top chord in tension, L_{pc}	5.59	m

Control for Concrete Tensile Strength

AS 3600 – 2009, Clause 3.1.1.3 defines the tensile strength for a given concrete member, this has been used to ensure that the deflection calculations are accurate without using the cracked second moment of area.

$$M^*_{pt} = \frac{f'_{ct.f} I_g}{y_g} \quad \text{where} \quad f'_{ct.f} = 0.6\sqrt{f'_c}$$

$f'_{ct.f}$	2.68	MPa
Limit state moment capacity for concrete strength, M^*_{pt}	23.61	kN.m
Maximum span based on controlling to concrete tensile strength, L_{pt}	2.23	m

Crack Control for Flexure

AS 3600 – 2009, Clause 9.4.1 states the requirements for crack control in flexure to be deemed controlled. Part (c) provides a limit based on the steel stress, which is used to define another maximum span limit.

Largest reinforcement diameter, d_b	6.8	mm
Steel stress limit, σ_{steel}	362.3	MPa
Area of bottom chord steel, A_b	623.4	mm^2
Area of mesh steel, A_m	447.5	mm^2
Total area of bottom steel	1071	mm^2
Equivalent axial force, N*	387.99	kN
Truss height, T_h	192	mm
Limit state moment capacity for steel stress, $M^*_{pf.s}$	74.49	kN.m
Maximum span based on flexural cracking	3.96	m

Deflection

The maximum deflection of the panel can be calculated from either of the following equations:
For an absolute value of a deflection limit,

$$\Delta = \frac{j_4 w_s L^4}{EI} \rightarrow L = \sqrt[4]{\frac{\Delta EI}{j_4 w_s}}$$

Where j_4 is a value that varies based on the number of spans and Δ is the deflection limit.
For a span : deflection ratio limit,

$$\frac{L}{\beta} = \frac{j_4 w_s L^4}{EI} \rightarrow L = \sqrt[3]{\frac{EI}{\beta j_4 w_s}}$$

Where j_4 is a value that varies based on the number of spans and β is the span : deflection ratio limit.

No. of spans	j_4
1	0.0130
2	0.0092
≥3	0.0099

As the precast slab is assumed to be cracked for deflection calculations, only the steel truss members are used for the values of E and I. Any reinforcing mesh is not treated as contributing to deflection control as it is not assumed to be sufficiently attached to the truss frames.

Surface quality class	2	
Maximum deflection (absolute)	3	mm
Maximum deflection (ratio), β	270	
Area of tensile steel, A_{st}	623.4	mm^2
Distance to tensile steel, d_{st}	29.9	mm
Area of compressive steel, A_{sc}	1112.2	mm^2
Distance to compressive steel, d_{sc}	221.9	mm
Neutral axis height for steel truss only, d_{sn}	152.9	mm
Second moment of inertia for steel truss only, I_s	1.47E+07	mm^4

Maximum span based on absolute limit, $L_{def.abs}$	3.05	m
Maximum span, based on ratio limit, $L_{def,rat}$	4.74	m
Maximum span, based on serviceability limits, L_{def}	3.05	m

Maximum Spans Summary

Top chord compression limit	3.41	m
Top chord tension limit	3.95	m
Bottom chord compression limit	1.33	m
Bottom chord tension limit	3.37	m
Diagonal lacing compression limit	5.59	m
Concrete panel compression limit	5.13	m
Concrete tension control limit	2.23	m
Flexural cracking limit	3.96	m
Deflection limit	3.05	m
Governing limit	**1.33**	**m**
Governing limit, ignoring bottom chord compression and flexural cracking	**3.05**	**m**

When choosing the maximum allowable span, the bottom chord compression limit may be ignored in some situations. The bottom chord compression limit can be ignored if the bottom chord is completely embedded in the panel concrete for its full length. This is because the concrete panel is assumed to provide sufficient restraint to prevent buckling.

When choosing the maximum allowable span, the concrete tension control limit may be ignored. This limit may be ignored as it is based on the uniaxial tensile strength of the concrete panel, as opposed to the flexural tensile strength. As the precast panel is not typically going to be axially loaded, this is not relevant.

The horizontal loading of the formwork (AS 3610 – 1995, Clause 4.4.5) must also be provided for. This will typically be a role of the edge form designer.

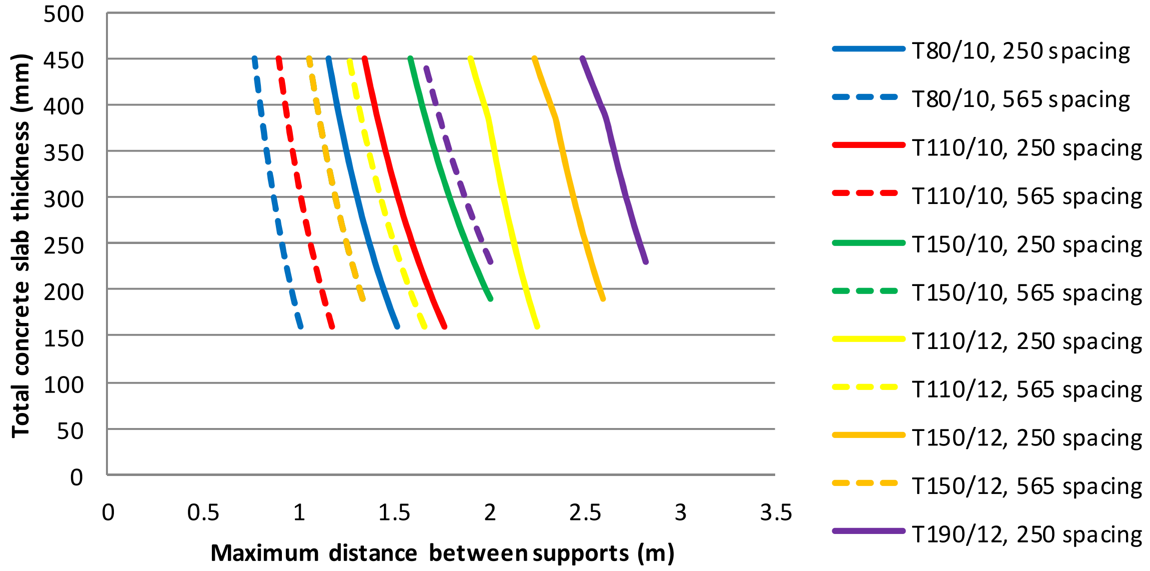
austral precast

precast. style solution

Appendix A[1]

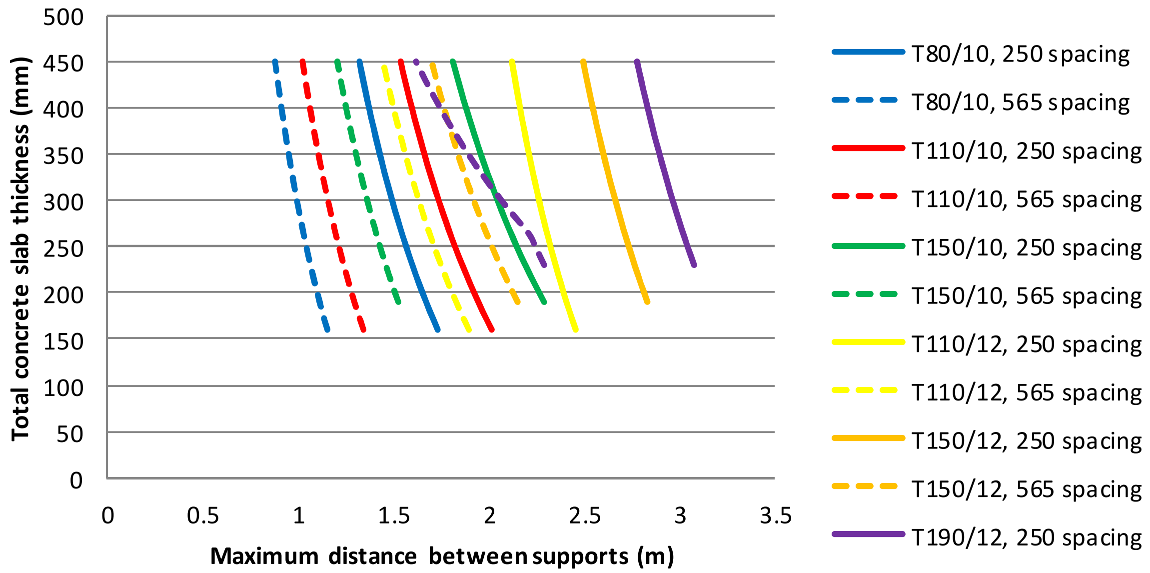
Single span

[variables: 75mm precast, f_{cmi} 20MPa, SL72 mesh, 20 cover, Class 2 finish]



Two span

[variables: 75mm precast, f_{cmi} 20MPa, SL72 mesh, 20 cover, Class 2 finish]



1. The information in the graphs are indicative, should be used as a guide only and does not replace the need for qualified structural design engineer.
2. The information in the graphs has been generated in accordance with Australian Standard™. Formwork for concrete. AS 3610 AS 3610-1995 Formwork for Concrete and AS3610.1-2010 Formwork for concrete – Part 1 Documentation and surface finish

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