DESIGN FOR EDGE LIFTING OF PRECAST CONCRETE PANELS

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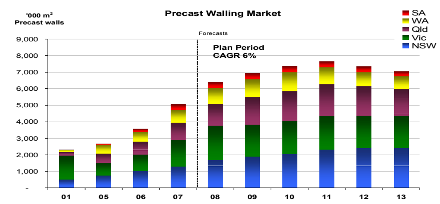
Synopsis:
The rapidly increasing market for precast wall panels demands better prediction tools for the safe and efficient design of the edge lifting anchors required to lift, handle and erect them safely.
Edge lifting anchors are loaded in shear when tilting panels after casting and in tension for transport and erection. In Australia the most popular edge lifting anchors are made from steel plate in a "hairpin" shape and the published design prediction equations for headed anchors may not always be valid. None of the equations predict shear performance of concrete lifted with hairpin anchors. This paper reviews the limit states for edge lift anchors, the statutory requirements, modes of anchor failure and the most popular design equations. A new model for the prediction of tension and shear capacity of thin concrete panels lifted with haripin anchors is presented together with test results. The limit states for hanger reinforcing bars required for the majority of edge lifting anchors are discussed and test results which verify the controlling limit states for design of popular hairpin anchors. The in-effectiveness of unconfined panel reinforcing in improving the ultimate concrete limit state is discussed with reference to a recent failure.

Keywords: Lifting anchor, hairpin, edge-lift, factory-precast, tilt-up, hanger-bar, limit state, prediction

Research Significance

The Australian market for precast concrete wall panels is currently estimated to be 5-6 million square metres per annum which represents about one third of the walling market.



Precast concrete walling is displacing traditional methods of construction. There has been strong growth in precast walling in recent years, having more than trebled since 2000[1].
Off-site precasting in factories is displacing site cast methods and other forms of construction and the number of panels to be lifted transported and erected is growing strongly. Maximum efficiency demands large panel sizes but transport limits the practical size of panels to about 3metres x 10metres and 20 tonnes mass.
The thickness of wall panels is normally controlled by the requirements for fire rating, typically 100-200mm thick with a majority 150mm thick. Panel cost is minimised by using only simple shrinkage reinforcing.
In Australia these large "thin" panels are predominantly cast horizontally and tilted up about one edge into the vertical or near vertical position for erection. Edge lifting facilitates handling, storage and erection but lifting stresses can limit panel dimensions or increase the need for expensive reinforcing.
The number and distribution of lifting points must be carefully designed to minimise panel stresses and provide safe lifting and handing in all situation in accordance with code requirements.
There has been no published research for predicting concrete failure for the types of anchors used in Australia. There is a growing need for consistent design methods for the safe and efficient application of concrete lifting anchors, critical to the success of every precast concrete construction project.

Panel manufacturing method and panel stresses

Two methods of manufacturing concrete wall panels are used in Australia:

Site cast:

> Popularly known as "tilt-up" construction, panels are cast on-site usually onto the floor of the building or adjacent casting beds. These panels are normally face lifted to facilitate stress design for large and complex panels.

Factory precast:

> Factory manufacturing concrete panels brings many benefits[2] compared to site casting; all weather production, improved efficiency of labour, better training and labour stability, higher production speed and efficiency, better planning, supervision and occupational health and safety outcomes to name only a few. The downside is that panels are limited in size by transport restrictions and the efficiency in lifting, transporting and erectin large numbers of panels becomes crucial to success.

The “typical” factory precast wall panel of 2-3m x 5-10m x 150mm has a mass of 5-10 tonnes and for the most part these are small enough to be edge lifted with anchors placed along the long edges of the panel. Edge lifting optimises handling storing and transport and facilitates erection because the edge located anchors allow the panels to hang perfectly vertical.
The critical time of lift is rotation from the horizontal position to the vertical from the casting bed, when the concrete strength is low, with a compressive strength typically 10MPa.

Panel Lifting Stress considerations

The bending strength of the panel between the lifted edge and the edge of rotation limits the span and hence the “height” (width) of the panel. There is no such limitation on the long edge because the bending and shear forces can be controlled by the number and spacing of lifting points and so panel length is restricted by handling and transport considerations.
Stresses are minimised in large panels by lifting with anchors placed in rows in the top face. The height can be increased by adding more anchor rows. The distribution of anchors in the face increases design flexibility to enable complex panels. Australian developed computer design programs using finite element analysis[3] assist the design of complex panels
Bending stresses may be largely eliminated by using tilting tables to rotate the entire casting bed and panel into the vertical or near-vertical position. The panel is then lifted and handled in the vertical plane by edge-located anchors. The installation of tilting tables has a high capital cost and for most Australian factories, if installed at all, tilting tables are limited in number and reserved for lifting only difficult panels with cut-outs and complex shapes and dimensions.

The complex bending and shear forces developed by concentrated anchor loads in thin panel edges limit the size of panel which can be effectively tilted without cracking.

Statutory requirements for limit state design of concrete lifting anchors

The statutory design requirements for concrete lifting anchors are provided by AS3850:2003[15] and National Code of Practice:2008[16]. Anchors are required to have a minimum limit state factor (LSF) of 2.5 between the Working Load Limit (WLL) and the ultimate strength capacity limit, *for all limit states.*

$$WLL \quad = \Phi \times R_u / 2.5$$

There are two ways an anchor can fail. Some part of the anchor itself could break or the concrete could fail around the anchor allowing it to pull out of the concrete:

Anchor Strength Limit State:
failure of some part of the anchor itself or other elements on which it depends for anchorage
Concrete Strength Limit State:
shear pullout, cone failure, bursting or splitting failure allowing the anchor to pull out.

In determining the WLL of an anchor both conditions must be investigated. The Design limit state used to calculate the WLL is the lesser of the two limit states. e.g. if the anchor is stronger than the concrete then the concrete strength is the controlling limit state and vice versa.
When long anchors are embedded in high strength concrete, well away from edges, the concrete is stronger than the anchor and the anchor strength limits the design.
On the other hand, when anchors are embedded in thin panels, the anchor is stronger than the concrete, (especially concrete less than 15MPa) and so concrete failure limits design.
For a “well embedded anchor”, the concrete failure load is dependent on:

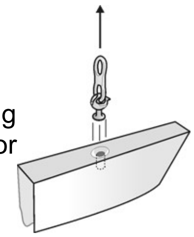
compressive strength of the concrete
distance from the anchor to any edge or face
depth of embedment of the anchor in the concrete

A “well embedded anchor” is one which connects intimately with the concrete and can fully transfer all of the load to the concrete over its full length, which defines its embedment depth from the surface. Load transfer of well embedded anchors is achieved by mechanical interlock between features on the body of the anchor e.g. a headed “foot” or projecting deformations or shapes.

Concrete cone failure

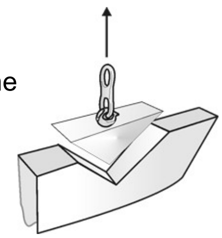
If the anchor is well embedded, it can be thought of as being “glued” to the concrete and so the concrete does not “know” or “care” about “what” is doing the pulling but only feels it is being pulled and it resists that pull! When the load causes concrete failure, it does so by a “shear cone” being pulled from the concrete with a depth equal to the embedment depth of the anchor.

Types of concrete pullout failure



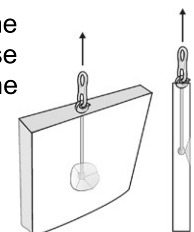
Direct Shear

Anchors of shallow embedment and low concrete strengths or high bearing stresses resulting from too low a ratio between the interlock feature and the anchor body. Failure is initiated by concrete crushing and then pullout of a shallow cone.



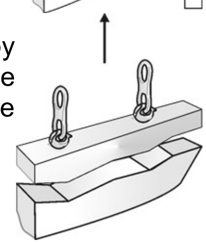
Pie failure in thin panels

In thin panels where the edge distances to the faces are small, the shape of the shear cone is changed to that of a "pie" shape.



Bursting (side blowout)

When long anchors are embedded in thin panels and where the initiation of the shear cone is from a headed "foot"at the base of the anchor, the transverse (bursting) stresses around the foot exceed the tensile bearing capacity of the concrete between the foot and the free surface, leading to bursting failure.



Panel splitting

When the panel splitting strength is less than the concrete strength for failure by other modes the panel splits to the surface and then along to the edges. Multiple anchors can fail in this way where the failure surfaces link up and strip of concrete containing the anchors is stripped from the panel.

Hanger bars and their design

Where the concrete strength is too low to support the desired WLL of the anchor a supplementary reinforcing bar "hanger bar" is fitted. This is a "V" shaped bar which effectively increases the embedment of the anchor and sheds the loads deep into the panel.

Hairpin edge lift anchors used in Australia for thin panels always need a hanger bar to achieve their full nominal WLL for low concrete strength (< 15MPa).

These types of anchors are also referred to as "eye anchors" because of the transverse hole in the body of the anchor through which the hanger reinforcing bar is attached. AS3850:2003 requires LSF=2.5 for failure by any strength limit state of the anchor system (anchor+hanger bar).

Hanger bar strength limit states:

Hanger bar tension strength limit state

The strength must be sufficient to ensure that the WLL based on failure strength exceeds the required WLL of the anchor they support. Since the load is shared by two legs:

$WLL = 2.\Phi.R_u / 2.5$ AS3850

where $R_u = A_b x 1.08 \times f_{sy}$ .... N class bar, AS4671[17]

The appropriate bar diameter is selected for the required WLL shown in table 1 below.

Hanger d_b	Area A_b	Total Area	R_u Design tensile strength capacity (for 2 legs)	WLL TENSION AS3850 $\Phi * R_u$ / 2.5
Rebar	Area	2 x A_b	kN	kN
N12	113	226	122	39
N16	201	402	217	69.4
N20	314	628	339	108.5

Table 1: Strength of N class bars and WLL calculated in accordance with AS8850

Hanger bar shear strength limit state.

The reinforcing bar may fails by shearing where it passes through the anchor hole at a load lower than tension failure of the legs shown in Table 1. This cannot be calculated and is determined from tests meeting the requirements of AS3850. App. 2.

Hanger bar concrete-pullout strength limit state

The development length L_{syt} of each leg of the bar is calculated from AS3600 Clause 13.1.2.1.

Hanger Leg Length $L_{syt} = k_1\, k_1\, f_{sy}\, A_b\ /\ (2{*}a + d_b)\ \sqrt{f'_c}$ and $L_{sy.t} \geq 25k_1 d_b$

Hanger bar pull-through limit state

Failure of some part of the anchor body where the hanger bar is supported. The design and strength of the anchor must be sufficient to fully transfer the load to the hanger bar.

Some anchors available in Australia do not meet this requirement, allowing the bar to be stripped from the anchor at loads less than the failure load of the bar.

Prediction of the Concrete Limit State of anchors in concrete

The behaviour of round bodied headed anchors cast into faces of concrete elements has been extensively reported [4,5] with the development of prediction equations for round bodied anchors of varying embedment depth, loading direction, concrete compressive strength and proximity to edges [4,6].

The "CCD method"[6]

First proposed by Fuchs[6] and published by ACI[4] after tests of cast-in headed anchors. It is useful for its design simplicity[10]. The method assumes a pyramidal failure cone with a base length of three times the embedment depth and is the basis for current precast concrete anchoring design in Australia[11].

For cast-in headed anchors loaded in tension, the CCD cone failure equation is:

$$N_c = k_{nc}\ .\ h^{1.5}.\ f'^{0.5}_c \qquad k_{nc} = 15.5$$

and for shear of a fastener of diameter d, loaded at distance c toward a free edge:

$$V_c = (l/d)^{0.2}\ .\ c^{1.5}.\ F'^{0.5}_c \qquad l = \text{"activated bearing length"} - h \leq 8d.$$

These predictions are good for anchors with embedments within the range of normal fastenings typically less than 100mm, but are overly conservative for deeply embedded anchors[4,6,10] where the ACI-349.85[12] predictions, proportional to h^2 are reported to provide better data fit [6,9,10]. Eligehausen[4] and others developed equations proportional to $f'^{0.67}_c$ based on a 30degree shear conical cone.

"Haeussler"method

A prediction equation for lifting anchors, proportional to h^2 and $f'^{0.67}_c$, with 30° shear cones was developed in Germany by Dr. Ernst Haeussler for anchors with embedments larger than 120mm. Although not widely published this has been the "standard" design method for spherical headed lifting anchors which he developed in 1967 and now the most commonly used concrete lifting system in Australia and world-wide.

$$N_c = k_c\ .\ h^2.\ F'^{0.67}_c \qquad k_c = 9.72\text{x}10^{-1}$$

Edge reduction factors where anchors are placed closer than 3h to an edge

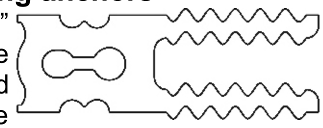
When an anchor is placed closed to an edge the shear cone is truncated. The CCD method modifies th basic prediction equation for a full cone failure by a ratio of projected areas at the concrete surface.

A simple reduction factor for a full cone of radius $3h$ and angle ≈ 30° may be calculated using the trigonometric reduction from the conic surface proportional to the hypotenuse of a triangle with sides h and $3h$. By Pythagoras' theorem the length of the conic surface varies by the square root of the two sides of the triangle, so the ratio of the reduced area of an anchor c from the edge is:

Reduced Conic area □ □ sin (30. c/h) when $c = 3h$ this reduces to sin90° = 1

$$N_{-reduced} = N_c\ .\ \square \sin(30.\ c/h)$$

Limitations of current prediction equations for Australian edge lifting anchors



Most edge lifting anchors are not round bodied but of a generally "hairpin" shape cut from steel plate, of length 200-400mm, with wavy or saw-tooth like legs which extend into the concrete for anchorage. They are centrally located with edge distances *c* in the range .20h - .5h. The rectangular head of the anchor has high bending resistance when shear loaded toward the panel edge. It has a hole to which the lifting device is attached and may or may not have a separate hole or keyhole for hanger attachment.

The CCD method is not suitable for embedment over 200mm. The Haeussler equation is valid but based on headed anchors and assumes a constant stress over a generally conic area where failure initiates from the anchor "foot" at low loads, propagating until about 90% of the bearing load is reached (dependent on foot shapet, concrete strengh and embedment depth). The maximum load is reached when the crack extends to about 50% of the conic surface[10].

What is required is a simple prediction model for "hairpin" anchors for the prediction of the concrete capacity according to panel geometry, anchor dimensions, embedment and concrete compressive strength at the time of lift, using the concrete tensile, flexural and compressive strengths.

Proposed concrete strength prediction model for hairpin edge anchors in thin panels

Observations from previous testing showed that the nature of the concrete failure in thin panels was different from the failure of isolated anchors placed well away from the edges in massive panels. Bending of the thin panel caused tensile splitting. Two cracking events precede final failure – initial splitting crack/s followed by propagation and then a final failure crack. In some cases, the pie shape assumed to occur for shear cone theories did not occur. The panel split normal to the anchor axis. This failure was less strongly influenced by the embedment depth factors ($h^{1.5}$, h^2) of the prediction equations.

Proposed prediction model for hairpin anchors

Tension

The first observable crack is a flexural/tensile splitting crack from the free edge and running along the embedded length of the anchor. The crack extends down from the panel surface to the side of the anchor. Tensile forces applied to the anchor result in compressive stresses in the concrete transferred by the deformations in the anchor through concrete interlock. Compressive forces are resolved at the surface of a thin panel by tensile splitting forces normal to the anchor axis. Additionally, the anchor load locally "bends" the concrete about its axis toward the edge. The weakest section is from the edge of the anchor to the panel surface normal to the axis of the anchor. The highest stress felt by the concrete is at the anchor, at the point of maximum moment as it bends in the loading direction. This crack occurs on both sides of the anchor and propagates to the full length of the anchor with increasing load.

$$N_{1st} = f'_{cf} * A_t$$

where

At = Area of cracked section between anchor sides and panel surface

f'_{cf} = $0.6 \sqrt{f'_c}$ ...Cl 6.1.1.2 AS3600[13]

The second and final crack initiates from the first when it reaches the base of the anchor. The crack then propagates laterally if the anchor is close to an edge or more generally back toward the surface describing a "pie" shaped section. The crack is assumed to propagate by plastic flexural failure because of the bending stresses induced by the anchor over an assumed 45° failure surface.

$$N_f = f'_p * A_f$$

where

A_p = Wedge shaped area of failure crack

f'_p = $0.26 \sqrt{f'_c}$ (fifth percentile of plastic flexural strength tests[14])

The fracture surface area varies from the anchor width at its foot, to the panel thickness at the panel edge.

Shear toward an upper face

The first observable crack results from a flexural tensile stress in the panel surface above the anchor as the panel bends over the anchor. This splitting crack initiates from the free edge and propagates along the length of the anchor and beyond. This model assumes the section resisting failure is between the side of the anchor and the (upper) panel surface.

$$A_s = 0.5 * L * (d - W)$$

$$V_{1st} = f'_{cf} * A_s$$

The triangular wedge shaped crack extends from the foot of the anchor to the edge and to to lowest upper facing plane of the anchor defining the embedment (e) from the top surface of the panel.

$$A_f = e * L \sqrt{2}$$

$$\text{Final Failure load} = f'_p * A_f$$

Testing Program

Concrete Testing

The main purpose was to determine whether the proposed model was valid. A second purpose was to examine whether the mode and morphology of the failure and the concrete strength limit state of the anchor was influenced by the different shapes of anchors available in the market.

An anchor supplier had claimed that a new type of hairpin anchor with hook shaped deformations on the inside faces of the legs and straight outer faces vastly improved performance, in particular tensile performance, generated no bursting forces and that when used with a hanger bar this bar could be lighter (smaller in diameter) than required for other hairpin anchors designed to AS3850. It was claimed that these anchors dramatically increase the tensile capacity of the system in the presence of typical mesh reinforcement (unconfined shrinkage crack control steel: central mesh and horizontal edge bar).

Concrete test method

150mm thick slabs with central shrinkage SL82 (100x200x8mmdia) control mesh were cast with concrete designed to provide 7 day strength of 25MPa. The actual concrete strength was tested by the concrete supplier on the day of the pour and verified independently with samples taken from the initial and final pour to check the consistency of the concrete over the pour. The concrete was cured under plastic.

The anchors were installed according to the manufacturer's recommendations with and without hanger bars and edge shrinkage reinforcing as specified. The load was applied using a hydraulic jack and digital pressure transducer monitored by a data logger and attached computer recorder.

Careful observation of the specimens was compared to the recorded data which provided visual and data confirmation of the loads at which first and subsequent cracking occurred, indicated by a drop in load, the type and nature of the cracking and the peak load reached. The strength limit state capacity ΦR_u for both initial cracking and ultimate failure was calculated in accordance with requirements of AS3850 App.A4 .

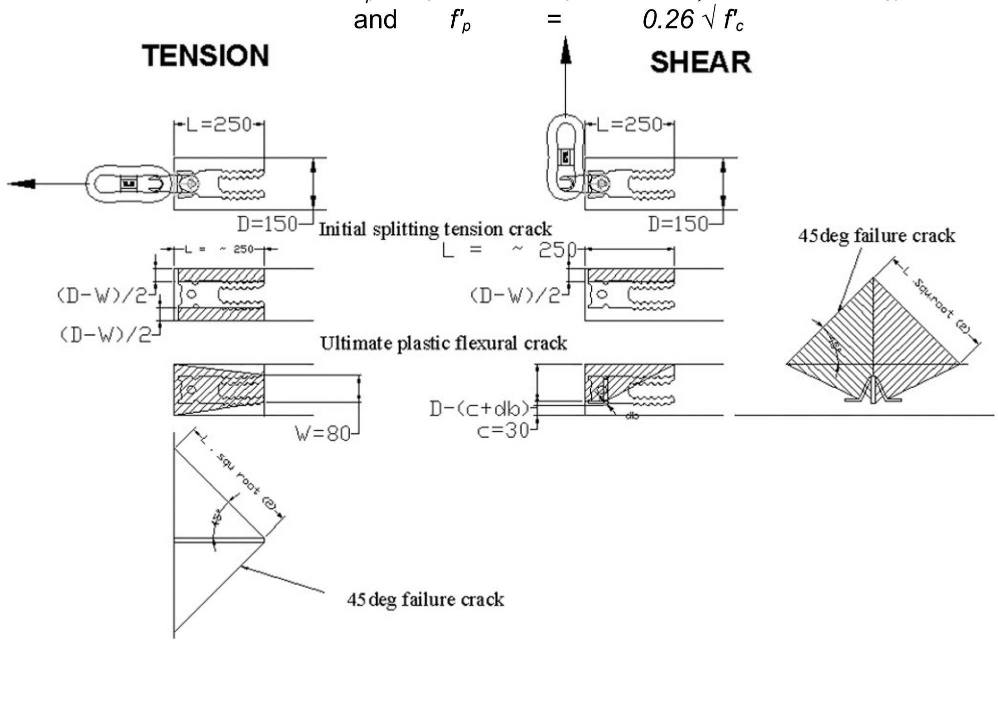
Results

The test values were compared to calculated values from the proposed model for slab thickness D=150, a central anchor length L=250mm, width W=80, cover c=30, shear bar diameter db =12

$$\text{Ultimate Tension failure} = f'_p * A_t \text{ where } A_t = 2 * (W + (D-W)/2) * L * \sqrt{2}$$

$$\text{Ultimate Shear failure} = f'_p * A_s \text{ where } A_s = e * L \sqrt{2},\ e = D - 30 - d_{sb}$$

$$\text{and } f'_p = 0.26 \sqrt{f'_c}$$



Test	Anchor	Length	Additional reinforcing	First crack kN	Ultimate kN
	A	275	N16 edge	49.7	102.4
2-2	B	250	N16 x700 hanger, N16 edge	121.3	148.0*
2-5	C	250	nil	118.9	118.9
3-2	B	250	Nil	82.3	93.9
3-3	C	250	N20x700 hanger	118.2	134.4*
3-4	D	360	N20x700 hanger	109.9	137.5*
Average of all tests				100	122.5
Average of anchors without hanger				83.6	105.1
ΦR_u				64.3	80.8
WLL = ΦR_u / 2.5					32.3

Table 2: Concrete failure for edgelift anchors in tension

NB: anchors with hanger bars continued to take higher loads opening the crack until bar or panel failure.

Test	e	First crack kN	Ultimate kN
Panel 1-8	115	32.3	47.4
Panel 2-1	104	23.3	46.3
Panel 3-1	108	10	53.7
Average value		21.9	49.1
ΦR_u		16.8	37.8
WLL = ΦR_u / 2.5			15.1

Table 3: Concrete failure for edgelift anchors in shear

NB: **e** is embedded depth from panel surface of the upwardly facing anchorage surface (inside of lower leg or top of shear bar)

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		Failure area mm^2	Test kN	Calculated kN	Test/Calc
TENSION	1st splitting crack - no hanger	17500	83.6	52.5	1.59
	all results		100		1.9
	Final Failure crack - no hanger	81317	105.1	105.7	0.99
	all results		122.5		1.16
SHEAR	1st axial splitting crack	8750	21.9	26.3	0.83
	Final Failure crack - pie chunk	38184	49.1	49.6	0.99

Table 4: Summary of concrete testing for anchors loaded in shear and tension

Discussion

The data indicates that the presence of a hanger bar tends to increase the load at which the concrete cracks over and above the load achieved by anchors without hanger bars. It is assumed that this results from pinning by the bar across the crack which assists concrete aggregate interlock preventing crack propagation until a higher load is reached after which the crack widens and interlock is lost.

The strength limit state capacity of the concrete and WLL shown in the table above were therefore based on the test results without hanger bars and are therefore conservative for anchors with hanger bars.

First cracking load

The model generally overestimated the loads for the shear failure and underestimated tensile loads. The visual detection of the cracks was difficult which affected the load correlation.
In practice this crack is not of significant interest other than to determine whether or not crack control steel is required over the top of the anchor and parallel to the edge.

Ultimate failure load

The final failure crack was much more easily detected both visually and from the reduction in peak load. There was good correlation between the model and test values. The values in the table show only the concrete failure limit and do not show the ultimate failure loads for anchors fitted with hanger bars which continued to accept higher loads beyond concrete cracking failure from the bottom of the anchor.

Anchor shape effect

There was no difference between the performance or mode of failure of any of the anchors tested. Anchor A was of a type claimed to have vastly improved tensile performance compared to other types however this was not seen in the results. All anchors failed in exactly the same manner by the same cracking mechanisms. There was no evidence of bursting failure provoked by any of the anchors.

Anchor length

The difference in anchor lengths did not have a marked effect on ultimate strengths achieved, confirming the postulation that predictions proportional to either $h^{1.5}$ *or* h^2 used by CCD and other equations is not valid for such thin panels.

Comparison of proposed unifying method to semi-empirical predictions

The method provides good agreement for both tension and shear to the test values and would appear to provide the basis for a unifying method.
The tested strength limit state capacity was compared to the predicted strength of a conic failure surface and headed anchor using equation

$$\Phi R_u \text{ Test} = 80.8 \quad \Phi R_u \text{ Calc} = 81.3 \quad N_{\text{-reduced}} = 82.1\text{kN}$$

These are all in good agreement and so the proposed method correlates to the semi-empirical Haeussler method for cone pullout when applied to anchors loaded in tension in thin panels.

Hanger bar testing

The purpose was to test whether hanger bars used with hairpin anchors failed in tension or shear at the hole and hence determine the appropriate design limit state.

Hanger bar test method

Three commercially available hairpin types were fitted with N class hanger bars manufactured to AS4671 and load was applied to the anchor with a standard lifting clutch and the hanger bars were gripped in wedge grips of a Universal Tensile Testing machine. The ultimate load, type and mode of failure were recorded and WLL calculated according to AS3850:2003 Appendix 4.

Results

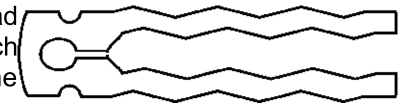
Anchor	Ref	Bar	KN failure	Claimed Max WLL	Claimed WLL*2.5	Failure
AA	# 06-055	N16	183.75	98	245	reinforcing bar
	# 06-057	N16	208.64			bottom bridge of anchor
B	#6-14mm	N16	246.84			top bridge of anchor
	#7-14mm	N16	240.59			reinforcing bar - tension
	# 8-14mm	N16	231.42	69.4	173.5	reinforcing bar - tension
	# 9-14mm	N16	232.57			reinforcing bar - tension
	# 10-14mm	N16	233.39			reinforcing bar - tension
ΦR_u			187.8	Allowable Design Tension AS3850:2003 **69.4**		
WLL = ΦR_u / 2.5			**75.1**			
B	#6-16mm	N20	286.87			top bridge
	#7-16mm	N20	280.91			reinforcing bar - shear
	# 8-16mm	N20	281.17	93.1	232.8	reinforcing bar - shear
	# 9-16mm	N20	298.85			reinforcing bar - shear
	#9-130-13	N20	265.11			reinforcing bar - shear
	#9-130-22	N20	262.13			reinforcing bar - shear
ΦR_u			232.6	Allowable Design Tension AS3850:2003 **108.5**		
WLL = ΦR_u / 2.5			**93.1**			
C	#4-16mm	N20	113.04			bar pulled through the gap
	#5-16mm	N20	122.28	88.2	220.5	bar pulled through the gap
	#6-16mm	N20	113.61			bar pulled through the gap

Table 5: Hanger bar strength tests and working load limits

Discussion

N16 bars failed in tension in the straight part of the bar at loads exceeding the design load calculated according to AS3850:2003.
N20 bars failed by shear at the hole in the anchor at loads less than the design load.
The WLL of N20 is limited by the tested shear failure strength, to WLL= 93.1kN, shown in table 5.

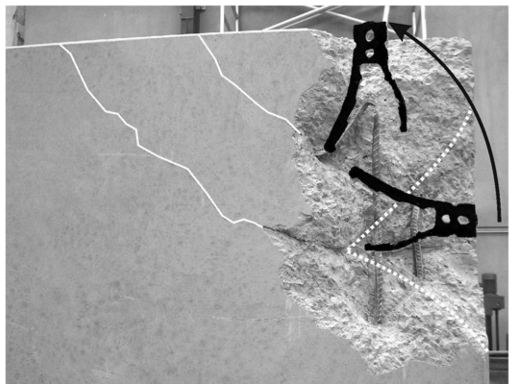
Pull through failure



Supplier C anchors have a split bottom bridge which supports the load and not surprisingly the hanger bar pulled through the gap at loads much less than the design load. This design is not effective in improving the WLL of an anchor above the concrete strength capacity.

Undersized hanger bar

Supplier of anchor AA claimed a WLL exceeding the allowable WLL of the N16bar on which it depends.
A recent failure of this type of anchor designed for a WLL of 98kN demonstrates that these claims cannot be justified. After the concrete cracked and spalled, the anchor rotated as the panel fell and the anchor pulled out after the tensile failure of the N16 bar to which it was attached.



This photo of the failure shows the initial (horizontal) position of the anchor and its final position superimposed over the failed surface. The initial shear crack is identified by a broken white line and the final shear cracks extending from the hanger bar have been mapped with firm white lines.

The N16 hanger bar provided anchorage after failure of the concrete but the anchor system (anchor+bar) did not have sufficient capacity to provide the required WLL with the statutory LSF of 2.5 (98x2.5=245). The system was limited by the N16 bar with only a WLL of 69.4kN.

Panel shrinkage reinforcing ineffective to increase failure load

It was claimed that the shape of the anchor increased the WLL in concrete and that the normal panel (shrinkage) reinforcing increases the capacity of these anchors and improves working load.
In the photo it can be seen that all of the reinforcing has been exposed because it is unconfined and spalls away. The panel did not offer any assistance to increasing the ultimate strength capacity.

Panel reinforcing cannot increase the ultimate failure capacity of the concrete unless it is fully confined and then only if it has been specifically designed to meet the requirements of AS3850 in transferring the full load (WLL*2.5) to the panel without failure.

Conclusions

The performance of hairpin style edge lift anchors, extensively used in Australia can be predicted from the proposed model by calculating the failure area with anchor and panel geometry and applying strength characteristics of the concrete derived from its compressive strength at the time of lift.
The results correlate to a semi-empirical tensile prediction equation used for headed anchors.
No performance or other benefit was evident for different shaped despite claims to the contrary.
Panel shrinkage reinforcing, being unconfined offers no improvement to the concrete failure strength.
All possible failure states must be examined for the proper design of hanger bars including tensile failure, shear failure and pull-through failure otherwise the hanger bars are ineffective in raising the strength of the anchor above the concrete strength limit state.
Further testing of various thickness panels and low concrete strengths typical of when panels are first lifted from the casting bed would provide better tuning for the model.
Finite element analysis may help understand the effects of the complex bending/shear/tensile forces and tri-axial states of stress generated in the concrete around lifting anchors.

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