Unanticipated Bond Failure over Supporting Band Beams in Grouted Post-Tensioned Slab Tendons with Little or No Initial Prestress

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Abstract: After grouting the ducts in a large band beam and one-way slab post-tensioned floor system in a large shopping mall (in excess of 1.5 km² in plan), doubts arose concerning the level of initial prestress and the strength and serviceability of the slabs was questioned. To assess the strength of the slabs, it was assumed, as a worst case scenario, that some spans may have been constructed with zero initial prestress. Load tests on prototype specimens, with similar dimensions and reinforcement quantities as the floor slabs, were performed to assess the stress development in initially unstressed, but fully grouted, tendons. Unexpected bond failures occurred at the junctions of slab and band beam where the abrupt change in section depth caused a sudden change in tendon stress. The maximum stress that was developed in the initially unstressed tendons at these locations was about 60% of the strength of the tendons.

Key words: Anchorage, development length, post-tensioned slabs, stress development, tendons, unbonded construction.

1. Introduction

Problems were encountered recently with the post-tensioning installation in the floor slabs of a large shopping mall constructed in the Middle East. The floors consisted of over 1.5 million square meters of continuous, post-tensioned, one-way slabs supported on transversely spanning band beams. Soon after construction, doubts arose concerning the level of prestress in the slabs and questions were asked concerning the strength and serviceability of the slabs in regions where the level of initial prestress may have been less than that specified. For the purpose of assessing the strength of the slabs, it was assumed, as a worst case scenario, that some spans may have been constructed with zero prestress in the tendons.

Two series of load tests were carried out on site. Eight full-scale concrete slab specimens (2.0 m wide and 6.0 m long) reinforced with a combination of

conventional deformed reinforcing bars and grouted strands were tested. The specimen dimensions, concrete properties, reinforcement types, reinforcement quantities and locations, duct type, tendon type and tendon profile were similar to (or where possible, identical to) the as-built floor slabs.

In each specimen, the tendons consisted of two post-tensioning ducts, each containing four 12.7 mm diameter strands. The ducts were fully grouted 28 days prior to testing. The initial prestressing force in each strand was zero, i.e., the bonded strand was initially unstressed and thus acted as non-prestressed reinforcement throughout the test.

Each specimen was subjected to increasing deformation to failure and the load-deflection response was measured electronically from first loading through to the peak load and then beyond into the post-peak unloading region. Slip of the strand at the end of each specimen was also measured. Tests were also conducted on companion specimens to determine relevant material properties.

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The purpose of the tests was to assess the tensile stress that develops in the initially unstressed post-tensioning strand at the critical section at peak loads. As the strands were initially unstressed, the stress that developed in the strand under increasing load was entirely due to bond via the concrete-duct, duct-grout and grout-strand interfaces.

2. Background

In a situation where the initial prestressing force is zero, the maximum stress that can be developed in a prestressing strand grouted inside a post-tensioning duct depends on:

- (1) the development length available for the strand at the section under consideration;
- (2) the quality of bond between the strand and the grout;
- (3) the slip that might take place between the duct and either the grout inside the duct or the concrete outside the duct;
- (4) the deformation that develops at the level of the strand due to the imposed actions.

The deformation referred to in four point can be determined from first principles based on strain compatibility and the requirements of equilibrium.

The Australian Standard for Concrete Structures [1], specifies that the minimum development length for untensioned 12.7 mm diameter strand is $\ell_d = 150 d_b = 1,900$ mm. Based on tests results [2-4], ACI 318-08 [5] specifies that the minimum embedment length required to develop a stress of f_{ps} on each side of the critical section in a pretensioned member may be taken as:

$$\ell_d = \left(\frac{f_{se}}{21} \right) d_b + \left(\frac{(f_{ps} - f_{se})}{7} \right) d_b \quad (1)$$

where, f_{se} is the effective prestress in the strand (in MPa), f_{ps} is the stress in the strand when the nominal strength is reached (MPa), and d_b is the nominal diameter of the strand. In the case of strand with zero initial prestress ($f_{se} = 0$), the first term in Eq. (1) disappears and the development length is the flexural

bond length given by $\ell_d = (f_{ps}/7) d_b$. Clearly, Eq. (1) was never intended to be applied to a situation where the effective prestress is zero [6, 7]. It does not account for many of the factors that affect the development length, including the concrete quality, the cover, the spacing between wires, the concrete confinement and so on. Little guidance is available for estimating or checking the development length of a strand in a grouted duct. For unstressed, deformed wire in normal weight concrete, ACI 318-08 [5] suggests that the development length may be taken as:

$$\ell_d = \left(\frac{f_y}{1.4\sqrt{f'_c}} \right) d_b \quad (2)$$

Although not strictly applicable for unstressed 12.7 mm diameter strand with $f_y = 1,800$ MPa in concrete with $f'_c = 60$ MPa, Eq. (2) gives $\ell_d = 2,100$ mm.

If the specified tensile yield strength f_{py} is to be developed in an initially untensioned strand at any cross-section, the imposed actions must cause a change in strain in the steel sufficient to generate f_{py} and the steel strand must be anchored on either side of the cross-section by a length in excess of the development length. The embedment lengths of the strands in the actual floor slabs in the Middle East shopping mall on either side of the critical positive moment region at mid-span and the critical negative moment regions at the interior supports were in excess of the requirements of both AS3600-2009 and ACI-318-08. However, because of the uncertain nature of the bond conditions in the ducts in these slabs, it was decided to explore this aspect more thoroughly by testing.

3. Cross-Sectional Analysis

3.1 Series 1

In the first test series, three one-way slab specimens (S1 to S3) were constructed and tested. The specimens contained the same non-prestressed reinforcement and the same prestressing steel and ducting as was used in the as-built floors and the same concrete was specified.

The grout materials and mix proportions were also similar to that used in the slabs in the shopping mall. Each specimen was subjected to the same curing regime as was applied to the actual floors and all were tested on the same day—28 days after grouting of the ducts.

Specimens S1 and S2 were tested in positive bending over a simply-supported span of 5.6 m, as shown in Fig. 1a. The specimens were prismatic and of rectangular section.

S1 was 2,000 mm wide and, at 250 mm deep, it had the same thickness as a typical exterior span of the actual slabs in the shopping mall. It contained the same conventional reinforcement as the actual slabs and also contained a similar prestressing strand layout. Specimen S2 was 2,000 mm wide, 200 mm thick, and it was the same thickness and contained the same conventional reinforcement and prestressing strand layout as the positive moment region of a typical interior span. Each specimen contained two fully-grouted

ducts, with four 12.7 mm diameter prestressing strands in each duct all with zero initial prestress. An elevation of slabs S1 and S2 and the loading arrangement are shown in Fig. 1, together with the cross-section of each specimen at mid-span.

To test behavior of the slabs in the negative moment regions over the band beams, an additional specimen S3 was tested in Series 1. Specimen S3 was 6 m long, 2,000 mm wide and had a depth of 200 mm at each end of the specimen and 450 mm in the central 2.1 m of the specimen. S3 contained the same conventional reinforcement and prestressing strand layout as the negative moment region of a typical interior support region of the shopping mall slabs. Details of specimen S3 and the loading arrangement are shown in Fig. 2. The specimen contained two grouted ducts, with each duct containing four 12.7 mm diameter strands. The level of prestress in all strands was zero.

3.2 Series 2

The results for specimen S3 (in Series 1) indicated that local bond failure occurred at the junction of the slab and the band beam where there is a relatively sudden change in the section depth and hence a sudden

Fig. 1 Elevation and cross-section of specimens S1 and S2.

Fig. 2 Elevation and cross-section of specimens S3.

change in the tendon stress. At the critical section A-A in Fig. 2a, slip occurred locally between the strands and the grout and the strain in the tendon was less than the strain in the adjacent concrete.

As a consequence of this observation, a second series of tests on specimens similar to S3 was conducted in order to provide confidence in the level of stress that may reasonably be assumed to develop in the strand at the critical negative moment region adjacent to the band. Five additional specimens were constructed, three with identical dimensions and reinforcement to S3 (Fig. 2) and two with identical dimensions to S3, but with heavier amounts of conventional reinforcement. These additional specimens were labeled S3A, S3B, S3C, S4A and S4B. The loading arrangement for each specimen was as shown in Fig. 2. The conventional reinforcement details for all specimens (Series 1 and 2) are given in Table 1.

3.3 Test Set-Up and Loading Frame

The specimens were tested on-site in a specially constructed testing frame (shown in Fig. 3). The load was applied by a hydraulic jack of capacity 1,000 kN and distributed to the specimen using an appropriate spreader beam arrangement. The loads were applied to the top surface of each specimen at each loading point as a line load across the full 2 m width of each specimen.

Throughout the tests the loading frame was adequately braced and the specimen and loading elements were adequately supported to ensure stability. Each test specimen and each spreader beam were simply-supported such that rotation was permitted at both ends and longitudinal (horizontal) movement was permitted at one end only.

The loading frame and spreader beams were stiff enough to ensure movements of the frame and loading elements were insignificant compared to the deformation of the test specimen. Frame deformations were monitored throughout the test to ensure movements were not excessive.

4. Test Results

4.1 Measurements Recorded

Each slab specimen was installed and appropriately supported in the test frame, and then subjected to increasing load P applied using a hydraulic jack. The weight of the jack and spreader beam arrangements was measured, so that the initial loads acting on the top surface of the specimen were known prior to commencement of the test.

Initially each specimen was subjected to a small applied load and then unloaded, to ensure that all measurement transducers and gauges were working, and to ensure full contact at all loading and support

Table 1 Conventional reinforcement details.

Specimen	Top reinforcement		Bottom reinforcement	
	Longitudinal	Transverse	Longitudinal	Transverse
S1	6 mm square mesh (200 ctrs)		13 – T12	T10@150
	– 25 mm		25 mm	37 mm
	Cover		Cover	Cover
S2	6 mm square mesh (200 ctrs)		11 – T10	T10@200
	– 25 mm		25 mm	35 mm
	Cover		Cover	Cover
S3 (S3A, S3B, S3C)	11 – T10 + 6 mm mesh	T10@200 + 6 mm mesh	11 – T10	T10@200
	50 mm	60 mm	25 mm	35 mm
	Cover	Cover	Cover	Cover
S4A (S4B)	20 – T12 + 6 mm mesh	T12@150 + 6 mm mesh	11 – T10	T10@150
	50 mm	62 mm	25 mm	35 mm
	Cover	Cover	Cover	Cover

Notes: T10 (T12) are deformed bars of 10 mm (12 mm) diam, bar spacings are in millimeters.

Fig. 3 Specimen S2 under load in testing frame.

points. Each test specimen was then subjected to increasing load to failure. The rate of deformation applied to each specimen was controlled so that an accurate picture of the load-displacement relationship was obtained from first loading to the peak load and beyond. After the peak load was reached, each specimen was then unloaded.

The load applied by the hydraulic jack was measured using an accurately calibrated load cell and the deflection of each specimen was continuously recorded at six locations along the span using LVDTs (linear variable displacement transducers). In addition, the slip of the tendons at the free ends of each specimen (relative to the concrete end face) was also measured using LVDTs.

The deflection of the cross-head or reaction beam at the top of the loading frame at the point where the hydraulic jack reacts against the cross-head were recorded for each test and proved to be insignificant. Deflection readings along the span of each specimen were taken relative to the specimen supports (i.e., the reaction slab) and not relative to the loading frame. The reaction slab on which the loading frame was mounted can be seen in Fig. 3.

As the load on each specimen was gradually increased, the position of each crack was recorded. From time to time, each test was paused and the height of each crack was recorded, together with the crack width at the soffit of each specimen.

For each slab specimen, companion specimens in

the form of standard 150 mm cubes and 150 mm diameter concrete cylinders were cast and appropriately cured in accordance with ASTM C 31M-98 and ASTM C 192M-98, as appropriate. At the time of testing each slab specimen, the companion specimens were tested to determine the concrete compressive strength and the concrete elastic modulus in accordance with ASTM C 39M-99 and ASTM C 469-94.

Tests were also conducted on samples of the conventional reinforcing bars and the strand to determine the complete stress-strain curve of the steel samples, from which the elastic modulus, the yield stress (0.2% proof stress), the tensile strength, the corresponding strain at peak stress and the fracture strain were determined.

4.2 Method of Evaluating Test Results

Each specimen was relatively lightly reinforced and the conventional reinforcement yielded well before the peak load was reached. From the measured peak loads for each specimen and the known strengths and stiffness characteristics of both the concrete and the steel, the stress in the 12.7 mm diameter strands at the critical section at peak loads (just prior to failure of the specimen) was readily determined using only the principles of mechanics and satisfaction of the requirements for equilibrium and compatibility of deformations.

If the calculated stress in the strand exceeded the 0.2% proof stress for the material, the full design strength of the specimen was deemed to have been reached and the strand had developed its full strength via bond between the strand and the grout inside the prestressing duct. If slip occurred between the strand and the surrounding concrete, the full strength of the strand may not have been developed.

4.3 Specimen S1

At the time of testing, the measured material properties obtained from the companion specimens

were:

- Mean concrete compressive strength, $f_{cm} = 60$ MPa;
- Elastic modulus of concrete, $E_c = 30$ GPa;
- Yield stress of the conventional steel, $f_y = 575$ MPa;
- Elastic modulus of conventional steel, $E_s = 200$ GPa;
- Tensile strength of strand, $f_{pu} = 2,060$ MPa;
- Elastic modulus of strand, $E_p = 198$ GPa.

The measured mid-span moment versus mid-span and quarter-span deflection curves are shown in Fig. 4. The specimen reached its peak load capacity when the mid-span moment reached 456 kNm (which included the moment caused by the self-weight of the slab) at a peak applied load $P_{max} = 454$ kN (consisting of the maximum load applied via the hydraulic jack of 419 kN, plus the self-weight of the loading rig = 35 kN). The mid-span deflection at the peak load was 128.5 mm. No draw-in of the tendons at the ends of the specimen was recorded at any stage during the test.

An ultimate strength analysis was undertaken on the cross-section at mid-span, using the measured material properties, in order to estimate the maximum strength in bending M_{max} . Perfect bond was assumed between the strands and the surrounding concrete (i.e., it was assumed that no slip occurred on the strand-grout interface, the grout-duct interface and the duct-concrete interface).

Fig. 4 Measured vs calculated moment-deflection curves, S1.

Using the measured material properties and the requirements of strain compatibility and equilibrium, the calculated maximum moment at mid-span was 437 kNm (compared with the measured maximum moment of 456 kNm). At the calculated peak moment, the calculated stress in the tendons was $\sigma_{pu} = 1,820$ MPa and, with the strain in the conventional tensile reinforcement well past the measured yield strain, $\sigma_{st} = f_y = 575$ MPa.

The mid-span moment versus mid-span deflection curve was also calculated, using $E_c = 30,000$ MPa and the mean flexural tensile strength of $f_{ctm} = 5.8$ MPa. Yielding of the conventional reinforcement took place at a moment of 239 kNm when the stress in the tendons was just 487 MPa. The post cracking deflection was calculated using the approach for calculating short-term deflection specified in Eurocode 2 [8]. This approach is far more accurate for non-prestressed slabs than the approach in ACI 318-08 (Bischoff [9] and Gilbert [10, 11]). The calculated moment deflection curve at mid-span is compared with the measured curve in Fig. 4.

4.4 Specimen S2

The material properties for S2 are identical to those reported earlier for S1. The measured mid-span moment versus mid-span deflection curve is shown in Fig. 5, together with the calculated response of the specimen. The maximum mid-span moment reached during the test was 330 kNm at a peak load of $P_{max} = 325$ kN (consisting of the maximum load applied via the hydraulic jack, 290 kN, plus the self-weight of the loading rig, 35 kN). The measured mid-span deflection under the peak loads was 182 mm and no draw-in of the tendons at the ends of the specimen was recorded at any stage of the test.

Using the measured material properties and enforcing the requirements of strain compatibility and equilibrium, the calculated maximum moment at mid-span was 298 kNm (compared to the measured 330 kNm). At this calculated peak moment, the stress

Fig. 5 Measured vs calculated moment-deflection curves, S2.

in the tendons was $\sigma_p = 2,050$ MPa and the conventional tensile reinforcement was well past yield with $\sigma_{st} = f_y = 575$ MPa. Clearly, the measure peak load exceeded the calculated peak load and the strand must have developed its full tensile strength.

The mid-span moment versus mid-span deflection curve is also calculated, using $E_c = 30,000$ MPa and the mean flexural tensile strength of $f_{ctm} = 5.8$ MPa, and the calculated curve is also shown in Fig. 5. Yielding of the conventional reinforcement took place at a moment of 131 kNm when the stress in the tendons was just 484 MPa. As for specimen S1, the post cracking deflection was calculated using the approach specified in Eurocode 2 [8].

4.5 Specimen S3

Specimen S3 was tested in an inverted position to that shown in Fig. 2, with self-weight contributing to the peak moment at the critical section (A-A in Fig. 2). The measured moment versus deflection curves for S3 at Section A-A is shown in Fig. 6. The specimen reached its peak load capacity when failure occurred at Section A-A at the junction of the 200 mm thick slab and the 450 mm deep interior band. The moment at failure at Section A-A was 195 kNm when the deflection at this point reached 80 mm and the load P was 163 kN, which included the self-weight of the loading rig (16 kN).

Fig. 6 Measured moment-deflection curve for S3 (at A-A).

No slip was recorded at the ends of the specimen during loading. In contrast to specimens S1 and S2, the strands did not develop their full strength during the load tests, with the tendon stress at peak load reaching just 950 MPa at Section A-A.

4.6 Specimen S3A, S3B and S3C

At the time of testing, the measured material properties obtained from the companion specimens were:

- Mean concrete compressive strength, $f_{cm} = 53.3$ MPa;
- Elastic modulus of concrete, $E_c = 46.9$ GPa;
- Yield stress of the conventional steel, $f_y = 575$ MPa;
- Elastic modulus of conventional steel, $E_s = 200$ GPa;
- Ultimate tensile strength of strand, $f_{pu} = 2,060$ MPa;
- Elastic modulus of strand, $E_p = 198$ GPa.

As for S3, each specimen was tested in an inverted position to that shown in Fig. 2, with self-weight contributing to the peak moment at the critical section (A-A in Fig. 2). Each specimen reached its peak load capacity when failure occurred at Section A-A at the junction of the 200 mm thick slab and the 450 mm deep interior band. For S3A, the moment at failure at Section A-A was 219 kNm when the deflection at this point reached 83 mm and $P = 190$ kN. For S3B, the moment at failure at Section A-A was 207 kNm when the deflection at this point reached 78 mm and $P = 177$ kN. For S3C, the moment at failure at A-A was 197 kNm when the deflection at this point reached 54

mm and $P = 165$ kN. These values of P include the self-weight of the loading rig (16 kN).

The measured moment versus mid-span deflection curves for S3A, S3B and S3C at Section A-A are shown in Fig. 7, together with the corresponding curve for specimen S3. No slip was recorded at the ends of either specimen during loading.

As observed in the series 1 tests, the tendons in S3A, S3B and S3C did not develop their full strength at Section A-A (Fig. 2) during these load tests. The stress at failure in each specimen was 1,150 MPa (S3A), 1,050 MPa (S3B) and 960 MPa (S3C).

4.7 Specimen S4A and S4B

At the time of testing, the measured material properties obtained from the companion specimens were:

- Mean concrete compressive strength, $f_{cm} = 59.6$ MPa;
- Elastic modulus of concrete, $E_c = 47.5$ GPa;
- Yield stress of the conventional steel, $f_y = 575$ MPa;
- Elastic modulus of conventional steel, $E_s = 200$ GPa;
- Ultimate tensile strength of strand, $f_{pu} = 2,060$ MPa;
- Elastic modulus of strand, $E_p = 198$ GPa.

As for S3, each specimen was tested in an inverted position with self-weight contributing to the peak moment at the critical section (A-A in Fig. 2). Each specimen reached its peak load capacity when failure occurred at Section A-A at the junction of the 200 mm thick slab and the 450 mm deep interior band. For S4A, the moment at failure at Section A-A was 328 kNm when the deflection at this point reached 77 mm and $P = 315$ kN. For S4B, the moment at failure at Section A-A was 312 kNm when the deflection at this point reached 64.5 mm and $P = 296$ kN. These values of P include the self-weight of the loading rig (16 kN).

The measured moment versus deflection curves for S4A and S4B at Section A-A are shown in Fig. 8. Also shown on these curves are the calculated tendon stresses at the peak load, based on the requirements of equilibrium. No slip was recorded at the ends of either specimen during loading.

Fig. 7 Measured moment-mid-span deflection curve for S3, S3A, S3B and S3C (at A-A).

Fig. 8 Measured moment-mid-span deflection curve for S4A and S4B (at A-A).

As observed in the S3 specimens, the tendons in S4A and S4B did not develop their full strength at Section A-A during these load tests. The stress at failure in each specimen was calculated to be 1,330 MPa (S4A) and 1,170 MPa (S4B).

5. Discussion of Results

For the simply-supported supported specimens S1 and S2, of uniform thickness and spanning 5.6 m, no slip occurred between the strands and the surrounding concrete at any stage of loading and the full strength of the tendons was developed through bond. The

stress in the strand at the ultimate limit state can be calculated by enforcing equilibrium and assuming full strain compatibility, i.e., by assuming the strain in the tendons is equal to the strain in the concrete at the tendon level at all stages of loading. For the strand in these specimens, the full strength of the tendons was developed over a development length of 2,600 mm from the end of the specimen to the maximum moment location under the interior loading point (Fig. 1).

For specimens S3, S3A, S3B, S3C4, S4A and S4B, local bond failure occurred at the junction of the slab and the band beam where the sudden change in the section depth resulted in a rapid change in the tendon stress and high local bond stresses. The development length of the strand from the end of each specimen to the point of maximum moment in each specimen was in excess of 2,100 mm and, according to AS 3600-2009 and ACI 318-08, this should have been sufficient to develop a yield stress of about 1,800 MPa. At the critical section, slip occurred locally between the strands and the grout and the strain in the steel was less than the strain in the adjacent concrete. In all of these specimens, the maximum stress that was developed in the strand at the peak load was between 46% and 65% of the ultimate strength of the steel.

To provide confidence in the level of stress that may be assumed to develop in the strand at the critical negative moment region adjacent to the band, a simple statistical analysis was undertaken on the results of the six relevant specimens. The results are summarized in Table 2.

Table 2 Strand stresses at Section AA in Fig. 2.

Specimen	Stress in tendon at peak load at Section AA, σ_{pu} (MPa)
S3	950
S3A	1,150
S3B	1,050
S3C	960
S4A	1,330
S4B	1,170
Mean, σ_{pu} (MPa)	1,102
Standard Dev., s (MPa)	145

If the sample of measured tendon stresses is assumed to be normally distributed, the lower characteristic value of the tendon stress at the ultimate load is $\sigma_{pu,0.05} = 790$ MPa. This is the value of tendon stress at peak loads that is smaller than 95% of all samples. This measure is consistent with the factors of safety imposed on material strengths in many international standards, including ACI319-08 [5].

It is suggested, therefore, that the ultimate strength of the negative moment region in the slabs at the edge of the band beam (where the section depth suddenly increases and the tendons are located in the top of the slab) may be calculated based on the requirements of equilibrium and strain compatibility, assuming slip does occur between the strand and the concrete and the maximum tendon stress does not exceed 790 MPa.

6. Conclusions

Eight full-scale concrete slab specimens reinforced with a combination of conventional deformed reinforcing bar and grouted strands (with zero initial prestress) were tested to assess the stress that could develop in the slab tendons in the positive and negative moment regions of a one-way slab supported on band-beams. In each specimen, the tendons consisted of two post-tensioning ducts, each containing four 12.7 mm diameter strands. The ducts were fully grouted 28 days prior to testing. The initial prestressing force in each strand was zero, i.e., the bonded strand was initially unstressed and thus acted as non-prestressed reinforcement throughout the test. In each specimen, the strand was full-anchored in accordance with AS 3600-2009 [1] and ACI 318.08 [5].

For the prismatic specimens of constant depth, the strand at the critical section developed its full design strength, with the stress reaching the specified yield stress of the strand as the peak load was approached. For the non-prismatic specimens where the slab depth suddenly changed at the slab and band-beam connection (from 200 mm to 450 mm), bond failure

occurred and the maximum stress that could be developed in the strand was little more than 50% of its design yield strength.

Where a post-tensioned concrete slab suddenly increases in depth, such as at the edge of a drop panel or band beam, there is a relatively sudden change in the stress in the tendon resulting in high local bond stresses. In situations where the initial prestressing force in the tendon is small, the assumption that the full strength of the tendon can be achieved at the ultimate limit state may not be reasonable.

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