Commission of Inquiry
PARADISE DAM

PARADISE DAM COMMISSION OF INQUIRY

Commissions of Inquiry Act 1950
Section 5(1)

STATEMENT OF FRANCISCO LOPEZ

Name of Witness:	**Francisco Juan Alberto Lopez**
Date of birth:	[redacted]
Current address:	**C/- SMEC** **Level 5, 20 Berry Street** **North Sydney NSW 2060**
Occupation:	**Civil Engineer**
[redacted]	[redacted]
Interview conducted by:	**Jane Menzies (Counsel Assisting)**

I **Francisco Juan Alberto Lopez,** Engineer, make oath and state as follows:

Background

1. I am the Technical Principal for Dams at SMEC. I hold a Bachelor of Engineering (Civil) and a Masters in Earthquake Engineering. My main field of work is on the structural analysis of concrete dams.

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2. This statement has been prepared based on my answers to questions presented by Jane Menzies (Counsel assisting the Commission) in a teleconference on 6 February 2020. Not all discussion traversed within that teleconference has been included in this statement. Annexed hereto and marked **FL-1** is a copy of the Transcript of the teleconference that took place on 5 February 2020.

3. I have had almost 22 years of experience in dams, spanning projects in four continents and different types of dams. I have worked on roller compacted concrete (**RCC**) dams before, including in Colombia, Costa Rica, the Dominican Republic and The Philippines.

4. A copy of my curriculum vitae is annexed hereto and marked "**FL-2**".

Involvement with the Technical Review Panel (TRP)

5. I first became involved in the TRP for Paradise Dam in about February or March 2019. The first TRP workshop was in May. I was engaged by SunWater.

6. The TRP provides comments, advice, and guidance on the upgrade works that are being prepared by GHD. The TRP members have different disciplines. My particular background is in the structural analysis of concrete dams.

7. I helped prepare and signed three reports of the TRP dated 13 June 2019 (**Report No 1**) [**SUN.009.003.0613**], 23 September 2019 (**Report No 2**) [**IGE.051.0001**] and 9 December 2019 (**Report No 3**) [**SUN.009.002.0001**]. In general, I agree with the views expressed in those reports. Some parts of those reports that relate to dam structural and stability analysis are my opinions and ones that I honestly hold.

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8. The date on the signature page of Report No 1 [**SUN.009.003.0613 at 0630**] says "13 June 2016" but that is a mistake and it should say "13 June 2019". The signed version of Report No 1 has a "DRAFT" watermark on it, but that is an error. The signed version is the final version of Report No 1.

9. Writing each of the three reports was a joint effort by the TRP members. Before each report was prepared, the TRP members attended a workshop. At the end of the workshop, the TRP members convened to discuss what had been presented to us. Peter Foster was in charge of producing the first draft of the reports. Mr Foster put the ideas that the TRP had discussed into a draft report that was then circulated to the rest of the panel members for approval, discussion or additional input.

Strength and stability analysis

10. The structural analysis of a concrete dam requires understanding whether the materials that are components of the dam are strong enough to support the demand that comes from different loads that it will be subject to during its design life. Those loads may include forces from the water, seismic forces from earthquakes, and other types of hydraulic forces that come with floods. Strength analysis looks at the integrity of the materials and their capacity.

11. One part of designing a dam (strength analysis) is comparing the capacity of the materials to the expected demand on those materials. The second part is to consider the stability of the dam as a complete body; whether the geometry and other restoring features of the dam are such that it can withstand forces that try to slide or overturn the structure.

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12. In RCC dams, the strength of the lift joints is paramount to the structural design of the dam and the stability of the dam itself. An RCC dam is a concrete dam that is made of horizontal, continuous layers of concrete. An RCC dam must be safe for stability at any given horizontal joint created by the RCC compaction, as well as for the stability of the dam as a whole. One must check for potential sliding planes within the foundation and at the interface between the concrete and the foundation rock, and within the body of the dam at any potential weak layer of RCC.

Review of SunWater documents in April 2019

13. The first workshop of the TRP was scheduled for May 2019. Before that, SunWater asked me to have a preliminary look at a number of documents and to provide my opinion on them without any explanation from GHD or anyone else. I was to give SunWater my preliminary thoughts. Appendix D of Report No. 1 [**SUN.009.003.0613 at 0654**] is the report that I wrote based on the information that SunWater provided to me. The report contains my preliminary views before any TRP discussions.

Previous TRP

14. To provide context to Appendix D, a previous technical review panel had been established by SunWater for a previous phase of work. Any reference to TRP documents in Appendix D is to the previous TRP.

15. In section 2.1.3.1 of Appendix D, I refer to meeting no. 4 of the previous TRP, which considered testing from vertical cores undertaken in November and December 2015 [**SUN.009.003.0613 at 0657**]. The previous TRP compared the observed core recovery to Enlarged Cotter Dam, which is the biggest RCC dam constructed in Australia and is

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the contemporary of Paradise Dam. My understanding is that Enlarged Cotter Dam is a high paste RCC dam.

16. As discussed in section 2.1.3.1 of Appendix D, the previous TRP suggested that a full width lift joint should have a peak shear strength of at least 45° and a residual shear strength close to 45° [**SUN.009.003.0613 at 0658**]. I think that the previous TRP suggestion was made by observation of the cores but no testing had been done at the time. In concrete dams in general, in the absence of actual shear strength tests, a reasonable starting point for analysing unbonded joints – and this has been documented in several dam guidelines – is an angle of friction of 45° and no cohesion.

Combination of factors relevant to dam stability

17. In section 2.2 of Appendix D, I discuss GHD's stability assessment using the January 2013 flood [**SUN.009.003.0613 at 0660**]. I wrote:

> *[T]he actual dam behaviour during the flood was most likely the result of a combination of factors. For instance, in regards to the shear strength of the lowest RCC lift joint, it is possible that part or all of the joints are actually bonded, therefore there will be an important cohesion component that would contribute largely to the stability of the section.*

18. I referred to a combination of factors. The first factor is that the actual shear strength would be a combination of the shear strength provided by both the bonded and the unbonded part of the joints. From very limited horizontal coring, coreholes found that some parts of the joints were bonded and some were not.

19. Another factor is uplift. That is the vertical pore water pressure at the dam foundation interface, or at any horizontal layer, that would push the dam up. The basic principle of

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how the dam resists sliding is that the heavier it is, the more difficult it would be to slide it horizontally. When there is a big uplift component, the overall vertical force is less because the weight of the dam pushing down is counteracted by the pressure of the water pushing up. The "effective weight" of the dam is less when the uplift is larger and so it would be easier to slide the dam along a horizontal plane.

20. The magnitude of the uplift force can be controlled by providing a barrier, called a "grout curtain", on the upstream side of the dam. That can be combined with a membrane along the face of the dam, which is connected to the grout curtain that goes down into the foundation. The membrane and grout curtain provide a barrier from the crest of the dam down to several metres into the foundation rock. That will prevent water permeating those layers of the dam and the foundation rock mass. The barrier therefore reduces the uplift force.

21. The effectiveness of the membrane and grout curtain is difficult to measure. That is why a number of different scenarios are normally assumed. For example, one might assume the uplift is reduced to 20, 50 or 80 per cent of what it would have been without a waterproof membrane. An example of an analysis using these different uplift assumptions is shown in the graph on page 14 of Report No 2 [**IGE.051.0001 at 0014**]. The different plot lines on the graph accord with the different uplift assumptions.

22. GHD's stability assessment using the 2013 floods adopted an uplift force of 20 per cent based on instrumentation readings that were not available for my review. GHD did not conduct a sensitivity analysis of this parameter. My comment, as set out above, was raising the question about different uplift forces. The stability of the Dam should come from a combination of factors, which may include:

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a. the actual shear strength friction of the Dam being higher than it was assumed in that particular analysis;

b. uplift scenarios different from the one adopted;

c. the Dam bending downstream as a beam, which prevents dislodging of the monoliths of the Dam; and

d. transference of deficit of shear strength of a weak monolith to adjacent monoliths with excess shear strength.

Additional frictional resistance in vertical contraction joints

23. In Appendix D of Report No 1, I also wrote [**SUN.006.003.0613 at 0660**]:

> *[I]t is possible that a particular monolith had an nominal FoS of less than 1.0, but its deficit of shear strength was transferred to the adjacent monoliths via the shear strength of the vertical contraction joints.*

24. The Dam is divided into monoliths. RCC is constructed in continuous, horizontal layers, but while the concrete is still fresh, a piece of plastic (a "joint breaker") is introduced in the concrete to create vertical contraction joints. Those joints are not smooth so there will be some shear strength that prevents the sliding of one monolith with respect to the next one. For instance, if there was a monolith in the middle of the Dam with a completely unbonded layer and therefore a good candidate to slide, the vertical contraction joints with the adjacent monoliths might provide additional frictional resistance. When the monolith tries to slide along that unbonded joint, its deficit in sliding capacity may be transferred to the adjacent monoliths with better sliding capacity.

25. To the inquiry question "Is that the very purpose of those vertical joints?" I mentioned that providing additional friction is not the purpose of the contraction joints. They are there to help manage concrete cracking. Concrete cracks because of the thermal reaction that takes place. Past studies in concrete dam technology have shown that dams crack in a vertical way every 15 to 40 metres, depending on the mix and on how much heat the mix generates when the concrete is poured and hardens. Instead of the dam cracking randomly, the induced vertical joints locate the cracks where the designers want them so that they can be controlled. Water stops can be located to prevent water penetrating the dam wall. The concept is similar to a concrete sidewalk. Contraction joints are used to crack the sidewalk exactly where the designers want to so that it does not look ugly.

Comparison of Paradise Dam to other RCC dams

26. Section 2.3 of Appendix D contains a graph that I generated as a starting point for comparing the different shear strength values reported in the design report and from GHD's analysis to other documented shear strength measures in RCC dams in different parts of the world. The graph plots data for similar RCC dams in Brazil, the US and Vietnam [**SUN.009.003.0613 at 0662**]. That data is from a paper by the US Society on Dams.

27. The graph shows data for dams constructed with high paste RCC, low to mid paste RCC, and low to mid paste RCC with bedding. Low paste and low paste with bedding are relevant to Paradise Dam. Bedding is included in the RCC design and construction for two main reasons. First, it helps with impermeability at the upstream face of the dam. It forms part of the system provided by the membrane and the grout curtain. The other reason is to help in bonding the horizontal layers when cold joints are generated.

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There have been discussions about how much bedding was used in Paradise Dam and whether that was enough. That issue is still in discussion.

28. In the graph I superimposed shear strength quoted in:

    a. the Paradise Dam design (being the points labelled "BDA 2004");

    b. the reports prepared during the dam safety review conducted in 2016 (I do not know the author); and

    c. GHD's work in 2018.

29. I estimated the total shear strength for each case from reported cohesion and angle of friction values and assuming a normal stress of 500kPA.

30. The graph gave me a comparison. It shows that the design values and those used by GHD are relatively low compared to the data that has been presented on the graph.

Identifying bonded and unbonded joints

31. In Section 2.3 of Appendix D, I included a photograph of core drilled from Paradise Dam (on the left) and a photograph of core taken from an RCC dam in Brazil (on the right) [**SUN.009.003.0613 at 0663**]. Of the Paradise Dam cores I said that:

> *Photographs of the cores tested for shear do not seem to clearly intercept any bedding mix. However, the bedding mix could be thin and difficult to identify.*

32. There have been difficulties in finding the bedding mix in any core hole from Paradise Dam. The TRP has asked for photographs to try and verify that the bedding mix was actually placed. In the picture on the left, we could not see any bedding mix, but it

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could be that the borehole location was outside an area where bedding was placed. It is not well-defined where bedding was actually placed.

33. In theory, when you have a bedding mix the core should look like the photograph on the right from the Brazilian dam. That photograph shows a thin line of bedding mix that you can identify.

34. If a corehole is taken where bedding mix has been used, the bedding mix can normally be seen in the core as long as the sample is intact. An example is on page 25 of Report No 1 in Appendix B [**SUN.009.003.0613 at 0637].** In the top photograph, bedding mix can be seen. That borehole was drilled at an angle.

35. The problem is that when vertical cores are drilled, the torque imparted by the bit of the drill can break the bonding between the bedding mix and the RCC. The mechanical process of drilling might chew the bedding mix and a joint may look unbonded. The bedding mix can in that circumstance disappear.

36. That is one of the challenges when bonding conditions are not good. It makes the bedding mix difficult to identify.

37. The process of identifying joints that were fractured by the coring process and joints that were previously unbonded is difficult. That is a relevant question for Paradise Dam. If it were easy to identify bonded and unbonded joints, it would be easier to understand if some cohesion should be allowed for in estimating shear strength.

38. There are methodologies to help with this problem that use an acoustic televiewer or an optical televiewer. The method compares the core that has been extracted with

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information retrieved by sending those exploratory tools down the corehole. They help visualise the condition of the joints within the dam itself. This allows a correlation to be drawn between bad joints observed in the televiewer, and observations of the core.

39. Paradise Dam does not leak much, but in dams that are subject to a lot of leakage through the joints, bad joints can be identified by calcite and chemical deposition at the joints. That helps identify unbonded joints. In Paradise Dam it is more difficult, because there are not many joints where there is active leakage or passage of water. That is a good thing, but it makes it more difficult to identify broken lift joints.

Possible text book explanations

40. On the last page of Appendix D [**SUN.009.003.0613 at 0664**], I provided a list of possible explanations for the low shear strength at Paradise Dam identified by GHD. Those explanations are "text book" and would need to be explored to either confirm or discard, individually or in combination.

Site visit on May 2019

41. I visited the Dam on 28 May 2019 along with the other members of the TRP. I wrote a report about the site visit, which is included as Appendix B to Report No. 1 [**SUN.009.003.0613 at 0634**].

42. On pages 23 and 24 of Report No. 1 in Appendix B [**SUN.009.003.0613 at 0635 and 0636]** there are photos that I took of cores taken from the Dam. We (TRP members) were visiting the Dam when the core shown in page 23 was still at the crest of the dam, so it was extracted either the day before or the same day of the site visit. The core

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shown is page 24 was stored in a shed several meters from the dam, so it was drilled and extracted before the TRP site visit.

43. The core log on page 23 of Report No. 1 [**SUN.009.003.0613 at 0635**] was from the crest of the Dam. The core shown on page 24 of Report No. 1 [**SUN.009.003.0613 at 0636**] had been stored in a shed about 300m from the Dam for several days. I do not know when the cores on pages 24 and 25 were drilled.

44. With respect to the core in the photograph on page 23, I said on page 24 [**SUN.009.003.0613 at 0636**]:

> *The observed conditions of this contact denote poor cleaning and preparation of the foundation during construction and may significantly impact the shear strength/sliding stability of the dam.*

45. This is relevant to the principle for design and stability of concrete dams. Concrete dams are supposed to be founded on good-condition rock. If the general foundation is too weathered or broken, other types of dams such as rock fill or embankment dams are normally used. Excavations for a concrete dam normally go down to good rock foundation.

46. Cores are drilled in a continuous cylinder, often 50 metres deep or more. They are broken and stored in boxes that are 1.5 metres long.

47. In the photograph on page 23 [**SUN.009.003.0613 at 0635**], the left of the top row of core is concrete. The interface with the rock is where the arrow is pointing. The core in the picture goes from more superficial on the left to deeper on the right. The second row of the box is the next bit of rock. Right after the concrete-foundation contact, the

rock is very fractured and is more like soil. In an ideal situation, the interface between the concrete and the rock is so sound that the most superficial rock may have an appearance similar to that of concrete. It may have a different pattern or colour, but it would look smooth and continuous. In this case, it is quite discontinuous. The rock in the core appeared to be a material that you can disturb easily. That is not the ideal foundation for a concrete dam. Other cores drilled at the dam did not show the same soil-like foundation.

48. On page 24, I wrote [**SUN.009.003.0613 at 0636**]:

> *[T]he observed joint conditions along each core were highly variable, from very good bonding (even without [bedding]) to completely unbonded. … GHD should carefully consider this variability when adopting the shear strength parameters for the RCC lift joints, as the interfaces are far from homogeneous.*

49. It is never an easy task to account for that variability. This is related to the discussion below about whether the adopted angle of friction is too conservative. The comment above points out that not all the joints were unbonded. A bonded joint will have a larger shear strength than an unbonded one. Shear strength along a lift joint would have contribution of the shear strength provided by both the bonded and unbonded zones. The TRP recommended that GHD take that into consideration where using a final shear strength for design purposes.

Dam stability including shear strength

GHD's angle of friction

50. I contributed to section 3.3 – headed "Dam Stability" – of Report No 1 [**SUN.009.003.0613 at 0620**]. In that section I said that, "*the strength estimated by*

GHD might be a bit too conservative". That was not a criticism of how GHD established a shear strength of 37 to 38 degrees. I understand why they were conservative in their approach. From memory, at the time GHD took six coreholes of between 100mm and 150mm diameter, which a very small sample, to characterise the whole dam for a stability analysis. The shear strength adopted by GHD (37 to 38 degrees) resulted from a regression of shear tests conducted from those few samples.

51. That must be put in context. The stability of the dam is being assessed along a whole horizontal layer of RCC, which can be hundreds of square metres in area. GHD has conducted a preliminary assessment of the stability of the whole dam using a shear strength derived from the tests conducted on a very limited number of samples. From the available test results at the time, GHD's adopted shear strength is valid. But there are other factors that affect the actual shear strength of a particular layer of concrete. For example, there is uncertainty about if the joints in the six holes were actually unbonded, and whether the layer is bonded in other areas that have not been investigated. The layers of RCC in the corehole samples appeared to be unbonded. That definitely affects the shear strength.

52. My view is that it could be overly conservative to assume that the whole layer is unbonded, because the whole layer was not investigated. It is impossible to investigate the whole layer, so some uncertainty could be factored in. It is one thing to determine shear strength from testing (which is 37 to 38 degrees here), but another thing to decide what angle to use in the stability analysis of the dam. The comment in the report is directed to this issue.

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53. I am not saying that GHD's approach is incorrect, but it was suggested that GHD would need to consider all the factors in their analysis. For example, if the surface is not completely horizontal but has some undulations, those undulations contribute to shear strength. Those other factors are very difficult to measure, if not impossible. But the TRP is making the point that just because a limited number of tests show 37 or 38 degrees, it could be too conservative across the whole dam. Unless that angle is proven with additional shear strength tests, which the TRP recommended later on, it may be too conservative.

54. The shear strength tests results were 37 or 38 degrees. I cannot recall seeing such low friction angles before in concrete layers.

Testing of coreholes

55. There is a comment on page 9 of Report No. 1 that "*tests did not follow all the same testing standards and were sourced from both vertically and horizontally drilled cores*" [**SUN.009.003.0613 at 0621**]. The TRP recommended that future shear strength testing be conducted using the same standards, and that the shear strength used for future design be adjusted accordingly.

56. Shear strength is the key parameter for identifying deficiencies in the Dam. Every effort should be made to use standardised procedures in testing to achieve a more credible result. This is because, even in GHD's estimation of a friction angle of 37 to 38 degrees, there is a statistical margin of error. Report No. 1 was at the start of the TRP's work and the TRP members were asking for more samples to be taken, and for those samples to be tested with the same standards to avoid any potential error and to

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provide confidence in the adopted shear strength parameters for future analysis and design.

GHD's stability analysis

57. In Report No. 1, GHD's stability analysis was described as assuming "*that a bilinear shear strength is applicable with a frictional strength of 37˚ with zero cohesion for normal stresses less than 600 kPa and an apparent cohesion (to be defined) with a friction of 28˚ for normal stresses above 600 kPa*" [**SUN.009.003.0613 at 0621**].

58. This is describing a bilinear graph with confining pressure on the horizontal axis. Imagine there is a mass lying on a very polished surface. It will slide relatively easily. But if a vertical force is applied to the top of the mass, then sliding it horizontally would be more difficult. The vertical force is the confining stress.

59. Between two layers of concrete, friction is proportional to the vertical force applied, up to a certain point. If there is no cohesion, when the vertical force is zero, shear strength is also zero. For any vertical pressure up to 600kPA, the shear is 37 degrees. The relationship might be defined as linear.

60. In reality, the relationship is not linear, and was adopted by GHD as bi-linear. It starts to curve and become flatter as the confining pressure (or the vertical force) increases beyond 600kPa. In GHD's analysis, after 600kPa the angle of friction is flatter at around 28 degrees. However, shear strength of a surface is defined not only by the friction but also by the cohesion. GHD is saying that above 600kPa, the angle of friction goes down from 37 to 28 degrees, but there is an apparent cohesion factor that was not present at lower vertical force on the first part of the curve.

GHD's assumed density

61. On page 9 of Report No. 1 [**SUN.009.003.0613 at 0621**], GHD's assumed density of 2,400 kg/m^3 is discussed. The TRP recommended that the density be verified using construction records. I do not know if GHD did that. 2,400 kg/m^3 is a conventional density for concrete.

Report No. 2

62. Page 14 of Report No. 2 [**IGE.051.0001 at 0014**] sets out the minimum factors of safety in sliding from the ANCOLD guidelines for a well-defined residual strength. The report quotes from the ANCOLD guidelines:

> *'Well-defined' means a sufficient number of tests have been done on concrete core from the dam and lift surfaces to give the strength parameters with reasonable certainty (e.g. assumed strength is exceeded by 80% of the test results from a test regime involving a significant number of tests).*

63. The expressions "sufficient number of tests" and "strength parameters with reasonable certainty" are vague. I think that the guidelines are vague on purpose. Because every dam is different, engineering judgment must be used. When the guideline says "sufficient number of tests", there is no way to put absolute bounds on that. What number of tests would be sufficient? One answer may be the number of tests that provide confidence to the designer.

Report No. 3

64. Page 3 of Report No. 3 [**SUN.009.002.0001 at 0003**] is a discussion about the work of TatroHinds and its review of GHD's adopted shear strength. TatroHinds made some comments about the strength testing conducted by GHD.

Shear strength testing

65. I did not see the tests being conducted, but I understand that there are different ways of testing for peak and residual strengths. I understand that the particular laboratory used by GHD did the testing in the following way, in general terms.

66. First the laboratory tests a bonded sample by subjecting it to a sustained vertical force and an increasing shear force until the sample shears. That creates a curve of applied shear force versus displacement until failure, which is the peak strength. Because the bonded sample has been sheared, the result would be representative of the shear strength of a bonded joint sample.

67. The same sample (now sheared and unbonded) is then subjected to a higher sustained vertical force with an increasing shearing force. The shearing force at which the sample fails (a large displacement was generated) defines it- as the residual shear strength.

68. The same residual path of the test is repeated three times on the same sample (as I understand it), each time with a larger sustained vertical force and an increasing shearing horizontal force.

69. In Report No 3 [**SUN.009.002.0001 at 0003**], the TRP discusses the TatroHinds comment that:

> *If the testing is to represent residual strength once a dam block begins to slide on an unbonded lift, it will require large displacements to define a residual strength,*

by saying that:

> *[L]arge displacements is a loose term.*

70. Mr Tatro said he was doubtful of that the shear strength found at the laboratory with this particular test was representative of large displacements.

71. The ANCOLD guideline that mentions large displacements does not give much guidance about how much displacement is needed to determine residential shear strength. The TRP's comment above was a reference to the ANCOLD guideline. This is a very open subject. The concept of residual shear strength for a concrete dam is not well defined in the guidelines.

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OATHS ACT 1867 (DECLARATION)

I, Francisco Juan Alberto Lopez, do solemnly and sincerely declare that:

(1) This written statement by me dated 11 Feb 2020 is true to the best of my knowledge and belief; and

(2) I make this statement knowing that if it were admitted as evidence, I may be liable to prosecution for stating in it anything I know to be false.

And I make this solemn declaration conscientiously believing the same to be true and by virtue of the provisions of the *Oaths Act 1867*.

……. Signature

Taken and declared before me at …….. this
…….. day of ……. 2020.

Taken By …….
Justice of the Peace / Commissioner for Declarations / Lawyer