Commission of Inquiry
PARADISE DAM

PARADISE DAM COMMISSION OF INQUIRY

Commissions of Inquiry Act 1950
Section 5(1)

STATEMENT OF PETER FOSTER

Name of Witness:	**Peter Francis Foster**
Date of birth:	[redacted]
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Occupation:	**Engineer**
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Statement taken by:	**Jonathan Horton QC and Thea Hadok-Quadrio**

I, Peter Francis Foster, Engineer, make oath and state as follows:

Background

1. I am a Principal Design Engineer employed by Stantec New Zealand (**Stantec**). I have been employed with Stantec since May 2016, and its predecessor MWH Global since April 2002. My area of specialisation is in dam design and dam safety. I contributed as an expert review panel member to the ANCOLD Guidelines on Design Criteria for Concrete Gravity Dams.

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2. I hold a Bachelor of Engineering (Hons) and have worked as a design and dam safety engineer since November 1975. I am a chartered professional engineer in New Zealand and a registered professional engineer in Queensland.

3. A copy of my curriculum vitae is attached to this statement and marked **'PF1'**.

Technical Review Panel (TRP)

4. The first meeting of the TRP which I attended took place in May 2019, by which time the TRP had already been established. The TRP members were engaged by Sunwater to offer suggestions and recommendations. We do not have Registered Professional Engineer Queensland (**RPEQ**) accountability (discussed later in this statement) for any changes to the design of the Dam, and nor is Sunwater bound to accept any advice that the TRP offers.

5. I also attended a meeting of the TRP in September 2019. By the time of this second meeting, the TRP had now expanded to include 2 new members. They were Mr Glenn Tarbox from the USA (an expert in Roller Compacted Concrete (**RCC**)) and John Young from Canada. Mr Young is very experienced engineer on geological and technical issues regarding the Dam's foundations. Both Mr Tarbox and Mr Young are also employed by Stantec. Mr Tarbox attended the meeting via Skype and Mr Young attended the meeting

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in person. The result of this meeting was TRP Report No. 2 dated 23 September 2019 [**IGE.051.0001**].

6. The third meeting of the TRP that I attended was on 19 November 2019. The product of this meeting and subsequent discussions and analysis was TRP Report No. 3 dated 09 December 2019 – [**SUN.009.002.0001**].

7. TRP Report No. 3 confirms the conclusions stated in TRP Report No. 2 and expresses the view that Paradise Dam (**the Dam**) clearly has safety concerns if left as it is.

8. I agree with the views expressed in TRP Reports No. 2 and No. 3. The opinions, however, that are within my area of specialisation, are those that concern dam safety and design, and do not include the detail associated with RCC and geotechnical and geological elements. On those topics, I depend upon Mr Young and Mr Tarbox for specialised knowledge on geotechnical matters and RCC respectively. The parts of TRP Reports No. 2 and No. 3 that relate to dam safety and design are my opinions and ones I honestly hold. I relied upon facts and assumptions in expressing those opinions and the source of them is set out in those reports and in this statement.

9. Mr Steven Tatro from the US joined the TRP's 19 November 2019 meeting in person and gave a presentation on RCC and joint strength. The TRP members concluded that the Dam in its present state has a risk profile that plots well above the ANCOLD tolerable risk criteria and does not meet industry standards of gravity dam stability. The Dam in my view is susceptible to base cracking at the Dam foundation and at unbonded lift joints

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with a likely failure mode (if cracking occurs) involving shear sliding failure along an unbonded lift joint within the RCC.

10. The TRP notes in Report No. 3 that lowering the Dam crest by 5 metres offers a first step in risk reduction, but the Dam would, even if that were done, still plot above the tolerable risk line and stability is not compliant with ANCOLD gravity dam guidelines. The 5 metre lowering should in my opinion be an interim step of short duration toward a greater lowering. Lowering the crest by 10 metres would provide a significant risk reduction relative to the Dam's existing situation.

11. The risk profile for the 10 metre lowering option (without the additional works of spillway protection downstream and the secondary spillway protection works) presented in GHD's recent memoranda which I have read (dated 5 September 2019 [**GHD.004.0001**] and 25 November 2019 [**GHD.005.0001**]) is very close to the tolerable risk criteria for the existing Dam, and stability and would meet ANCOLD criteria to an extreme flood of 1 in 100,000 AEP (approximately). It should be noted that in the zone below the ANCOLD Limit of Tolerability, the risks are tolerable only if they satisfy the ALARP principle (as low as reasonably practicable). The TRP notes in Report No 3 that it is supportive of the Sunwater proposal to lower the Dam spillway crest by 5 metre in 2020 and proceed to a 10 metre lowering in 2021.

12. The TRP considers it prudent to undertake other work for the purpose of resisting alternative failure modes associated with foundation materials scouring and eroding downstream of the stilling basin and on the right abutment in the secondary spillway flow path. The necessity of these works and any subsequent anchoring of the Dam monoliths

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should, in my view, be considered using the ALARP principle when the risk profile plots below the tolerable risk line.

13. The TRP suggested breach calculations be run for failure of the secondary spillway blocks once the secondary spillway monoliths were breached. As the secondary spillway commences to operate, separate inundation maps should be prepared and included in the emergency action plan for Paradise Dam.

14. The emergency action plan that presently exists seems to show that most of the inundation mapping was done for failure of the whole Dam. The view of the TRP was that when that secondary spillway comes into play, the amount of erosion that could occur is quite significant and at that point inundation maps ought be made available because, from an emergency planning perspective, it may be best to commence evacuating those who might be affected early and not wait until notice is given that the Dam has failed on an abutment. By way of example, the order to commence an evacuation downstream of Oroville dam in California was given ahead of any breach of the emergency spillway once the Sheriff was advised of the consequence of the spillway beaching, rather than waiting for a breach to be confirmed. In the end the Oroville emergency spillway did not breach as the lake was able to be lowered to cease spilling at the emergency spillway.

15. While I am not an RCC expert, the principles for stability and stress analysis of an RCC dam are the same as for a conventional concrete gravity dam, and I have extensive experience conducting stability and stress analysis on conventional concrete gravity dams. In addition, John Young and I were both engaged to conduct a review of the foundation stability (John's expertise) and stability (my expertise) of the 144 metre high

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RCC dam, known as "Murum Dam", in Sarawak. As such, I consider that I have sufficient expertise to comment upon the stability issues with the Dam.

16. My approach to looking at the issues associated with dam stability is to try to answer the question: 'What is the failure mechanism? How could the dam fail?'

17. There is evidence in the history of dams going back to the 19th century that if there is sufficient load on a gravity dam, such that the heel of the dam goes into tension it can crack. Then water can get into the dam wall or foundation interface under high pressure and increase the extent of cracking at the dam base or within the dam body. This can create a shear failure through the remaining uncracked or compression section of the dam body. The end result is a dam body rupture and uncontrolled release of the reservoir contents.

18. The stability work reported by GHD showed that the base cracking was likely and that sliding factors of safety went below recommended ANCOLD criteria for extreme flood loads, and below a sliding factor of safety of 1.0. The Dam could basically shear on any unbonded joint which has a deficiency in strength. And because the Dam is built with vertical joints between Dam block monoliths, there is a side release mechanism that could allow a monolith or group of monoliths to fail by sliding and give rise to a Dam breach.

19. I subsequently completed my own analysis of an RCC Dam block assuming a potential sliding plane at a lift surface at EL 32.4 metre for a 1 in 2,500 AEP flood. I considered two scenarios. The first is that the sliding plane also runs through the stilling basin apron, and the second is that the sliding plane daylights at the toe of the Dam block above the apron floor level. I assumed a drainage efficiency of 50% (as was assumed in the original

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design). For the first case scenario I calculated a similar base cracking and a sliding factor of safety of 0.78, similar to the result reported by GHD for a 1 in 2,500 AEP flood.

20. For the second case, the stability was higher but was sensitive to the assumed drainage efficiency. Piezometric data presented to the TRP by GHD indicated that some piezometers, particularly PG02 and PG03 in spillway monolith G, have a drainage efficiency below 50%. This data is set out in section 6.3.3 of GHD's preliminary design report [**SUN.009.003.0001 at 0045**]. The ANCOLD Guidelines for Design of Gravity Dams recommend caution regarding the extrapolation of drainage efficiency to reservoir levels that are higher than the reservoir levels where the measurement was recorded. Unfortunately, during the 2013 flood, no piezometric data within the Dam was obtained as the data logger was located in an area that was flooded by the tailwater level rise. Therefore, there is currently no available information from which a reliable extrapolation above the full supply level may be made. For the second case, I also found that when drainage efficiency was below 35%, there was the potential for base cracking and sliding factors of safety were below 1.0.

21. In my opinion, there are credible uplift and drainage efficiency scenarios for the Dam that indicate a crack scenario can develop and lead to a sliding shear failure on unbonded RCC lift joints within the Dam.

22. With respect to the drainage system within the Dam, it differs from traditional gravity dams in that it is not connected to a gallery within the Dam that allows access for maintenance of the drains. Therefore, it is difficult to obtain information upon which the drainage efficiency may be determined. As such, GHD considered in their stability analysis a sensitivity check regarding the effect of drainage efficiency based on failure

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(full uplift), 20% drainage efficiency and 50% drainage efficiency. In my opinion this is the correct thing to do as it appears the original design intent of 50% drainage efficiency is not conservative in some areas of the Dam.

Adequacy of apron

23. The stilling basin apron was not long enough to prevent erosion developing in the rock downstream of the apron in the 2013 flood. In the presentations that the TRP saw, GHD had done some two and three dimensional computational fluid dynamic hydraulic modelling that demonstrated that, for floods larger than the one in 2013, the power of the jet separates from the Dam face and impacts the foundation downstream of where the existing apron is. A larger flood could start a whole new erosion process immediately downstream of the post 2013 remedial works and the existing apron. In my opinion, if a flood larger than the 2013 event was to occur, the downstream rock erosion would still be likely to occur due to the length of the apron. This would present a major risk to the Dam stability.

24. In terms of the apron on the stilling basin, the hydrology and the hydraulics showed that flows coming down from the spillway could in fact erode rock downstream of where the current basin is, and that foundation geology is affected by fault systems. The TRP geologist (Mr Young) described this as a 'melange' rock, that is, it has been tectonically sheared and broken up and so it is quite susceptible to erosion. It follows that there could be an event that created scour similar to the problems experienced in past floods, but more extensive. If it were strong enough, it could allow a potential foundation failure mechanism to eventuate.

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25. The right abutment's spillway is founded on extremely weathered and highly weathered rock. In my opinion, it would have been preferable to take the excavation down to less weathered stronger material. The existing foundation material is probably quite erodible, because some of it is near soil-like. This is again, a reminder of the Oroville situation, where the emergency spillway only had a foot of water going over the top of it, but because the materials downstream were quite erodible, erosion gullies were working their way back towards the dam and threatened to take out the spillway foundation. The right abutment of the Dam has a nominal apron protection on it to take the large flows that could occur, and if there is a very deep channel eroding out at the toe of the right abutment spillway blocks, then these blocks could fail by sliding or toppling leading to a breach of the reservoir through the right abutment.

26. GHD produced the opinion that either on the existing Dam or the 10 metre lowered option, the right abutment would have to be substantially protected with a much thicker apron extending out a lot further and anchored further down into the rock. Their modelling suggested that, for the existing Dam, an additional high wall would be required at the downstream end of the slab to contain the flows. It was a rather expensive solution with a lot of concrete going in. For the 10 metre lowering, the option that was presented to the TRP was a scaled down version of the option for the full supply level. The existing apron in my opinion is not sufficiently robust in terms of size and anchoring to protect the right abutment from scour when the secondary spillway operates.

27. The two biggest concerns in my mind are the potential for shearing through the Dam, or a right abutment failure mechanism developing under unusual or extreme floods larger than the 2013 flood of record.

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28. If there was such a flood, the secondary spillway might kick in before the main Dam has failed. If that occurred, erosion at the toe of the secondary spillway could be the trigger for a right abutment failure mechanism. The TRP noted in TRP report No 3 the approximate flood events when the secondary spillway would begin to operate, based on a range of options, would be as follows:

    (a) for the existing Dam, the secondary spillway would begin to operate in a 1 in 800 AEP flood event;

    (b) for a 5 metre lowering of the primary spillway crest, the secondary spillway would begin to operate in a 1 in 2,000 AEP to 1 in 3,000 AEP flood event; and

    (c) for a 10 metre lowering of the primary spillway crest, the secondary spillway would begin to operate in a 1 in 10,000 AEP to 1 in 15,000 AEP flood event.

    As can be seen, lowering the primary spillway crest significantly reduces the probability of the secondary spillway ever operating and, therefore, reduces the risk of a right abutment failure mechanism at the Dam.

29. TRP Report No. 2 (page 10 [**IGE.051.0001 at 0010**]) includes a memorandum written by Dr Ernest Schrader that had been made available to us. That memorandum discussed the reduction of the amount of bedding mix on the cold joints as defined in the RCC specification. For a reason of which I am not aware, those building the Dam decided to reduce the areal extent of bedding mix required on a cold joint. They seemed to take the view that a friction only component of the strength of the joints would be sufficient for sliding stability. However, I did not see any calculations in the Detail Design Report for the Dam where those designing the Dam had tested the sensitivity of their stability

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analysis for a friction-only situation. They seem to have taken on faith Dr Schrader's view that the Dam essentially achieves stability with current friction values alone. This, in my view, is a problem, because the recent analysis by GHD clearly shows that it does not meet friction only criteria under current ANCOLD Guidelines, nor would it have met the criteria provided by other internationally recognised agencies that were available at the time of the design. Dr Schrader's view may be correct in some dams, but in a situation where you get such large reservoir level rises during flood loads (as occurs in in many Australian rivers), it is not necessarily a valid assumption.

30. In the early days of dam engineering, engineers did not understand there was the additional force called 'uplift' on surfaces of contrasting permeability. If there are unbonded joints in the Dam, that could be another interface that causes a problem. The initiator for cracking at the higher flood levels is tension on the lift or foundation surface. Typically, all the design manuals for gravity dams now require consideration of the full pressure from the reservoir getting into any tension cracks that may form at the foundation interface. The ANCOLD guidelines recommend that the same drain effectiveness and uplift assumptions that are applied to the concrete to foundation interface should also be adopted for the interface between lift joints within the body of the Dam structure.

31. In the Dam, near the base, the drainage system does not go all the way down to the lowest lifts or into the foundations. It is therefore appropriate to assume that at the foundation interface there is no uplift reduction from drainage and that the uplift profile extends to the end of the stilling basin for the spillway blocks. If you get tension on an unbonded joint or at the Dam to foundation interface a crack can form. That is the starting mechanism for failure because the Dam is moving from compression stress in the heel of

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the Dam to tensile stress. In some cases a crack with reservoir pressure within it will self-stabilise within the Dam body. In other cases, the crack can extend right through the Dam body and then you have lost all the cohesive bond, if it ever existed, and you are just down to the friction resistance to resist the reservoir thrust on the Dam body.

32. With respect to the RCC lift strength parameters, Mr Steven Tatro has been somewhat critical of the RCC lift joint testing undertaken at the request of GHD because one sample had been used to get a number of different results, such as peak strength, peak unbonded and residual strengths. I refer in particular to Mr Tatro and Mr Hinds' report of 25 November 2019 'Paradise Dam Shear Strength Evaluation Comments' [**IGE.028.0001**].

33. It is true that when testing is done for the first time on a sample, the result peaks, and when the sample starts to move, it shears further and this produces different strength test results. Mr Tatro's concern, as I understand it, was that he believed that perhaps the sample had been degraded too much by doing it this way, and he suggested that GHD ought to have obtained more single-point data rather than try to extract so much from the single sample.

34. My experience in rock mechanics is that we generally follow the direct shear testing procedures used by GHD at the Dam. Stantec has recently done similar testing on rock joints at Burdekin dam. If, for example, a shear failure of the type that I was envisaging occurs at the Dam, in which one starts getting both cracking and shear sliding on an unbonded joint, then shearing movements would exceed those done in a direct shear test. The shear strength results at the larger displacement would be relevant in order to assess the residual shear strength and the relevant safety criteria given in the ANCOLD guidelines.

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35. It seemed to me that taking our samples out to 50mm of shearing was still giving an indication of where the residual strength lies. The ANCOLD guidelines compel us to look at unbonded strength from that perspective and Steven Tatro, on my reading of his report, seems to acknowledge as much. When those strengths are used in the stability analysis, the Dam is shown to be unstable and the critical flood load is not much larger than the flood of record.

36. Dams can fail quickly. A bigger flood than the dam is designed for can bring about quite a brittle failure: one moment it looks good, and then the next moment it goes. I recently re-read a paper related to the failure of Khadawasla dam written 40 years ago. A flood came down from failure of a dam upstream and it is reported that the Khadawasla dam could be seen straining for a while with the overtopping, and then all of a sudden it opened up like a door. It sheared down one side towards the base of the dam. That is the sort of picture that I had in my mind here if Paradise Dam spillway monoliths were to ever fail by shearing on an unbonded RCC joint. A copy of that paper, 'Tensile Failures in Stone Masonry Gravity Dams in India', is attached to this statement and marked **'PF2'**.

37. Regarding lift joint RCC failure, the resistance forces from the friction angle of 39.3 degrees on the surface lifts are lower than the applied shear load on the surface so that it could start sliding on that surface because the driving force is larger than the resisting force.

38. A copy of a study by the USBR, 'Concrete Dams Case Histories of Failure and Non Failures With Back Calculations', is attached to this statement and marked **'PF3'**. Some of the failures referred to in that study are effectively base cracking and shearing of

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monoliths causing failure in these dams. That led me to pose the rhetorical question included in TRP Report No. 3 which is: have other dams failed by this sort of mechanism? We answered that question 'yes'. It is important in engineering that we learn from past errors and not repeat a problem previously experienced which can be avoided.

Construction records, specifications and communications with the original dam designers

39. All of the records referred to at page 23 of the TRP Report No. 2 [**IGE.051.0001 at 0023**] came from Sunwater as part of the documents that were provided to the TRP.

40. At the first workshop, TRP members received records from Sunwater which were initially mainly existing Dam drawings, including:

    (a) some photographs of the primary spillway RCC construction and the right abutment foundation;

    (b) a quality control report containing photographs dated September 2004;

    (c) typical dam section drawings; and

    (d) GHD's geotechnical model report and a Preliminary Design Report for the design of rehabilitating the Dam, keeping it where it was, or 5 metre or 3 metre lowering.

41. Later, during the second workshop that I attended, more shear test results became available and some additional photos of RCC placement were found as well as RCC test results.

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42. Between those workshops, a telephone call took place with Richard Herweynen facilitated by Craig Hillier. After that call, Mr Herweynen sent the TRP members a number of documents. They included:

    (a) some quality control monthly reports from July 2004 to about January 2005;

    (b) a report on pouring the RCC in the Dam which was done near the end of the job;

    (c) a due diligence report from Mr Schrader;

    (d) two papers written by Herweynen, Schrader and others (published in 2004 and 2005 for the ANCOLD conference);

    (e) a section from the Detail Design Report on geology and geotechnical design; and

    (f) section 5 of the Detail Design Report which covered the stability analysis at the time of the design (although I have subsequently obtained a full copy of the Burnett Dam Detail Design report, dated 2004 [**GHD.002.0001**]).

43. One of the questions that I asked Mr Herweynen was where he had got the strength parameters from for the design of the Dam, and about their concept of 'lift joint quality index'. I recall that Mr Herweynen said that the design parameters for joint strength came from Dr Schrader's experience on other projects and that Dr Schrader had developed the concept of the lift joint quality index himself.

44. Dr Schrader wrote about this concept in a paper published in 1999 titled "Shear strength and lift joint quality of RCC" [**PDI.040.0001**] that Mr Herweynen had, I understand, sent to Sunwater, and which Sunwater then sent to the TRP members. That paper said that,

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although those building the Dam had done a lot of testing of RCC in terms of compression strength and tensile strength during the project, they did no shear strength testing and relied on international data to arrive at their shear strength parameters for the lift joints.

Adequacy of foundations

45. When considering the impact of the foundations on the stability of a concrete gravity dam, the question is, how could this dam slide on a planar defect? If the dam has a continuous seam through the foundation that is full of clay, and it is orientated in an adverse position, then the dam could slide along that seam. If the dam has a highly erodible rock mass, but the joints are all interlocked and there is no continuous seam, then the strength will be governed by the rock mass and it may develop a plane of sliding if the applied load exceeds the rock mass strength.

46. One of the challenges for the TRP is to assess the strength of the right abutment. This is a difficult question because one cannot normally obtain an intact sample and test it in a shear test because the sample breaks down. We do test the strength of rock joints and that helps to give us an indication as to what the rock mass strength might be. My view at present is that the weakest link for the main part of the Dam is the strength of the RCC joints rather than the foundation. But the weak link for the right abutment may be the erosion ability and the strength of the soil-like conditions that the Dam is actually founded on. The geologists have been able to describe to us what is going on, but we have not reached the stage where we have been provided with the appropriate strength parameters to use in stability calculations.

47. The primary concern I have about a possible failure of the right abutment is erosion on the downstream side. In a flood situation, because the flow going over the Dam can be

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so large depending on the size of the flood, what I envisage is that the flow coming down the right abutment could be causing a very large gulley to form. The gulley could get some turbulence and rolling of water going and that could cut the hole at the toe of the Dam so that it starts cutting underneath the Dam. It could get to the point where there is not enough shear resistance in the Dam foundation interface and the Dam could slide or topple into the hole that has been created. That would be the main failure mechanism for the right abutment.

48. After TRP Report No. 2, GHD did a lot more work mapping on mapping the foundations and formulating their views. Much of this work was presented at the TRP meeting in November 2019. Mr Young was not able to attend that meeting, however GHD did have a skype call with Mr Young to present the same information to him. Mr Young supplied, as I understand it, some recent papers on ascertaining the strength of these very highly jointed tectonically altered melange-type rocks, which certainly exist under the spillway because of the presence of what is call the 'Paradise Fault' on the left abutment where the big erosion hole occurred in the past. Mr Young supplied me with notes to include in TRP Report No 3. Mr Young's points of particular note related to stability analysis are reproduced below.

    (a) **Rock Mass Classification and Strength Assessments:** As discussed in earlier TRP meetings, it is necessary to derive rock mass properties, including shear strength assessment, from the current study program. The use of Hoek-Brown strength parameters, including the Geological Strength Index (**GSI**), are endorsed by the TRP as appropriate for the foundation rock mass zone. The TRP recommend the latest Hoek-Brown GSI classification chart, which includes quantified block size parameters, be used. This will be used for relatively homogeneous layers and

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blocks of rock for input into geological models. It should be noted that soil mechanics methods may be needed for estimating properties of some soil-like gouge/breccia zones.

(b) **Weathered Rock in Right Abutment and Secondary Spillway Foundation:** The TRP understands that the physical properties of the extensive highly weathered zone are not well understood. The situation should be addressed as part of the ongoing works. New boreholes and laboratory testing of samples are recommended.

49. GHD is, I understand, in the process of bringing more of that information together in order to come up with their assessment of the rock and foundation strength. These numbers will then be incorporated into the stability analysis.

50. I saw some photographs of the right abutment prior to completion of the Dam which were forwarded to us in one of the SharePoint files, probably for the first TRP meeting. They showed that those building the Dam had dug test holes in the foundations using an excavator, backfilled them with concrete and then started placing the RCC on top. It immediately made me think, well, if you can excavate a trench into the material just with an excavator, it is probably not the sort of sound foundation you want to found a gravity dam on. Normally, you would take the foundations down to another level of rock that you could not excavate with the excavator before placing the RCC.

51. At some point it appears that Golder must have given some signature on a piece of paper to the designer to say they could start pouring RCC on this foundation. I do not know how they made that choice. I also have no knowledge of the decision making process in

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terms of looking at the implications of the faults in deciding the basin. I do not know what they were contractually required to do. A dam builder would normally engage a geotechnical engineer to advise on the size and extent of the apron.

52. By way of example, I worked in my early career on the Clyde dam in New Zealand, an 80-some metre high concrete gravity dam. In that case the Government contracted geotechnical scientists and technical advisors right through the whole project with engineers and geologists onsite advising what the defects in the foundation meant. They arranged testing of materials and strength that fed into the design process. That would be normal on a concrete gravity dam project. The dam designer does not just work in his office by himself coming up with the design. The dam designer should bring together all the skills of the geotechnical people, hydrologists, and (in New Zealand) seismologists that feed into developing the design. In Clyde Dam, we over-excavated some areas because of foliation shears in the foundations, which are seams of clay running through the rock. These were unfavourable from a stability point of view at Clyde dam. Once we got all the information to the chief designing engineer, our recommendation was to go down deeper and excavate more of the rock. The chief designing engineer then approved the work and we excavated more rock. The process should be ongoing. It does not finish when the investigations are done, it continues throughout the whole process of the construction phase because the first time you get the see the foundations in their glory is when you divert the river and you can clean the whole thing down and have a look.

53. One of the issues during the design of the Dam was the decision to leave what is called the 'basalt pimple' in place under a gravity block on the right abutment. This is a particular area where the foundation strength gets weaker at depth due to a stronger basalt flow overlying a weaker alluvial material. The alluvial layer underneath the basalt which

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has gravel soil-like properties could be a potential sliding plane weakness. In the paper that Richard Herweynen co-authored at the ANCOLD 2004 Conference [**PDO.035.0001**], he noted that the diversion channel was located such that the basalt to the right of the channel was totally removed, therefore eliminating any risk of instability in this section of the Dam foundation. However, a "subject to approval" decision was made to leave the basalt "pimple" in place. In Appendix I5 of the Detail Design Report [**GHD.002.0001 at 1499**], there are stability results presented from a dam analysis program CADAM for a dam section with the alluvial layer at elevation 36.5 metre in the model. A strength of 45 degrees is assigned to the material and a minimum sliding factor of safety of 1.66 is reported for the 1 in 1000 AEP flood.

54. There was also a GHD memo presented to the TRP, dated 22 February 2019, that reviewed the stability of the Dam undertaken by AECOM using the finite element model PLAXIS [**IGE.053.0001**]. The model considered the Dam as it was and also strengthened with ground anchors. One of the sections analysed included Block L that sits on the basalt pimple. AECOM assigned a strength to alluvial material of 35 degrees which, if used in the CADAM model above, would have reduced the 1 in 1000 AEP flood sliding stability to 1.2. When I looked at the section analysed by AECOM for block "L", I queried some aspects related to how the reservoir thrust was modelled down through the basalt to alluvial layer and how uplift was modelled along the alluvial plane at the base of the model. As the modelling was not done by GHD they were not in a position to answer my query. My current opinion is that there is not yet a satisfactory model to analyse block "L" sliding on the alluvial layer below the basalt pimple with agreed strength parameters and modelling of the reservoir and uplift loads.

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55. To my mind, the stability of Block L is still a work in progress that John Young and I need to address with GHD.

Geological testing and mapping

56. There is a lot of effort going into mapping the foundations, but it has not reached the stage where they have been able to turn that into numbers we could use in a stability calculation for the shear strength of the foundation zone immediately underneath the contact with the Dam body. John Young supplied a number of recent updates of papers covering the melange to GHD but they still have to get to the point where they generate some shear strength parameters. While we have some meaningful numbers for the RCC lift surface, we still have not received meaningful numbers between points across the foundation for the rest of the rock mass at the interface with the Dam.

57. By way of example, earlier in my career I had been involved in analysing tests done on Aviemore dam in New Zealand which has what is called a greywacke foundation which is also a hard sandstone rock disturbed with tectonic movement with close jointing (similar to the melange at the Dam). In the 1960s, they did some large scale in situ shear tests where they had an anchored test block built on to cleaned-down greywacke rock and then had a reaction lock behind it. They then jacked them apart and worked out what was the load that failed them (the shear strength). Work that I did with GNS later on indicated that the actual shear strength on those rocks was less than you were getting out of standard computer software using the Hoek-Brown procedures. It appears some disturbance factor for near surface rocks at the Dam interface should be used to reduce the strengths.

Witness Signature

~~Justice of the Peace /Commissioner for Declarations~~ /Lawyer

58. I made a cautionary note about that in TRP Report No 1. At that stage, John Young had not joined the team, so I was just alerting GHD and Sunwater to the fact that all these Hoek-Brown derived strength numbers that AECOM had used in their PLAXIS analysis did not have a lot of basis behind them. In fact, the ANCOLD guidelines stress that just using Hoek-Brown derived rock strength parameters to find the strength of the rock without doing tests on the joint strength and similar is not the way to determine well defined strength parameters.

59. The enquiry about the foundations strength parameters is ongoing. We do not yet have all the numbers we need for a reliable stability analysis, so we do not know whether the foundations are a failure mechanism to be concerned about, relative to the RCC lift joints shear failure mechanism.

60. There are also Canadian Dam Safety Guidelines updated in 2007 which talk about how to consider unbonded joints in the concrete and how to sample joints to get shear strength information. It recommends the use of horizontal investigation holes where the diameter of the core is at least 150mm. This is exactly what GHD did to get some of those cores we tested. Horizontal drilling is difficult, but GHD did get core where you can actually see the lift joints, so we are able to see the samples to do shear testing.

61. There is quite a lot in the ANCOLD guidelines about how you need to get sound strength parameters because you cannot use low factors of safety if you do not have well-defined strength parameters. I recall one job in New Zealand on a highly jointed rock foundation I worked on where they had an eminent rock consultant advising on it and he set out how the geologists should map the joints. They mapped all the joints, the extent, what percentage had rock bridges you could see, etc., and he produced some research papers

Witness Signature

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that suggested that the strength parameters could be based on the continuity ratio of the joints and its rock bridges. Unless you go through that process, it is very hard to justify how you say the strength parameters are well-defined. If they are not well-defined, then you have to use higher factors of safety to protect yourself from the uncertainty that still exists.

62. My current view is the weak lift joints in the RCC is the most likely to dominate all the possible failure mechanisms, except for erosion of the weaker materials on the right abutment if the secondary spillway ever operates. But the foundations may still have a strength, however, that does not meet the ANCOLD stability criteria for foundation stability.

RPEQ requirements

63. I do not know who was to fill the requirements of engineering works for the Dam and direct supervision of a suitably qualified engineer as required by the *Professional Engineers Act* 2002 (Qld). I have been in that role myself where I have signed off the design and construction reports as the RPEQ engineer. In that role, I ensured that the Stantec engineers on site during the construction were under my direct supervision and kept me informed with email correspondence and weekly reports on what was going on and how the quality assurance testing was going. I made sure I visited the site frequently during construction, maybe once every few months, so that I could form my own opinion and audit what was done to satisfy myself that everything was meeting the design intent.

Witness Signature

~~**Justice of the Peace /Commissioner for Declarations /**~~**Lawyer**

Commission of Inquiry
PARADISE DAM

OATHS ACT 1867 (DECLARATION)

I, Peter Francis Foster, do solemnly and sincerely declare that:

(1) This written statement by me dated 19 February 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 WELLINGTON **this** 19th **day of** FEBRUARY **2020.**

Taken By VICTORIA MARGARET TATAM

~~**Justice of the Peace / Commissioner for Declarations**~~ **/ Lawyer**

Victoria Margaret Tatam
Solicitor
Morrison Kent
Wellington

"PF1"

Stantec

PETER FOSTER

Internal Review Panel

Peter has over 35 years' experience in dam design, dam safety reviews and rehabilitation. He has specialised in dam and hydraulic structures engineering and as a result of his specialist knowledge is frequently sought after as an independent expert reviewer.

Peter has worked on major dam projects including the design of the $1b 102m high Clyde Dam, the largest concrete gravity dam in New Zealand. This project ran for over 20 years and Peter is still providing safety inspections of the dam today, exemplifying his on-going commitment to a client project. He has also worked on the design of the Ross River Dam upgrade and Wivenhoe Alliance to provide a fuse plug spillway. His on-going work in Australia continues to expose him to current practice and modern dam design work with Fairbairn and Boondooma spillway upgrade projects for SunWater being recent examples.

He is a member of the New Zealand Society on Large Dams and is a past chairman This role required Peter to participate in many dam conferences around New Zealand and Australia and represent NZ interests at international forums. Peter has assisted ANCOLD as an expert review panel member for their Guidelines on Design Criteria for Gravity Dams (2013)

Peter makes regular trips to Brisbane to support the Brisbane dams team and has been the certifying RPEQ on 25 CSG Brine Storage dams, mentoring a local and international design team of up to 8 engineers on these works at any one time.

Since 1987, he has published 16 technical papers in New Zealand and internationally and now mentors the next generation of dam designers.

EDUCATION AND MEMBERSHIPS

- BE / BEng, Civil Engineering (Hons)
- New Zealand Society on Large Dams (Past Chairman)
- Engineering New Zealand, Fellow
- Chartered Professional Engineer – New Zealand
- Registered Professional Engineer Queensland (RPEQ)
- New Zealand National Society of Earthquake Engineering – Member
- New Zealand Geomechanics Society - Member

PROJECT EXPERIENCE

Burdekin Falls Dam Improvement Project and Dam Raise Project
SunWater, 2019
Technical Lead to develop concept designs for a dam improvement project to increase flood capacity by stabilising existing dam with buttress or post-tension anchors and also to incorporate the solution into either a 2 m or 6 m raise of full supply level at the dam.

Paradise Dam Upgrade
SunWater, 2019
Technical Review Panel member to review options for safety improvements at Paradise Dam to ensure a lowering of the risk profile to meet ANCOLD tolerable risk criteria.

Wivenhoe Dam Upgrade Project
SEQWater, 2018 to 2019
Technical Review panel member to review options for upgrading the spillway capacity at Wivenhoe dam incorporating potential dam raise options from 0 to 4 metres for extreme floods.

Burdekin Falls Dam Upgrade
SunWater, 2017 to 2018
Technical Lead to complete preliminary design of safety upgrade at Burdekin dam to improve flood capacity using post-tension cables to stabilise the dam.

Boondooma Spillway Upgrade
SunWater, 2016
Design Review and Assistance to SunWater Design Team to develop tender design to protect the spillway chute from rapid rock erosion and stabilise spillway crest under extreme flood loads.

Fairbairn Spillway Upgrade
SunWater, 2015 to 2020
Design Assistance to SunWater Design Team to develop concepts and detailed design solutions to enhance the stability of spillway chutes and stilling basin at Fairbairn Dam. Provision of technical advise during construction.

Warragamba Dam Raising Study
Infrastructure NSW, 2014 to 2015
Assessment of options to raise the dam by 14 m to provide flood detention benefits, stability analysis, flood routing. Development of a preferred option for detailed costing.

ANCOLD Gravity Dam Guidelines
ANCOLD, 2011 to 2013
Expert Review Panel Member through the period of development and finalisation for the ANCOLD Guidelines on Design Criteria for Concrete Gravity Dams.

PETER FOSTER

Internal Review Panel

APLNG Ponds
Origin Energy, 2011 to 2018
Certifying Engineer for design and construction of over 25 regulated ponds (3 Ponds ongoing) to store Brine and CSG associated water.

Baleh and Baram Dams Feasibility Review
Sarawak Energy Berhad (SEB), 2011 to 2012
Dam layout optioneering, spillway sizing, flood routing, and stability review for CFRD and RCC dams in height range 150 to 200 m.

Murum Dam Technical Advice
Sarawak Energy Berhad (SEB), 2010 to 2011
Technical advice on dam foundation stability, grout mix trials for 144 m high RCC dam.

Nadarivatu Hydro Project
Fiji Electricity Authority (Fiji), 2009 to 2012
Peter had a role as Owners Representative for the design and construction of the 32 metre high concrete weir structure which contains three spillway gates and a gated sluice structure.

Catagunya Dam Upgrade
Hydro Tasmania, 2007 to 2010
Peter was part of a review panel providing ongoing review services for the Catagunya gravity dam stability upgrade. He gave specialist advice on analysing the stability of the dam with post-tension cables which included the consideration of dam foundation failure mechanisms.

Lake Manchester Dam Upgrade
Brisbane City Council, 2006 to 2007
Peter was part of a review panel for the stability and flood capacity upgrade of the Manchester concrete gravity dam. He was involved in a stability review of the dam and foundations, a raised crest and post-tensioned cables to ensure stability at extreme flood loads. Peter gave specialist advice on establishing the design criteria for the upgrade.

Ross River Dam Upgrade
NQ Water, 2004 to 2006
As Design Engineer, Peter was responsible for the upgrade of the Ross River Dam. The upgrade included providing gates to the spillway and enhancing the capacity of the stilling basin. Peter was responsible for the spillway gate operating rules, the concept design for spillway gate options, the spillway blocks stability review and design of post-tensioning requirements, the hydraulic model testing of stilling basin upgrades, the design of structural upgrade for stilling basin and sidewalls as well as the seismic analysis for spillway piers. The spillway and stilling basin can now accommodate flows that are 3.5 times higher than originally designed for and the spillway stability has been enhanced with post-tension cables for extreme flood loads. The project was successfully delivered with the lake returning to full supply level in February 2008.

Wivenhoe Spillway Alliance
Seqwater, 2003 to 2005
Expert Review Panel Member through the period Peter was a member of a Peer Review Panel for Wivenhoe Alliance. The project involved providing additional fuse plug spillways, modifying spillway gates and a stability upgrade of existing spillway blocks using post-tensioned cables.

Clyde Dam
Electricity Corporation of NZ, 1977 to 1990
As Design Engineer, Peter was responsible for developing conceptual layouts for the $1B, 102m high Clyde gravity dam as well as initial investigations, right through to the completion of the structures. Peter was involved in foundation design and remedial works, including slip joint on fault through dam foundations, together with any aspects of civil design for the dam structure.

"PF2"

COMMISSION INTERNATIONALE
DES GRANDS BARRAGES

Treizième Congrès
des Grands Barrages
New Delhi, 1979

Q. 49
R. 31

TENSILE FAILURES IN SOME STONE MASONRY GRAVITY DAMS IN INDIA (*)

Y. K. MURTHY
Ex-Chairman, Central Water Commission

P. M. MANE
Consulting Engineer, Ex-Member (Designs and Research), CWPC

B. PANT
Joint Director, Central Water and Power Research Station

INDIA

I. INTRODUCTION

Hand laid stone masonry in lime or cement mortar has been extensively used for construction of dams in India. Deployment of large labour force without increasing total cost of the project has a great social value under Indian economic conditions. In contrast to cement concrete, however the structural behaviour of stone masonry and its failure mechanism is not well understood [1]. One of the major hand laid stone masonry dams constructed in recent times is the 114 m high Nagarjunasagar dam [2] in Andhra Pradesh. Although most of the old masonry dams, even though of a very slender section from modern concepts, like the Khadakwasla dam [3], have shown a remarkable performance, some others have gradually deteriorated or shown signs of distress. In this paper, case histories of three recent failures or distress in such handlaid masonry dams are examined, based on which some general precautions during design, construction and operation of stone masonry dams are suggested.

(*) *Fissures de traction dans des barrages-poids en maçonnerie en Inde.*

II. FAILURE OF KHADAKWASLA DAM

Situated 16 km from Pune, the Khadakwasla dam is one of the oldest masonry dams in India. It was constructed in 1870 to protect Pune District from famine. With a storage capacity of 90.5 hm^3 the dam is constructed in uncoarsed rubble masonry in lime mortar. The foundation consists of sound basalt rock. It has a trapezoidal section and has a maximum height above the deepest foundation of 33 m and a base width of 17.2 m, ie. 52 % of the height. The cross section and the longitudinal section of the dam are shown in Figure 1. No drainage gallery or other drainage arrangements

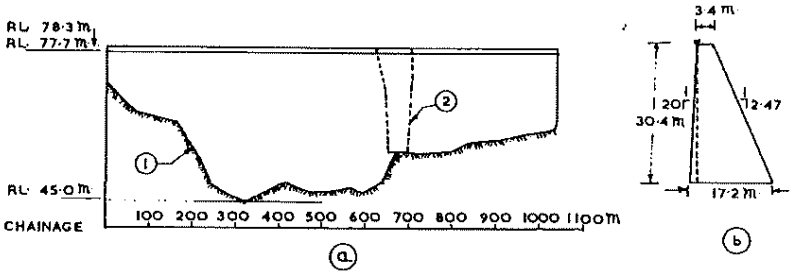

Fig. 1

Khadakwasla dam. Elevation and section.

Barrage de Khadakvasla - Élévation et coupe.

(a) Longitudinal elevation.
(b) Section at chainage 568 m.
(1) Foundation level.
(2) Breached portion.

(*a*) *Élévation.*
(*b*) *Coupe à 568 m.*
(*1*) *Niveau de la foundation.*
(*2*) *Zone de rupture.*

have been provided nor has uplift been taken into account in the design. The dam has been constructed in one continuous length without any transverse joints. No cracks were noticed in the dam. Calculations based on 100 % uplift at the heel going down to tail water level at the toe indicated tensile stress at the heel of the order of 3.9 kg/cm^2. The dam was already unstable according to the present day criteria. Several alternative proposals had been examined in the past to strengthen the dam. These included prestressing by cables, addition of downstream buttresses in masonry and backing by earth. There was also a proposal to construct an entirely new dam downstream of the existing dam. The deeper sections of the dam had been strengthened by earth backing.

While the investigations on these proposals were in progress, on 12th July 1961, in the early hours of the morning, the Panshet dam which was an earthfill dam under construction, upstream of this dam, failed due to subsidence followed by overtopping. The large amount of discharge of the order of 200 hm^3 released by this failure flooded the Khadakwasla dam and overtopped it by more than 2.7 m. The dam which was already consi-

dered unstable was thus loaded with extreme loads for which it was not at all designed. However, inspite of these heavy loads, it resisted the floods for a number of hours and finally failed in tension and shear. The earth backing of the dam had been completely washed off and water level exceeded the design level by 4.27 m. At the time of failure, eye witnesses had stated that the dam was vibrating and opened like a single door leaf with a hinge on the right side. The leaf finally toppled over. This is apparent from the photograph of the breached section, given in Figure 2.

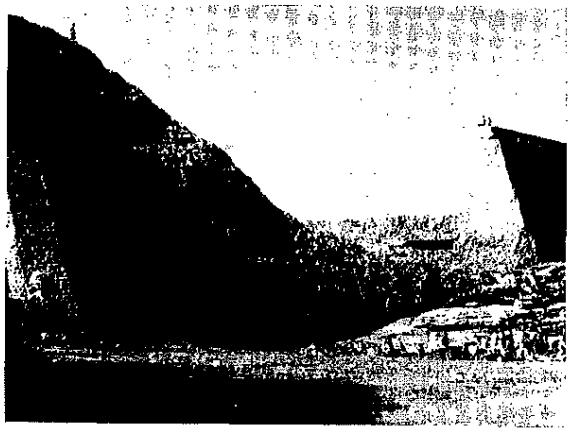

Fig. 2

Khadakwasla dam - Breached section.

Barrage de Khadakvasla. Zone de rupture.

The failure showed that the strength of masonry [1] which had a density of 2.4 kg/m^3 even after failure could be quite high. Tests carried out on 91 cm diameter cores (91 cm in height) gave an average compressive strength of 120 kg/cm^2 and a split tensile strength of 6 kg/cm^2. The comparative values of compressive strength of the basalt rock are 250 to 1 000 kg/cm^2 and for the mortar 50 kg/cm^2. The average tensile strength of mortar was 0.4 kg/cm^2.

Although there was nothing unexpected in Khadakwasla dam failure, as the dam was never designed for the extreme loads to which it was subjected, the actual mechanism of failure throws considerable light on the behaviour of masonry dams. The most unusual feature was that the failure had occured not at the maximum section of the dam where the computed stresses by conventional methods of analysis were the highest but at a location where the height was only 2/3 of the maximum. To make a realistic analysis of this condition, detailed three dimensional photoelastic tests on a 1/600 geometrically similar model were carried out simulating about 170 m of rock and dam profile in the failure region. Some of the stress patterns

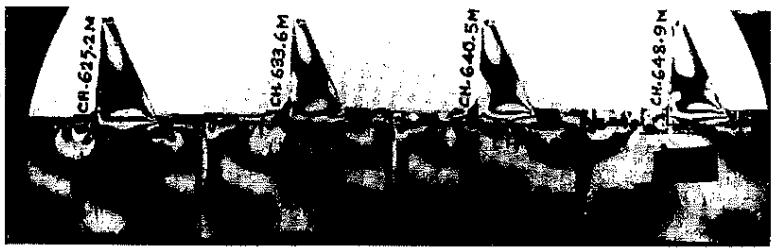

Fig. 3

Khadakwasla dam -
Photoelastic stress patterns in sections between chainage 620 m and 650 m.
Barrage de Khadakvasla. Étude photoélastique à 620 m et 650 m.

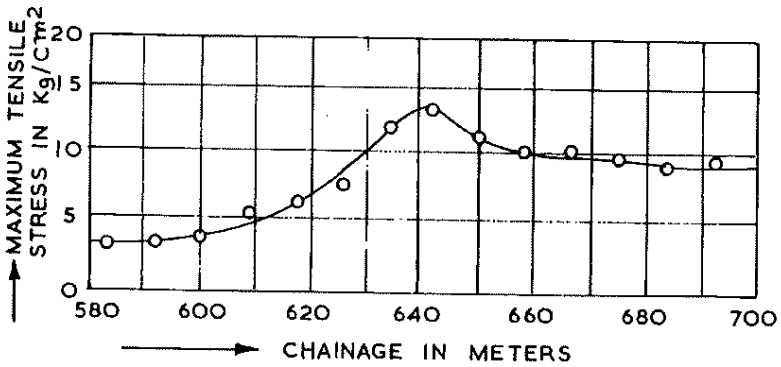

Fig. 4

Khadakwasla dam -
Photoelastic test results. Maximum tensile stress at foundation.
Barrage de Khadakvasla. Étude photoélastique. Traction maximale dans la fondation.

obtained after stress freezing under simulated water load and slicing the model are shown in Figure 3. The distribution of the maximum tensile stresses along the foundation level is shown in Figure 4. This shows that a peak tensile stress develops at the location where there is a sudden change in elevation in the foundation. Three dimensional analysis thus indicates correctly the location of most vulnerable region in the dam. The mode of failure of the dam could be visualized as follows.

A tensile crack in masonry had formed at the location of the kink near the foundation where the resultant stress excluding the effect of uplift was of the order of 10 kg/cm^2. Once this failure has started, the crack tends to progress along the base, thereby increasing the uplift. In the process more load is thrown on the vertical face at the kink. This section, therefore, is subjected to increasing shear stresses on near vertical planes and associated tensile stresses on the upstream face due to horizontal beam bending effect

and ultimately a stage is reached when a vertical crack is formed at this location due to tensile and shear failure. In the next stage the rectangular portion of the dam without support on one side bends as a plate and high tensile stresses are developed on the upstream face at plane inclined about 45° to the horizontal. The dam was noticed to have been vibrating for quite some time before it was over-topped, the vibrations being evidently due to suction of the air from behind the falling nappe. These vibrations may have accelerated the progress of the tensile cracking in the masonry near the foundation of the dam.

III. DISTRESS IN BHANDARDARA DAM

Completed in 1926, the Bhandardara dam [4] [5] is one of the oldest and highest stone masonry dams in the Maharashtra State. Bhandardara

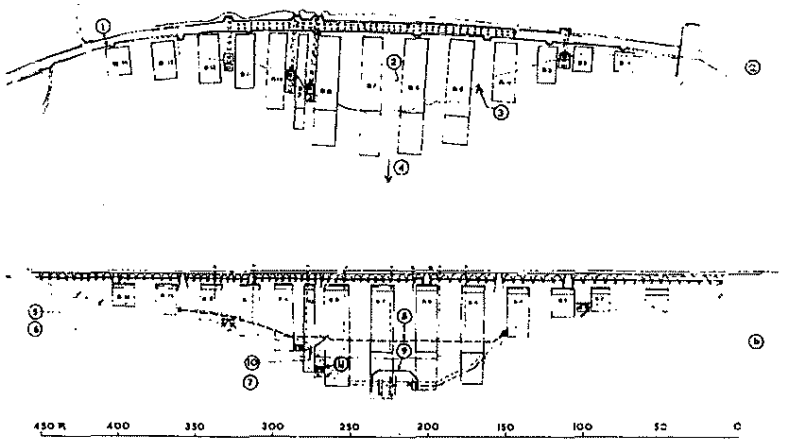

Fig. 5

Bhandardara dam - Plan and elevation.

Barrage du Bhandardara. Plan et élévation.

(a) Plan.
(b) D/S elevation.
(1) Roadway.
(2) Adit.
(3) Toe of existing dam.
(4) Flow.
(5) Ground line.
(6) Rock line.
(7) Proposed drainage gallery at foundation level (Not yet excavated).
(8) Cracks on upstream face.
(9) Vertical cracks on downstream face.
(10) Cracks on side walls.
(11) Drain pipe of plumb bob well, jet on 10-9-69.

(a) Plan.
(b) Vue d'aval.
(1) Chaussée.
(2) Fenêtre.
(3) Pied du barrage.
(4) Écoulement.
(5) Terrain naturel.
(6) Rocher en place.
(7) Galerie de drainage proposée au niveau de la fondation (pas encore creusée).
(8) Fissures sur la face amont.
(9) Fissures verticales sur la face aval.
(10) Fissures sur les murs latéraux.
(11) Tuyau de drainage du puits du pendule.

"PF3"

U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Concrete Dams
Case Histories of Failures and Nonfailures with Back Calculations

U.S. Department of the Interior
1849 • 1999

DSO-98-05

Structural Analysis

December 1998

DSO
DAM SAFETY OFFICE

Concrete Dams Case Histories of Failures and Nonfailures with Back Calculations
DSO-98-05

by
Chuck Anderson
Caroline Mohorovic
Larry Mogck
Bitsy Cohen
Gregg Scott

U.S. Department of Interior
Bureau of Reclamation
Dam Safety Office

December 1998

Table of Contents

1.0 Introduction

This compilation of case history summaries is intended to assist risk analysis teams in estimating probabilities related to concrete dams. Much can be learned examining the successes and failures of other projects. What makes this compilation unique and hopefully more useful is the back calculations of strength and response which are provided. If a case history can be found in this compilation that is similar to a dam being considered in a risk analysis, then the information may be useful in making reasonable estimates.

Case histories can provide valuable insight for identifying failure modes and for breaking them down into sequences of events. Failures, and incidents that did not lead to failure are equally valuable, and provide information about what happened at other dams. This information provides the means for conceptualizing and specifying the occurrences, conditions, and interventions that could be pertinent to the dam under consideration. In addition, back analysis of these situations provides valuable insights when interpreting the results of analyses for the dam under consideration.

2.0 Sliding of Gravity Dams

The following summaries describe sliding failures of concrete gravity dams. In most cases sliding occurred in the foundation along weak horizontal planes. In some cases complete failure occurred. The dams summarized here (in order) are:

Austin Dam, Pennsylvania
Austin Dam, Texas
Bouzey Dam, France
Upper Stillwater Dam, Utah
Morris Shepherd Dam, Texas

CASE HISTORY SUMMARY

Name of site or structure: Austin Dam, PA (Cyclopean concrete gravity)
Location: Austin, Pennsylvania, USA
Type of event: Structural failure with foundation sliding
Date of event: First event 1910; second event 1911
Date of construction (if applicable): Started in May, 1909, and built very rapidly with completion on December 1, 1909.

Loading: Dam failed under static loading accompanying full reservoir

Description of site, structure and materials: Austin Dam (sometimes referred to as Bayless Dam) was constructed 1½ miles (2.4 km) above the town of Austin, Pennsylvania by the Bayless Pulp & Paper Company. The reservoir had a capacity between 550 and 850 acre-feet (0.68×10^6 and 1.05×10^6 m^3). The dam was constructed on horizontally bedded sandstone with interbedded layers of shale and disintegrated sandstone. Very few vertical joints were present. A tight gravel deposit overlying the bedrock was excavated with difficulty, and the dam was founded on the first solid stratum at least 2 feet (0.6 m) thick. The surface of the rock was well washed and grouted. A concrete cutoff key, 4 feet (1.2 m) thick and 4 feet (1.2 m) deep, was constructed at the upstream heel. Holes were drilled into the foundation rock from 5 to 8 feet (1.5 to 2.4 m) deep at 2.66-foot (0.81-m) centers along the cutoff trench, and 1-1/4 inch (32 mm) diameter steel rods, 25 feet (7.6 m) in length, with expansion head anchors were installed and grouted. The concrete gravity dam was 43 feet (13.1 m) high and 534 feet (162.8 m) long. It was built very rapidly without alternating block placements and some of the later placements were in freezing weather. However, the majority of the concrete was found to be strong and competent. Local sandstone was crushed and screened on site to provide sand for the concrete. This resulted in excessive fines in the sand which tended to float and cause a "cream" at the surface of placements in some areas. The construction used Cyclopean concrete with large quarry stones from ½ to 2½ cubic yard (0.4 to 1.9 m^3) size embedded firmly in wet concrete so as to lie across the lift lines. Cracks, approximately 1/16th inch (2 mm) wide, formed 50 feet (15.2 m) west and 40 feet (12.2 m) east of the spillway. The cracks were attributed to shrinkage since no water was yet impounded. Construction was completed on December 1, 1909.

Behavior under loading: Rain and rapidly melting snow during a brief warm period the week of January 17, 1910, following a very cold and snowy December, filled the reservoir and caused a significant spillway flow. An undetermined thickness of ice still covered the reservoir. On January 22nd, a landslide dropped about 8 feet (2.4 m) on the eastern bank downstream of the dam, and water leaking from beneath the slide was evidently from the reservoir. Water in large quantities began coming up through the ground from 15 to 50 feet (4.6 to 15.2 m) downstream from the toe of the dam. On January 23, 1910, the center portion of the dam slid horizontally downstream as much as 18 inches (0.46 m) at the base, with a corresponding crest displacement of as much as 31 inches (0.79 m). A photo, taken from the eastern abutment and aligned with axis of the dam, shows a distinct bend along the length of the dam (figure AP-1). Five or six

large vertical cracks through the dam formed at this time. Two 8 to 10-foot (2.4 to 3.1 m) gaps were blasted in the crest of the dam, one near the spillway and one near the right abutment to lower the reservoir and reduce the loading. After a review of the structure, engineering consultants analyzed the situation (see figure AP-2) and advised strengthening the dam. The recommendations went unheeded and the dam was put back into service without the recommended repairs. Subsequent photos show significant leakage spilling through the large vertical cracks in the dam.

Rainfall in September 1911 was unusually heavy. Water began flowing over the spillway for the first time in about 20 months. On September 30, 1911 (a warm sunny day) water was flowing about 7 inches (0.18 m) deep over the crest when the dam suddenly gave way (see figures AP-3 and AP-4). Eyewitness accounts indicate that a plug shaped section near the base of the dam about 100 feet (30.5 m) from the right abutment blew out and water surged through the opening. Other large segments of the dam immediately moved downstream or swung open like a gate. Sections of the foundation were still attached to the base of some dam remnants, indicating that sliding had occurred along weak planes in the foundation under some segments. At about 2:00 p.m., Harry Davis, who observed the failure from a boarding house on a mountain slope near the dam, phoned the Austin telephone operators to sound the alarm. The paper mill whistle was sounded in response to a phone call from the telephone operators. Unfortunately, the mill whistle had blown twice earlier in the day when false signals had been received from telephone company employees who had been repairing the lines. Many people did not heed the warnings and the flood wave arrived in Austin with deadly results.

Consequences: Austin was a town of about 2300 people and most of the town was in the narrow valley below the dam. The water traveled from the dam to the town in about 11 minutes. The flood swept everything away except a few brick buildings and houses above the crest of the flood wave (see figures AP-5 and AP-6). There were a total of about 78 fatalities, all of which occurred in the Austin area. The flood wave had dissipated by the time it reached the town of Costello, 3 miles (4.8 km) below Austin, and no additional deaths occurred downstream.

Back Calculations: Stability calculations were performed for the dam simulating conditions at the time of failure as shown on figure AP-7. Since the dam had no foundation drains, an uplift distribution varying linearly from reservoir head at the heel to tailwater head at the toe was assumed to occur near the base of the structure. The 4 feet by 4 feet (1.2 by 1.2 m) shear key at the upstream heel is not shown. With a water surface at the crest elevation of the spillway, stability calculations show that tensile stresses occurred at the heel of the dam. With uplift, the maximum tensile stress is about 26 lb/in^2 (0.18 MPa), with the linear stress distribution going to compression at a distance of about 9.7 ft (2.96 m) from the upstream face. If it is assumed that full uplift penetrates to the crack tip and that any tensile stress would cause cracking, the calculations show the crack would continue to propagate all the way through the dam. This may not be entirely realistic since the materials would have some tensile strength, and uplift would likely dissipate through joints in the foundation rock. The dam was built with horizontal construction joints, with little attention given to joint preparation between lifts, but there is

somewhat conflicting information about the condition of the lifts. An investigative report on the causes of the failure, prepared by consulting engineer Walter Sawyer states:

> "Examination of the concrete ... indicates that it must have been deposited in a very wet state. Every concrete worker knows that after such concrete has stood for a short time a pulpy, gelatinous mass rises to the top. This substance, called 'laitance,' sets slowly, has little or no hardening or bonding properties, and but little strength; in fact, not much more than ordinary chalk....In my opinion, this 'laitance' was not removed from the horizontal joints at the Austin dam. Each line of fracture, which can be seen, took place through this soft material, and layers of it can be seen at many of the joints on the dam where no movement took place."

However, other accounts indicate, "These surfaces, however, seem to be quite hard and show evidence of having set to some extent before the overlying concrete was placed." The concrete was probably quite variable. In any case, the weak shale layers in the foundation and horizontal lift lines in the structure were probably much weaker in tension and shear than the parent concrete.

The 25 foot (7.6 m) long anchor bars grouted 5 to 8 feet (1.5 to 2.4 m) into the foundation and extending well up into the dam at 2.66 foot (0.81 m) centers would have resisted some of the tensile stress (from Sawyer's report):

> "In the dam, as constructed, 13-in. steel rods were said to have been located 2 ft. 8 in. on centers and about 6 ft. From the upstream face of the dam. These steel rods were placed in holes drilled into the ledge and were supposed to be secured by means of expansion nuts. Two of these bolts in the bottom of the overturned pieces from the dam had the expansion nuts attached, force enough having been exerted to pull these bolts and nuts from the ledge. I found other bolts which had been reduced in section and finally broken."

The reinforcement bars were described as "twisted steel" indicating they were probably square bars twisted in a spiral fashion to provide some roughness. The "breaking strength" of the rods is reported to be 52 kips (0.23 MN) each for a total tensile resistance of about 19.5 kips per foot (0.28 MN/m) of dam. The actual average tensile force from the base pressure diagram would only be about 18 kips per foot (0.26 MN/m). Thus, it is likely that the bars were able to resist cracking at the base of the dam and transmit the tensile stress into the foundation below the shear key. The primary cause of failure being reported as sliding on weak shale layers in the foundation therefore appears to be reasonable. If it is assumed that only the portion of the base that is in compression would contribute to sliding resistance and that the foundation planes daylighted near the toe of the dam, then a friction angle of less than 41 degrees would indicate sliding of the two-dimensional section. It is not known if the shale layers daylighted. It could be that the shale layers were weaker than the 41 degree friction angle, and this combined with rupture of a small amount of passive rock resistance caused failure.

Discussion: The failure of the owners to strengthen the dam after the initial movement was the subject of much discussion in the articles written. Based on a photograph taken in January 1910, showing rather dramatic bending along the dam axis and tipping downstream at the center, it is amazing that no corrective action was taken and the dam was allowed to be put back in service.

It is likely that the initial movement in January of 1910, in which the top-center portion of the dam moved 31 inches (0.79 m) downstream and the bottom-center moved 18 inches (0.46 m), started as tipping and subsequent sliding due to large lateral forces at the crest due to high reservoir and ice loading. With water flowing from springs below the dam and unstable wet areas on the abutment, it was apparent that reservoir water had entered the foundation. The resulting bending along its length left the dam broken at several places but amazingly, the pieces remained in place while the reservoir was drained and the dam inspected. The anchor bars were perhaps the only thing holding the dam in place.

When the dam was returned to service, without performing the strengthening recommended by a consultant, it was only a matter of time before critical loads returned to destroy the weakened dam. That finally happened on September 30, 1911, when rain brought the reservoir level 7-inches (0.18m) above the spillway crest. According to Harry Davis's eyewitness account, a huge plug, 4 or 5 feet (1.2 or 1.5 m) above the base and west of the center segment, burst forth from the dam. The portions of the dam above and beside the plug then tumbled into the ensuing flood. East of the spillway one huge segment slid forward at a slight angle and another pivoted like a hinged gate opening to nearly 45-degrees. The description of this sequence of events leads one to recall the excessive fines that formed a "cream" at the surface of some placements in the same area as the blown out "plug". The compressive strength of some samples taken from this area were as low as 49 to 98 psi (0.34 to 0.68 MPa), so low that they had to be handled with great care to keep them from crumbling. A combination of weak horizontal planes in the dam and foundation, and the vertical cracks from bending could have produced a block which was susceptible to being blown out. Once the pieces of the dam began breaking loose, each moved independently according to the flood forces and foundation resistance present. Many of these huge segments stand nearly in place and upright to this day.

An Austin Dam Memorial Association was founded in Austin, PA 16720 and still operates today.

References: "The Destruction of the Austin Dam,"Engineering Record, October 7, 1911, Vol. 64, No. 15, pp. 429-435.

Other articles were written, but most seem to rely heavily on the information contained in the above reference.

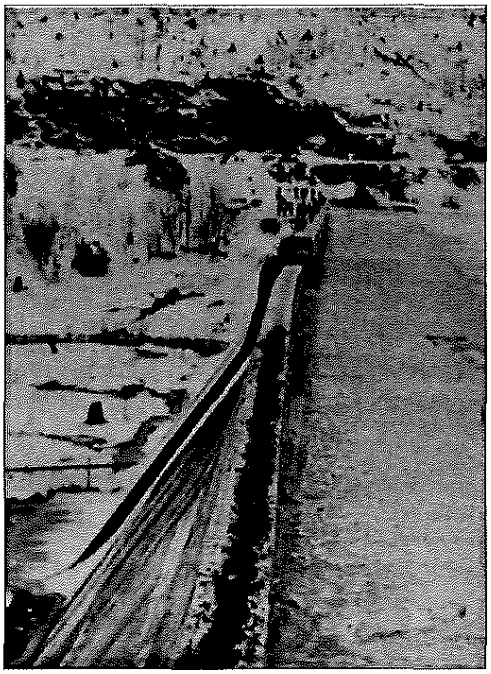

Figure AP-1. Photo showing bulge in crest during initial filling (after Potter County Leader, 9-24-1986)

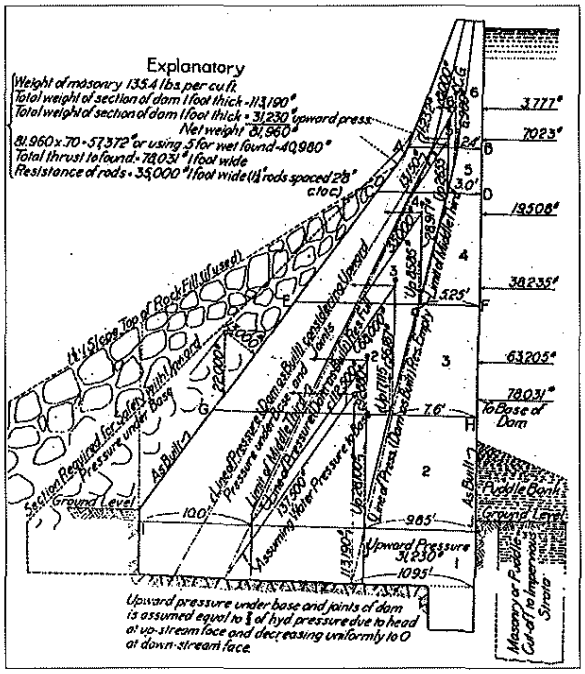

Figure AP-2. An analysis of the dam by Edward Wegmann, Jr. (After Engineering Record 10-7-11)

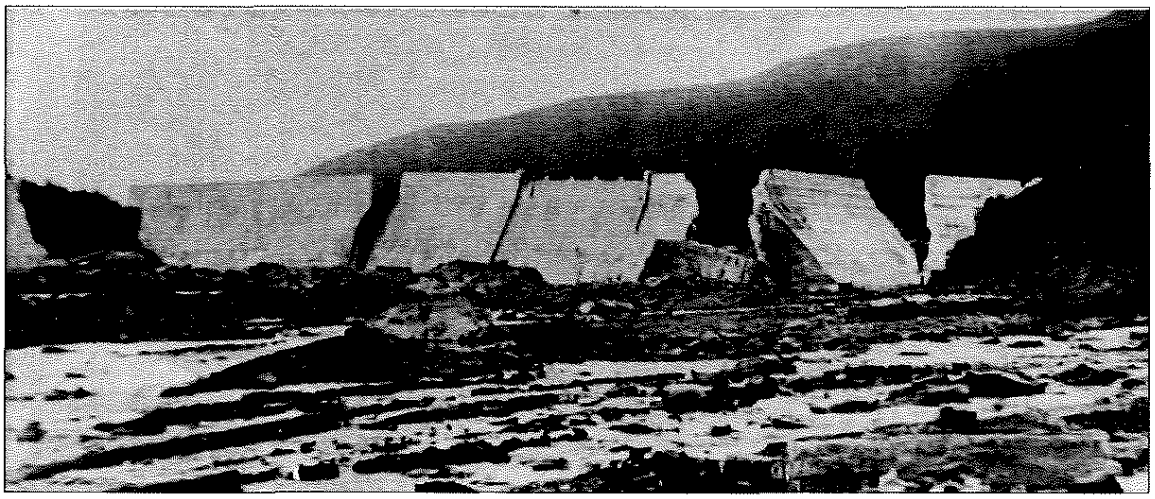

Figure AP-3. Photo showing downstream area after dam failure (after Potter County Leader 9-24-1986)

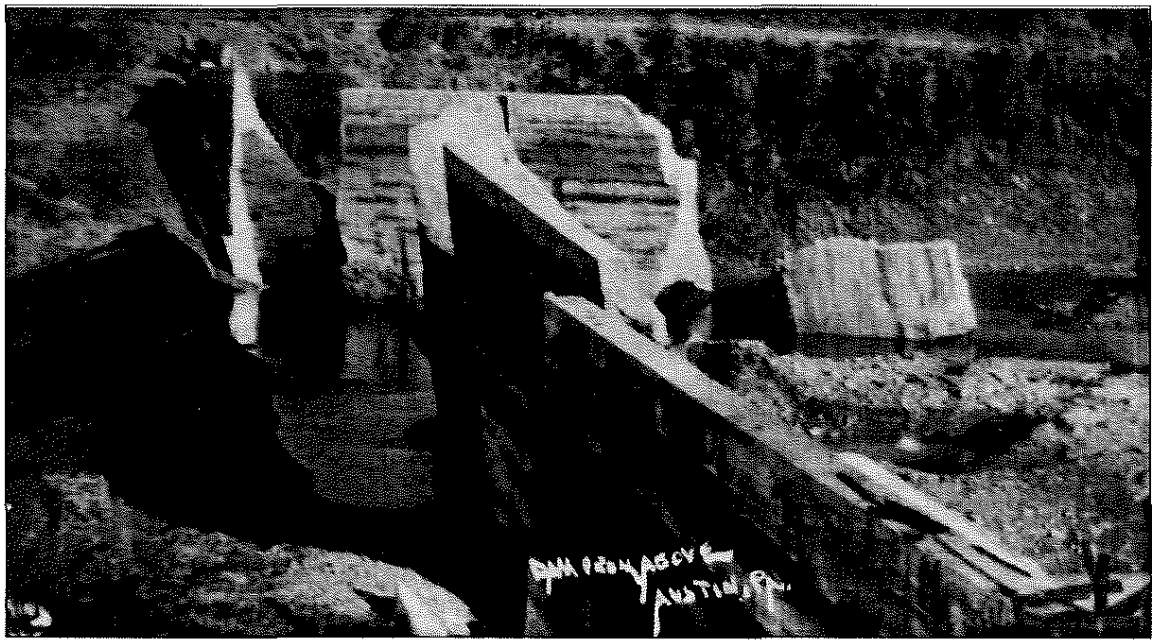

Figure AP-4. Photo of dam ruins after failure (after Potter County Leader 9-24-1986)

Figure AP-5. Town of Austin, PA before failure (after Potter County Leader 9-24-1986)

Figure AP-6. Photo of Austin, PA after failure (after Potter County Leader 9-24-1986)

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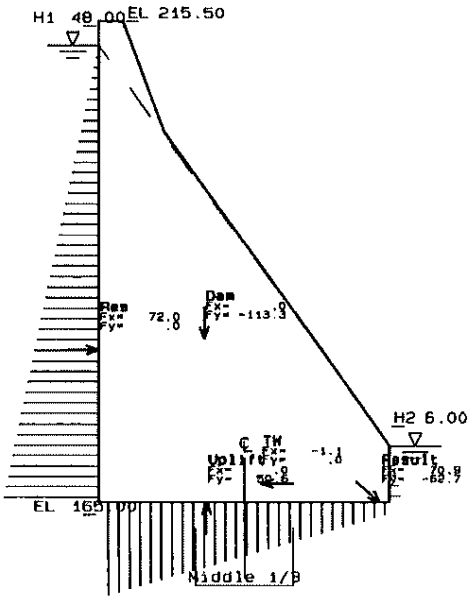
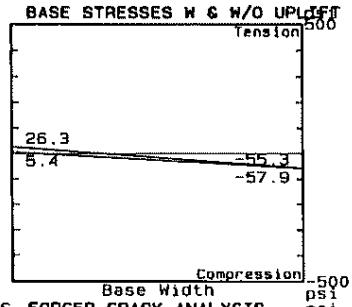
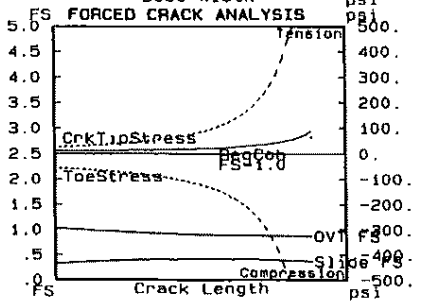
AUSTIN DAM





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DAM DIMENSIONS
 Analysis type                    Standard
 Criteria at drains               Corps
 Top of dam elevation             215.500
 Base of dam elevation            165.000
 Thickness of crest                 2.500
 Base width                        30.000
 Reservoir elevation              213.000
 Tailwater elevation              171.000
 Silt elevation                      .000
 Initial crack length                .000
 Drain distance from axis            .000
 Drain distance from heel            .000
 Drain efficiency                   1.000
 Drainage gallery elevation          .000
 Head at H1, Heel                  48.0000
 Head at H2, Toe                    6.0000
 Head at H3, Drain                 48.0000
 CORPS  H3=H1

MATERIAL PROPERTIES:
 Density of concrete              .13500 K/ft3
            water                 .06250 K/ft3
            horiz sat silt         .085 K/ft3
            vert  sat silt         .120 K/ft3
 Cohesion:   Break bond            .000 lb/in2
             Apparent              .000 lb/in2
 Friction:          Bonded         .364    20.0 deg
  (tangent&angle)   Unbonded      1.000    45.0 deg
 Fraction of area bonded          1.000

CG ABOUT 0,0,  FORCE & MOMENT ABOUT CL UNCRACKED BASE
Desc   ForceX  ForceY     CGX     CGY MomentX MomentY
Dam       .0  -113.3      4.1    17.2       .0  -465.4
Res      72.0      .0     15.0    16.0   1152.0      .0
Tail     -1.1      .0     -1.7     2.0     -2.3      .0
Uplt      .0     50.6      3.9      .0       .0   196.9

FORCES                   w/uplift   w/o uplift
Horizontal (+=d/s )        70.88      70.88 K
Vertical   (-=down)       -62.72    -113.34 K
Resultant                  94.64     133.68 K
  Location from CL         14.05       6.04
  Distance 1/3 base         5.00
  Distance 1/2 base         7.50
MOMENTS
 15.00 from heel     881.22     684.35 k-ft
OVERTURNING SF =        1.03

VERTICAL STRESSES AT HEEL AND TOE (Tension is positive)
                         w/uplift     w/o uplift
Area of base, A                         30.00 ft2
Moment of inertia, I                  2250.00 ft4
Moment arm, c                           15.00 ft
Axial stress, P/A        -14.52       -26.24 lb/in2
Moment stress, Mc/I       40.80        31.68 lb/in2
Vertical stress at heel   26.28         5.45 lb/in2
Vertical stress at toe   -55.31       -57.92 lb/in2

MINIMUM COMPRESSIVE STRESS AT HEEL FOR NO CRACKING
szu = pwh - ft/s
Drain factor, p=  1.000
Safety factor, s=   1.00
  Tensile      szu          Heel
  Strength   Allowable    Condition
   lb/in2     lb/in2
      .00      20.83      Cracked
    25.00        .00      Cracked
    75.00        .00      Cracked
   125.00        .00      Cracked
   175.00        .00      Cracked
   225.00        .00      Cracked

SLIDING FACTORS OF SAFETY
  Driving force              70.88 kips
  Total vertical forces     -62.72 kips
  Safety Factor =              .32 for above strengths
  Cohesion(Bonded) = 11.12 psi req for FS= 1.0
Sliding safety factors for various cohesions:
BreakBond-psi      .0    50.0   100.0   150.0   200.0   250.0
Resisting-K      22.8   238.8   454.8   670.8   886.8  1102.8
Safety factor     .32    3.37    6.42    9.46   12.51   15.56
Required strengths to get these safety factors while
keeping other given values constant    1.0     2.0     3.0
     Bonded area:      cohesion (psi)   11.1    27.5    43.9
                 friction angle (deg)   48.5    66.1    73.6

FORCED CRACK RESULTS
Heel initially in tension
Crack extends entire width of base
```

Figure AP-7. Calculations simulating conditions at time of failure

CASE HISTORY SUMMARY

Name of site or structure: Austin Dam, TX (masonry gravity)
Location: Austin, Texas, USA (298 river miles (479 km) from the Gulf of Mexico)
Type of event: Foundation sliding failure and spillway gate failure
Date of event: First event 1900; second event 1915; third event 1935
Date of construction (if applicable): Original construction completed 1894

Loading: The failures in each case occurred during massive flood events.

Description of site, structure and materials: Austin Dam was constructed on the Colorado River of Texas about 2.5 miles (4 km) upstream of Austin. The dam was 65 feet (19.8 m) high, 1125 feet 342.9 m) long, and had a total base width of 66 feet (20.1 m). It was constructed of limestone masonry and faced with granite blocks. The mortar of the masonry core had a compressive strength of 2320 lb/in^2 (16.00 MPa). A powerplant was constructed downstream of the left abutment and the central section had an ogee-type shape to allow water to flow over the dam during flood events. The dam is founded on interbedded limestone that dips downstream 3 or 4 feet per 100 feet (3 or 4 m per 100 m). The beds are up to a few feet thick and alternate between hard and soft layers. The rock is extensively fractured and contains many open or clay filled seams. A shallow (about 4 feet (1.2 m) deep) cutoff trench was excavated at both the upstream and downstream foundation contacts into the rock. Flowing springs were encountered in both the river bed and abutments during construction.

Behavior under loading: Shortly after completion in 1894, problems were encountered at the powerplant headgate structure due to water flow along a fault beneath the structure. The structure was badly damaged (exact failure mode unknown), necessitating costly repairs. On April 7, 1900, a heavy rainstorm resulted in 11 feet (3.4 m) of water going over the crest of the dam. At that time, about 440 feet (134.1 m) of the left side of the dam slid downstream about 40 or 50 feet (12.2 or 15.2 m) (see figures AT-1 and AT-2). This section of the dam maintained its line and vertical position. It then broke into two pieces, which remained intact for about 1 hour before breaking up and washing away. The water flowing over the structure had eroded the toe and exposed daylighting bedding planes. The strength of these planes was insufficient to resist the loading.

Twelve years later rebuilding of the dam began. A hollow reinforced concrete buttress dam was built in the gap created by the 1900 failure. The buttress footings were carried to the first hard rock layer encountered, shallow cutoff walls were constructed, and foundation grouting was performed to depths from 5 to 11 feet (1.5 to 3.4 m). The crest of the rebuilt masonry dam was 9 feet 2.7 m) below the old masonry dam. Fifteen-foot-high (4.6-m-high) spillway gates (apparently radial gates) were constructed on top of the buttress dam, and 6-foot (1.8 m) high crest gates (some type of slide gate, flap gate, or stoplog) were placed on the masonry dam. Reconstruction was completed in 1915, but soon afterward another flood destroyed 20 crest gates and blocked the tailrace and draft tubes of the powerhouse with debris.

In 1917 the dam was investigated by Daniel W. Mead, a consulting engineer called in by the City of Austin. He recommended extensive repairs to the foundation and superstructure, but no action was taken. In 1935 a flood greater than any of record, and three times as large as that which caused the original failure, resulted in water flowing 25 feet 7.6 m) over the crest of the dam. The flood tore out all but three of the crest gate piers on the old masonry dam, and more that half of those on the buttress dam. A portion of the buttress dam crest slab broke away, and over 200 feet (61.0 m) of the downstream face slab tore away.

Consequences: The original failure left the city of Austin, Texas without water or power. There is no mention of loss of life in the available documentation.

Back Calculations: The foundation of the original dam was virtually on the surface of the river bed rock. The foundation here consists of nearly horizontal layers of Edwards limestone, which is extensively fractured and contains many open or clay-filled seams. Stability calculations for the overtopping conditions which were occurring at failure of the original dam (see figure AT-3) show that tension existed at the heel of the dam when a linear uplift distribution is assumed from reservoir head at the toe to tailwater at the heel. However, the tensile stress is very small (4 lb/in^2 (0.03 MPa)). If zero tensile strength is assumed, and full uplift is assumed to penetrate to the crack tip, the calculations show that the crack would continue to propagate through the dam thickness. However, the dam was probably bonded to the foundation at least somewhat, and the foundation jointing probably interconnected enough that full uplift at a crack tip would not occur. It is unlikely that such a small tensile stress would result in complete instability. The more likely scenario stems from the fact that the design did not account for the potential riverbed erosion and undercutting of the dam toe during flood flows of the magnitude experienced (11 feet (3.4 m) over the spillway crest). The weak bedding planes would daylight into the eroded hole with the loss of any keying effect from downstream rock layers. Assuming resistance only from the portion of the dam base in compression, a friction angle less than 50 degrees would lead to sliding. It is likely that the friction angle of the weak bedding layers was considerably less than this, but there probably was some downstream passive rock, roughness, cohesion, and/or interlocking rock that prevented sliding until the reservoir reached the height at which failure occurred.

Discussion: In 1937, following the flood of 1935, the Lower Colorado River Authority began extensive investigations and testing for the design of a new dam. The new hollow buttress reinforced concrete structure, named Tom Miller Dam after an Austin mayor, was completed by 1940. It was built atop the remains of the two earlier structures after extensive foundation repair and underpinning. The masonry section alongside the buttresses had the crest gates removed and was repaired with a thick concrete surface on the downstream face and toe. The new design recognized the need for protecting the riverbed downstream of the toe by providing an extensive reinforced concrete apron having a minimum thickness of 3 feet (0.9 m).

The purpose of the dam is to provide hydropower and water supply. Tom Miller Dam is now 100.5 feet (30.6 m) high with a crest length of 1,590 feet (484.6). The base is 155 feet (47.2 m)

wide and the crest is 22.8 feet (6.9 m) thick with nine radial gates having a capacity of 110,000 ft^3/s (3115 m^3/s). The powerplant's 2 units provide 15,000 kilowatts while passing 2,000 ft^3/s (57 m^3/s) each.

References: "The Failure of the Masonry Dam at Austin, Texas," Scientific American, April 28, 1900, pp. 265-266.

Freeman, G.L. and R.B. Alsop, "Underpinning Austin Dam," Engineering News Record, January 30, 1941, pp. 52-57.

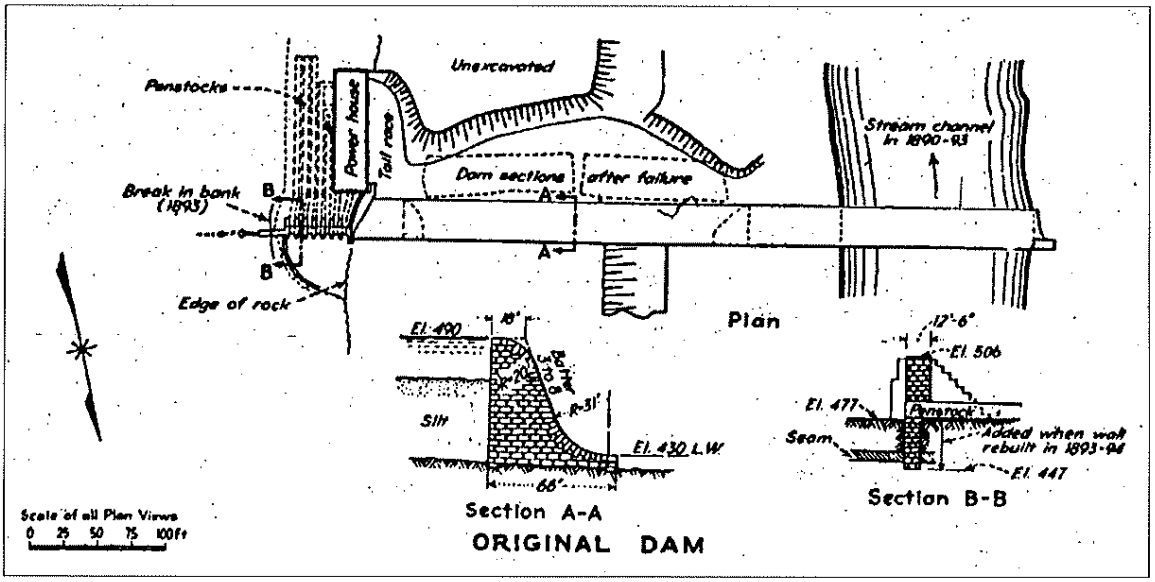

Figure AT-1. Plan and section of Austin Dam TX (after Engineering News Record, 1-30-1941)

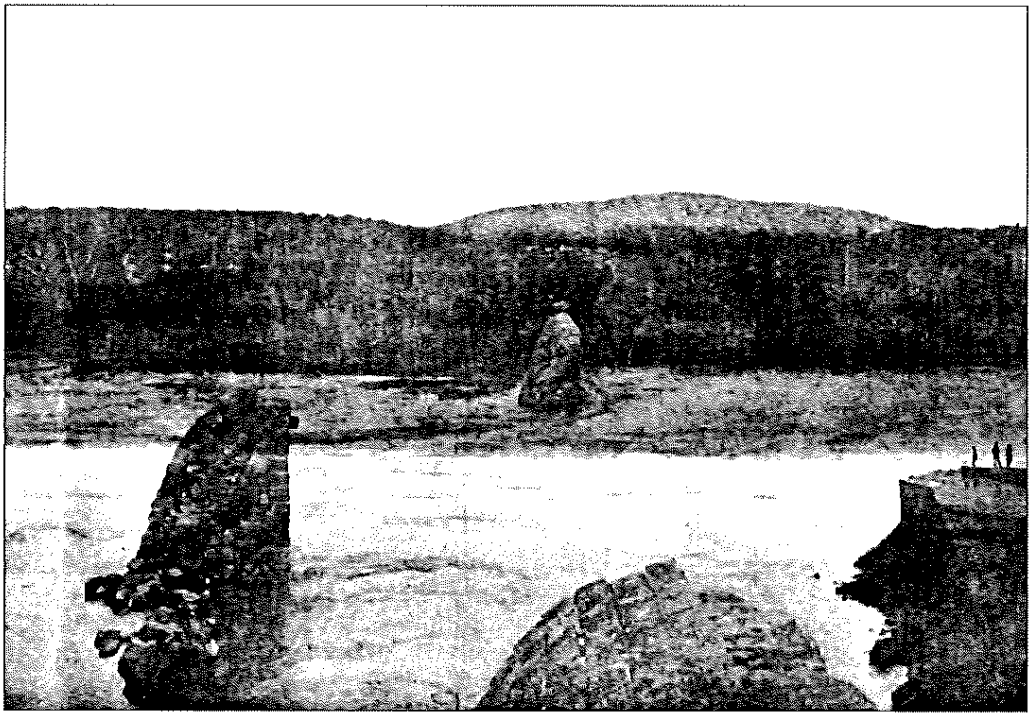

Figure AT-2. Failure of Austin Dam, TX (after Scientific American, 4-28-1900)

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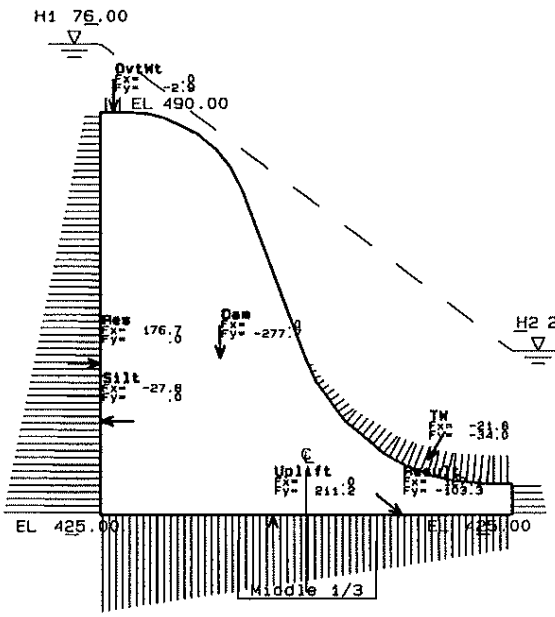
AUSTIN DAM - COLORADO RIVER OF TEXAS



DAM DIMENSIONS

Analysis type	VaryReserv
Criteria at drains	Corps
Top of dam elevation	490.000
Base of dam elevation	425.000
Thickness of crest	4.140
Base width	66.000
Reservoir elevation	501.000
Tailwater elevation	451.400
Silt elevation	470.000
Initial crack length	.000
Drain distance from axis	.000
Drain distance from heel	.000
Drain efficiency	1.000
Drainage gallery elevation	425.000
Head at H1, Heel	76.0000
Head at H2, Toe	26.4000
Head at H3, Drain	76.0000
CORPS H3=H1	

MATERIAL PROPERTIES:

Density of concrete		.13000	K/ft3
	water	.06250	K/ft3
	horiz sat silt	.035	K/ft3
	vert sat silt	.120	K/ft3
Cohesion:	Break bond	100.000	lb/in2
	Apparent	.000	lb/in2
Friction:	Bonded	1.000	45.0 deg
(tangent&angle)	Unbonded	1.000	45.0 deg
Fraction of area bonded		1.000	

MOMENT ABOUT UNCRACKED BASE AT 33.0 425.0

Desc	ForceX	ForceY	CGX	CGY	MomentX	MomentY
Dam	.0	-277.7	19.2	450.3	.0	-3827.5
Res	176.7	.0	.0	449.4	4313.0	.0
Silt	-27.8	.0	.0	440.0	-417.7	.0
Tail	-21.8	-34.0	52.4	433.8	-191.7	659.4
Uplt	.0	211.2	27.7	425.0	.0	1125.3
Ovt	.0	-2.8	2.1	490.0	.0	-88.0
Rslt	127.1	-103.3	48.2	425.0	1572.8	.0

FORCES	w/uplift	w/o uplift
Horizontal (+=d/s)	127.10	127.10 K
Vertical (-=down)	-103.30	-314.50 K
Resultant	163.78	339.21 K
Location from CL	15.23	1.42
Distance 1/3 base	11.00	
Distance 1/2 base	16.50	

MOMENTS
33.00 from heel 1572.82 447.53 k-ft
OVERTURNING SF = 1.15

VERTICAL STRESSES AT HEEL AND TOE (Tension is positive)

	w/uplift	w/o uplift	
Area of base, A		66.00	ft2
Moment of inertia, I		23958.00	ft4
Moment arm, c		33.00	ft
Axial stress, P/A	-10.87	-33.09	lb/in2
Moment stress, Mc/I	15.04	4.28	lb/in2
Vertical stress at heel	4.18	-28.81	lb/in2
Vertical stress at toe	-25.91	-37.37	lb/in2

MINIMUM COMPRESSIVE STRESS AT HEEL FOR NO CRACKING
szu = pwh - ft/s
Drain factor, p= 1.000
Safety factor,s= 1.00

Tensile Strength lb/in2	szu Allowable lb/in2	Heel Condition
.00	32.99	Cracked
25.00	7.99	No crk
75.00	.00	No crk
125.00	.00	No crk
175.00	.00	No crk
225.00	.00	No crk

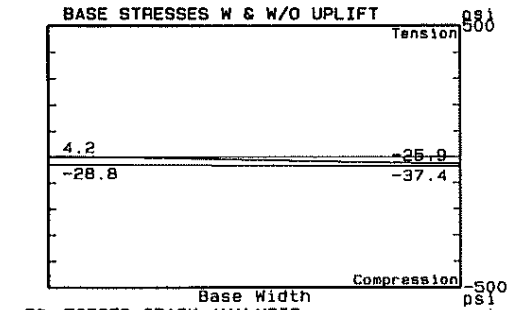
SLIDING FACTORS OF SAFETY
Driving force 127.10 kips
Total vertical forces -103.30 kips
Safety Factor = 8.29 for above strengths
Cohesion(Bonded)= 2.50 psi req for FS= 1.0
Sliding safety factors for various cohesions:

BreakBond-psi	.0	50.0	100.0	150.0	200.0	250.0
Resisting-K	103.3	578.5	1053.7	1528.9	2004.1	2479.3
Safety factor	.81	4.55	8.29	12.03	15.77	19.51

Required strengths to get these safety factors while keeping other given values constant

	1.0	2.0	3.0
Bonded area: cohesion (psi)	2.5	15.9	29.2
friction angle (deg)	.0	.0	.0

FORCED CRACK RESULTS
Heel initially in tension
Crack extends entire width of base



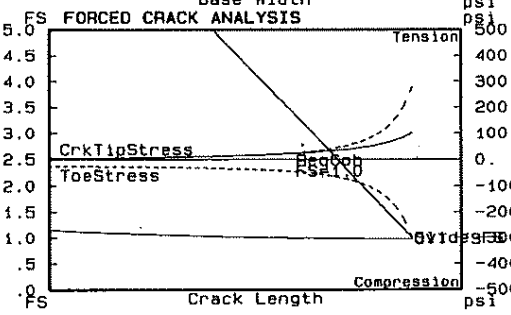


Figure AT-3. Calculations for Austin Dam, TX for conditions at failure

CASE HISTORY SUMMARY

Name of site or structure: Bouzey Dam (masonry gravity)
Location: France - Near Belfort
Type of event: Tipping, horizontal cracking, internal hydrostatic uplift and sliding failure of masonry dam and its foundation
Date of event: First event 1884; second event 1895
Date of construction (if applicable): Original construction completed 1880
Loading: Static loading during spring runoff

Description of site, structure and materials: Bouzey Dam was a 22-m-high (72-foot high) masonry gravity dam situated on the L'Aviere River near Epinal in Vosges Province, France. It was founded on horizontally bedded sandstone. The upper portion of the sandstone was variegated (i.e. possibly interbedded), described as somewhat jointed and porous. The strata were described as "without cohesion" and a cutoff key, about 2 m (6.6 ft) wide and 6 to 10 m (19.7 to 32.8 ft) deep, was constructed at the upstream face of the dam. The dam's original cross-section was unusually thin for a gravity dam, especially over the middle third of its height, but also at the base.

After serious performance problems, bordering on failure and uncorrected for an extended period of time (described below), the dam was finally modified by adding a longitudinal masonry course to cover a cracked and dislocated heel cutoff wall. The new masonry was covered with a clay layer to provide additional protection against leakage. The downstream side was thickened over the bottom third of the dam height and keyed deeper into the foundation, but the narrow top two thirds was not changed, and the dam eventually failed catastrophically (see figure BF-1).

Behavior under loading: During initial filling, when the water level reached elevation 352 m (1154.9 ft) (about 10 m (32.8 ft) from the dam crest, el. 371.95 m (1220.3 ft), springs appeared downstream of the dam with a flow rate of about 50 l/s (793 gpm). The seepage increased to 75 l/s (1189 gpm) when the reservoir reached el. 364.5 m (1195.9 ft). The reservoir reached elevation 368.8 m (1209.97 ft) (2.7 m (8.9 ft) below maximum anticipated level) for the first time on March 14, 1884, when a 135 m (443 ft) long part of the dam suddenly moved downstream, and the seepage rate increased to 230 l/s (3646 gpm). The maximum movement of 0.34 m (1.11 ft) sheared the cutoff key without causing any discernable vertical settlement. A horizontal crack extended 93 m (305 ft) along the upstream heel of the dam. The rock was crushed and dislocated to a depth of 2 to 3 m (6.6 to 9.8 ft) under the dam, with lenticular clay deposits several mm thick and openings that produced seepage flows. The reservoir was allowed to remain at this level for nearly a year, during which time conditions remained stable.

From 1888 to 1889 the dam was strengthened by the addition of a downstream buttress and the upstream crack was sealed. During the subsequent reservoir filling the dam failed suddenly on April 27, 1895 when the reservoir reached elevation 371.4 m (1218.5 ft). The upper central portion of the dam, about 10.5 m (34.4 ft) high and 170 m (558 ft) long was swept away. A

nearly horizontal failure surface occurred at the upstream face and through the masonry for a distance of about 3.5 m downstream, where it dipped suddenly downstream. Numerous crush marks and shears were noted near the downstream face. This time the foundation did not move.

Consequences: The sudden failure released a torrent of water upon the village of Bouzey which left it and several other villages in the valley of L'Aviere in ruins. The flooding caused the death of more than 100 people.

Back Calculations: 1) First failure in foundation: As previously mentioned, the foundation partings beneath the dam were described as "without cohesion." To remedy this lack of cohesion, a cutoff key, about 2 m (6.6 ft) wide and 6 to 10 m (19.7 to 32.8 ft) deep, was constructed at the upstream face of the dam. Stability analyses, with the reservoir water surface set equal to that which caused the failure (see figures BF-2 and BF-3), indicate tension occurred at the heel of the dam. Since there were no foundation drains, a linear uplift distribution from reservoir head to tailwater head was assumed. Assuming zero tensile strength and full uplift penetrating to the crack tip, analyses indicate a crack would extend through the cutoff regardless of tailwater assumptions or passive resistance offered by the downstream fill. However, these analyses indicate this crack would not continue to propagate, but would stop within the dam section, which represents the actual behavior reasonably well. It is unlikely that full uplift would extend to the crack tip in the foundation or at the contact due to drainage from interconnecting joints, so a crack may not propagate far in this area. Based on force resolution only, and assuming no cohesion for the foundation (i.e. open daylighting bedding planes), friction angles from 42 to 52 degrees, depending upon the presence of tailwater and passive pressures, would be required to provide a stable structure. The weak planes in the foundation could probably not provide these high friction angles, the result perhaps being a sudden movement of the dam downstream until the whole block mobilized downstream resistance. Modifications to the dam, which included addition of a downstream buttress to thicken the base, did not address the critical mid-height thickness inadequacy.

2) Second failure at mid-height: Stability calculations performed for the second failure scenario (see figure BF-4) with a linear uplift distribution show that tension (up to about 0.23 MPa (33 lb/in^2)) existed at the upstream face of the dam at the level of the mid-height failure plane. Assuming zero tensile strength and full reservoir head up to the crack tip, the analyses show that the crack would continue to propagate until the top part of the dam broke away. The amount of tensile strength that may have existed in the masonry mortar is unknown, but based on descriptions, the mortar was probably weak. If the masonry was relatively water tight and cracked, full reservoir head could have been transmitted to the crack tip. Thus, cracking completely through seems to be a real possibility. At that point failure by overturning or sliding could occur. The crushing observed indicated that large compressive stresses existed at the toe of the dam. The downstream dip of the sliding surface near the downstream face could have followed the path of lowest strength, and was not modeled in the analyses. If it is assumed that a linear uplift distribution existed at the time of failure, then a friction angle less than about 37 degrees would be sufficient to cause sliding.

Discussion: As a result of this failure, attention was given for the first time to the effects of internal pressure in masonry dams and of uplift in the foundation. Maurice Levy submitted a memoir to the Academy of Sciences in 1895 describing these mechanisms which had been ignored up until that time. The sandstone foundation was found to be cracked and the cutoff wall at the heel of the dam did not extend adequately into the rock, thus allowing water under the structure. Faulty masonry joints permitted similar leakage at higher points within the dam where the structural thickness was already marginal. These flaws were considered the primary cause of the eventual destruction by internal hydrostatic pressures and uplift. Other factors undoubtedly also contributed to events leading up to the failure.

The builders had mixed lime with dirty sand of poor quality for the mortar and it is alleged that the preparation of the mortar was done carelessly. If some of the lime remained unslaked as it was placed in the joints, subsequent wetting during reservoir filling would have caused it to swell. This is considered a possible mechanism for causing joint separation. The comparatively thin masonry section would have been susceptible to tilting away from the thrust of the reservoir water and this combined with cracks at the deteriorating mortar joints provided a path for the hydrostatic uplift forces.

References: Lessons from Dam Incidents (complete edition), International Commission on Large Dams, Paris, 1974, pp. 26-29.

Jansen, Robert B., "Dams and Public Safety", U.S. Department of the Interior, Bureau of Reclamation, 1983, pp. 126-128.

Other references in both French and English dating back to 1895 are listed at the end of the above reference, but have not been reviewed in detail.

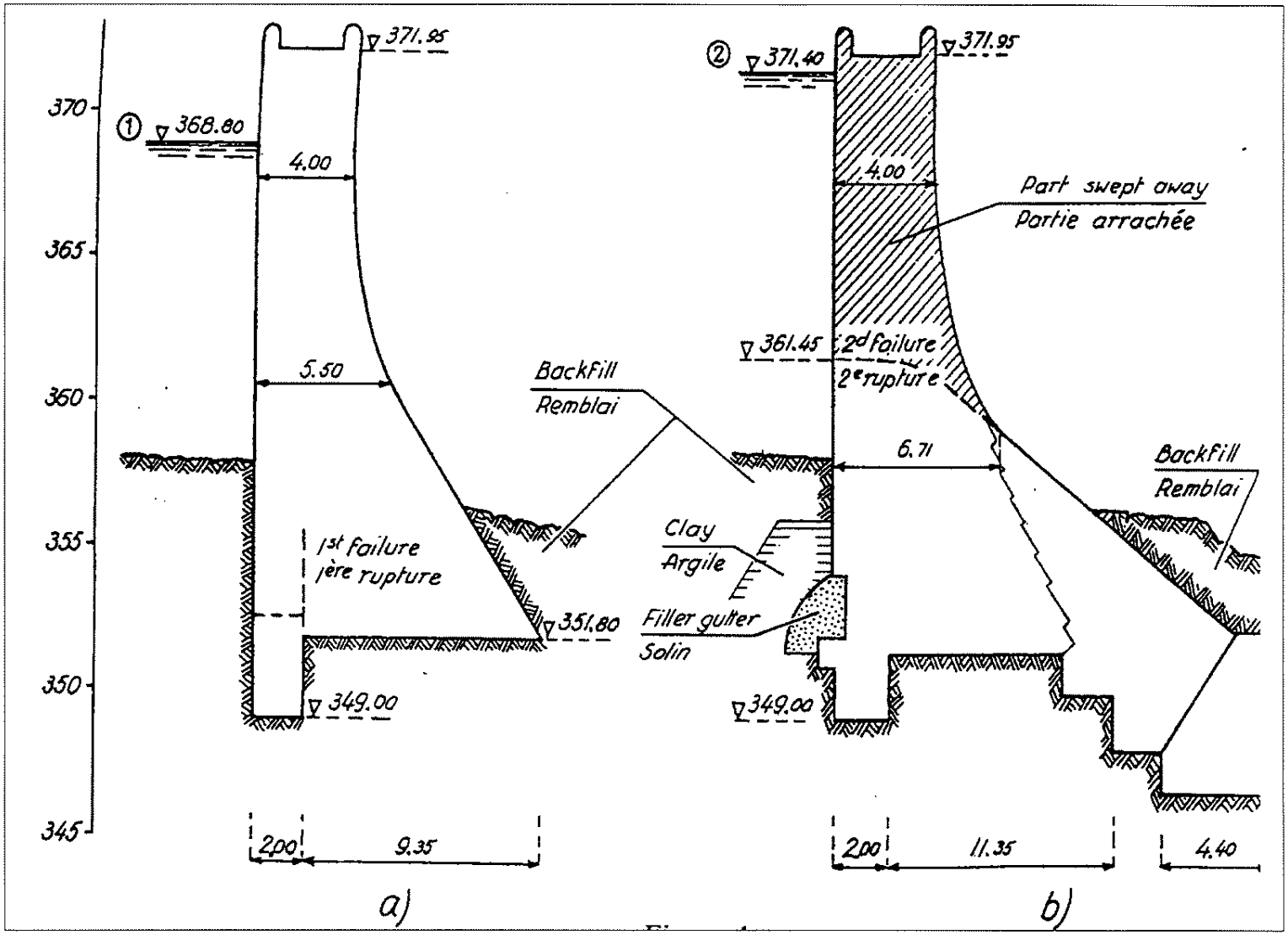


Figure BF-1. a. First failure of Bouzey Dam, b. Second failure of Bouzey Dam (after ICOLD 1974)

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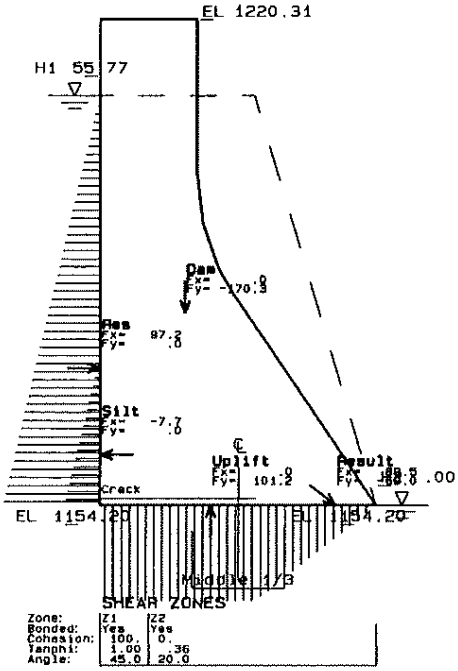
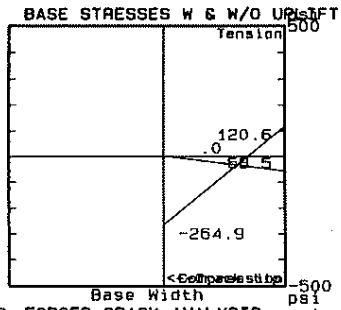
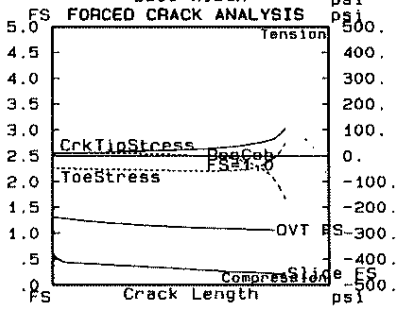
Bouzey Dam 1st Failure

Cracked - No Tailwater-Zero Passive





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DAM DIMENSIONS
 Analysis type                    Standard
 Criteria at drains               Corps
 Top of dam elevation             1220.310
 Base of dam elevation            1154.200
 Thickness of crest                 13.120
 Base width                         37.240
 Reservoir elevation              1209.970
 Tailwater elevation              1154.200
 Silt elevation                   1174.500
 Initial crack length               20.850
 Drain distance from axis             .000
 Drain distance from heel             .000
 Drain efficiency                    1.000
 Drainage gallery elevation       1154.200
 Head at H1, Heel                  55.7700
 Head at H2, Toe                     .0000
 Head at H3, Drain                 55.7700
 CORPS  H3-H1

MATERIAL PROPERTIES:
 Density of concrete              .13000 K/ft3
            water                 .06250 K/ft3
            horiz sat silt          .025 K/ft3
            vert  sat silt          .120 K/ft3
Zoned Shear     Zone1  Zone2  Zone
X Dist (ft)       .00   6.56
Bond(0-U/1-B)    1.0    1.0
Cohesion (psi):
  Bonded       100.0     .0
  Unbonded     100.0     .0
Friction tangent:
  Bonded Comp  1.000    .364
  Bonded Tens   .000    .000
  Unbonded     1.000    .364
Friction angle:
  Bonded Comp   45.0   20.0
  Bonded Tens     .0     .0
  Unbonded      45.0   20.0
Tens limit        .0     .0

MOMENT ABOUT UNCRACKED BASE AT      29.0  1154.2
Desc   ForceX  ForceY     CGX      CGY MomentX MomentY
Dam        .0  -170.2    11.4   1180.3       .0 -2997.4
Res      97.2      .0      .0   1172.8   1806.9      .0
Silt     -7.7      .0      .0   1161.0    -52.3      .0
Uplt      .0    101.2    14.9   1154.2       .0  1431.2
Rslt     89.5   -69.0    31.8   1154.2    188.5      .0

FORCES                  w/uplift    w/o uplift
Horizontal (+-d/s )       89.47       89.47 K
Vertical   (--down)      -69.01     -170.25 K
Resultant                112.99      192.33 K
  Location from CL         2.73       -7.30
  Distance 1/3 base        6.21
  Distance 1/2 base        9.31
MOMENTS
 29.05 from heel     188.47  -1242.77 k-ft
OVERTURNING SF =          1.09

VERTICAL STRESSES AT HEEL AND TOE (Tension is positive)
                         w/uplift     w/o uplift
Area of base, A                        16.39 ft2
Moment of inertia, I                  366.91 ft4
Moment arm, c                           8.20 ft
Axial stress, P/A          -29.24     -72.13 lb/in2
Moment stress, Mc/I         29.23    -192.76 lb/in2
Vertical stress at heel       .00    -264.90 lb/in2
Vertical stress at toe     -58.47     120.63 lb/in2

MINIMUM COMPRESSIVE STRESS AT HEEL FOR NO CRACKING
szu = pwh - ft/s
Drain factor, p=  1.000
Safety factor,s=   1.00
  Tensile        szu          Heel
  Strength    Allowable    Condition
   lb/in2       lb/in2
      .00       24.21      No crk
    25.00         .00      No crk
    75.00         .00      No crk
   125.00         .00      No crk
   175.00         .00      No crk
   225.00         .00      No crk

SLIDING FACTORS OF SAFETY BASED ON STRESS
  Horiz force      89.47 Kips
  Vert force from Load and Stress  -69.01  -69.01 Kips
  Safety Factor    =     .28 for above strengths
  Cohesion(Bonded) =  27.22 psi req for FS= 1.0
Sliding safety factors for various cohesions:
 BreakBnd-psi      .0    50.0   100.0   150.0   200.0   250.0
 ResistFrce-K    25.1   143.3   261.5   379.7   497.9   616.2
 Sliding FS       .28    1.60    2.92    4.24    5.57    6.89
Resist Force:   Zone1  Zone2  Zone
 from cohesion     .0     .0
 from friction     .0    25.1

FORCED CRACK RESULTS
Heel initially in tension
Crack stops at   20.85ft from heel
```

Figure BF-2. Analysis of Bouzey Dam, first failure, no tailwater or passive resistance

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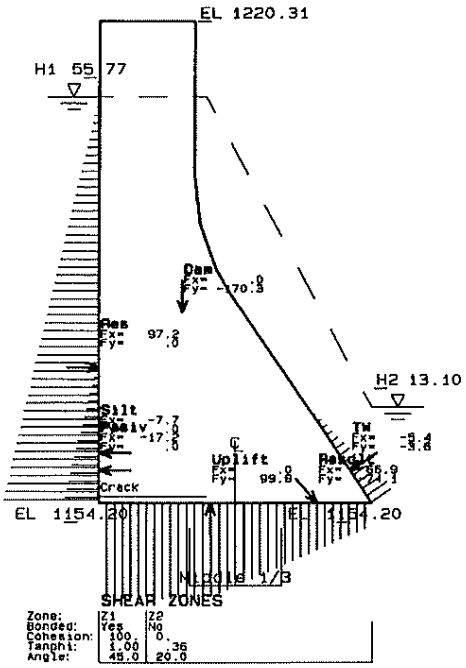
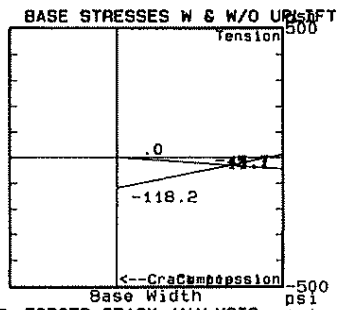
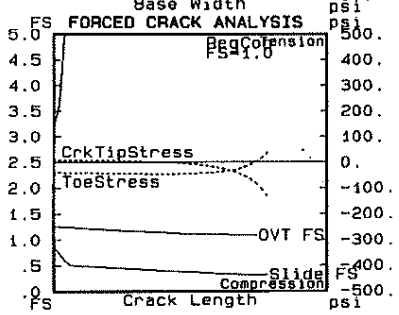
Bouzey Dam 1st Failure

Cracked - Tailwater to Groundsurface





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DAM DIMENSIONS
 Analysis type                    Standard
 Criteria at drains               Corps
 Top of dam elevation            1220.310
 Base of dam elevation           1154.200
 Thickness of crest                13.120
 Base width                        37.240
 Reservoir elevation             1209.970
 Tailwater elevation             1167.300
 Silt elevation                  1174.500
 Initial crack length              14.700
 Drain distance from axis            .000
 Drain distance from heel            .000
 Drain efficiency                   1.000
 Drainage gallery elevation      1154.200
 Head at H1, Heel                 55.7700
 Head at H2, Toe                  13.1000
 Head at H3, Drain                55.7700
 COARS  H3-H1

MATERIAL PROPERTIES:
 Density of concrete             .13000 K/ft3
            water                .06250 K/ft3
            horiz sat silt        .025 K/ft3
            vert  sat silt        .120 K/ft3
Zoned Shear     Zone1  Zone2  Zone
X Dist  (ft)      .00   6.56
Bond (0-U/1-B)    1.0     .0
Cohesion (psi):
  Bonded        100.0     .0
  Unbonded      100.0     .0
Friction tangent:
  Bonded Comp   1.000   .000
  Bonded Tens    .000   .000
  Unbonded      1.000   .364
Friction angle:
  Bonded Comp    45.0    0.0
  Bonded Tens      .0     .0
  Unbonded       45.0   20.0
Tens limit         .0     .0

MOMENT ABOUT UNCRACKED BASE AT     26.0  1154.2
Desc  ForceX  ForceY     CGX     CGY MomentX MomentY
Dam       .0  -170.2    11.4  1180.3      .0 -2473.9
Res     97.2      .0      .0  1172.8  1806.9      .0
Silt    -7.7      .0      .0  1161.0   -52.3      .0
Tail    -5.4    -3.6    34.3  1158.6   -23.4    30.1
Uplt      .0    99.7    15.3  1154.2      .0  1067.0
Psi    -17.2      .0      .0  1158.6   -75.8      .0
Rslt    66.9   -74.1    29.7  1154.2   278.6      .0

FORCES                  W/uplift   W/o uplift
Horizontal (+-d/s )       66.89      66.89 K
Vertical   (-=down)      -74.10    -173.85 K
Resultant                 99.83     186.28 K
  Location from CL         3.76      -4.53
  Distance 1/3 base        6.21
  Distance 1/2 base        9.31
MOMENTS
 25.97 from heel     278.58   -788.39 k-ft
OVERTURNING SF =        1.14

VERTICAL STRESSES AT HEEL AND TOE (Tension is positive)
                          w/uplift     w/o uplift
Area of base, A                          22.54 ft2
Moment of inertia, I                    954.29 ft4
Moment arm, c                            11.27 ft
Axial stress, P/A          -22.83       -53.56 lb/in2
Moment stress, Mc/I         22.85       -64.66 lb/in2
Vertical stress at heel       .02      -118.22 lb/in2
Vertical stress at toe     -45.68        11.09 lb/in2

MINIMUM COMPRESSIVE STRESS AT HEEL FOR NO CRACKING
szu = pwh - ft/s
Drain factor, p=  1.000
Safety factor, s=   1.00
  Tensile        szu          Heel
  Strength    Allowable     Condition
   lb/in2       lb/in2
      .00        24.21      No crk
    25.00          .00      No crk
    75.00          .00      No crk
   125.00          .00      No crk
   175.00          .00      No crk
   225.00          .00      No crk

SLIDING FACTORS OF SAFETY BASED ON STRESS
  Horiz force      66.89 Kips
  Vert force from Load and Stress  -74.10  -74.10 Kips
  Safety Factor   =      .40 for above strengths
  Cohesion (Bonded) =9999.00 psi req for FS= 1.0
Sliding safety factors for various cohesions:
 BreakBnd-psi        .0    50.0   100.0   150.0   200.0   250.0
 ResistFrce-K      27.0    27.0    27.0    27.0    27.0    27.0
 Sliding FS         .40     .40     .40     .40     .40     .40
Resist Force:    Zone1   Zone2   Zone
 from cohesion      .0      .0
 from friction      .0    27.0

FORCED CRACK RESULTS
Heel initially in tension
Crack stops at   14.70ft from heel
```

Figure BF-3. Analysis of Bouzey Dam, first failure, with tailwater and passive resistance

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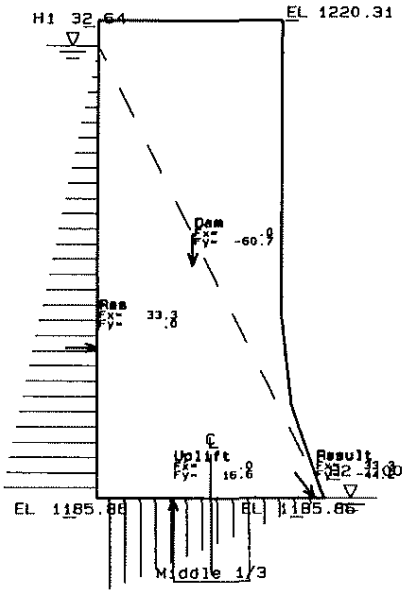
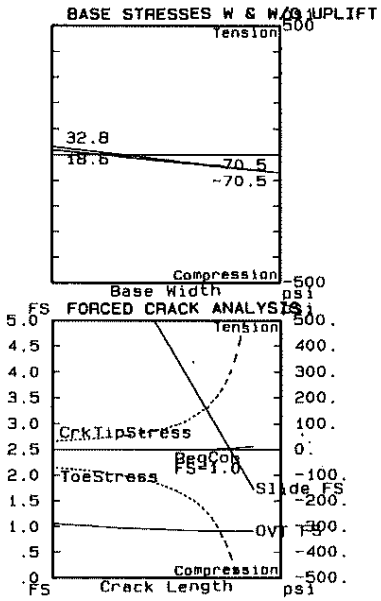
Bouzey Dam 2nd Failure



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DAM DIMENSIONS
 Analysis type                    Standard
 Criteria at drains               Corps
 Top of dam elevation            1220.310
 Base of dam elevation           1185.860
 Thickness of crest                13.120
 Base Width                        16.240
 Reservoir elevation             1218.500
 Tailwater elevation             1185.860
 Silt elevation                  1185.860
 Initial crack length                .000
 Drain distance from axis            .000
 Drain distance from heel            .000
 Drain efficiency                   1.000
 Drainage gallery elevation      1185.860
 Head at H1, Heel                 32.6400
 Head at H2, Toe                   .0000
 Head at H3, Drain                32.6400
 CORPS  H3=H1

MATERIAL PROPERTIES:
 Density of concrete             .13000 K/ft3
            water                .06250 K/ft3
            horiz sat silt        .085 K/ft3
            vert  sat silt        .120 K/ft3
 Cohesion:   Break bond        100.000 lb/in2
             Apparent             .000 lb/in2
 Friction:        Bonded        1.000    45.0 deg
  (tangent&angle) Unbonded       1.000    45.0 deg
 Fraction of area bonded         1.000

MOMENT ABOUT UNCRACKED BASE AT        8.1  1185.9
Desc  ForceX  ForceY     CGX      CGY MomentX MomentY
Dam      .0   -60.7     6.8   1202.6      .0   -80.1
Res    33.3      .0      .0   1185.7   362.2      .0
Uplt     .0    16.6     5.4   1185.9      .0    44.8
Rslt   33.3   -44.1    15.5   1185.9   327.0      .0

FORCES                     w/uplift    w/o uplift
Horizontal (+-d/s )          33.29       33.29 K
Vertical   (--down)         -44.15      -60.71 K
Resultant                    55.30       69.24 K
  Location from CL            7.41        4.65
  Distance 1/3 base           2.71
  Distance 1/2 base           4.05
MOMENTS
  8.12 from heel     327.01     282.17 k-ft
OVERTURNING SF =        1.06

VERTICAL STRESSES AT HEEL AND TOE (Tension is positive)
                              w/uplift     w/o uplift
Area of base, A                             16.24 ft2
Moment of inertia, I                       356.92 ft4
Moment arm, c                                 8.12 ft
Axial stress, P/A            -18.88       -25.96 lb/in2
Moment stress, Mc/I           51.66        44.58 lb/in2
Vertical stress at heel       32.78        18.62 lb/in2
Vertical stress at toe       -70.54       -70.54 lb/in2

MINIMUM COMPRESSIVE STRESS AT HEEL FOR NO CRACKING
szu = pwh - ft/s
Drain factor, p=  1.000
Safety factor,s=   1.00
  Tensile        szu          Heel
  Strength    Allowable    Condition
   lb/in2       lb/in2
      .00        14.17      Cracked
    25.00          .00      Cracked
    75.00          .00      Cracked
   125.00          .00      Cracked
   175.00          .00      Cracked
   225.00          .00      Cracked

SLIDING FACTORS OF SAFETY
  Driving force          33.29 kips
  Total vertical forces     -44.15 kips
  Safety Factor =    8.35 for above strengths
  Cohesion(Bonded) =    .00 psi req for FS= 1.0
Sliding safety factors for various cohesions:
BreakBond-psi      .0    50.0   100.0   150.0   200.0   250.0
Resisting-K      44.1   161.1   278.0   394.9   511.9   628.8
Safety factor    1.33    4.84    8.35   11.86   15.37   18.89
Required strengths to get these safety factors while
keeping other given values constant    1.0     2.0     3.0
    Bonded area:     cohesion (psi)      .0     9.6    23.8
                 friction angle (deg)     .0      .0      .0

FORCED CRACK RESULTS
Heel initially in tension
Crack extends entire width of base
```

Figure BF-4. Analysis of Bouzey Dam, second failure

CASE HISTORY SUMMARY

Name of site or structure: Upper Stillwater Dam
Location: Northern Utah, USA - On the south flank of the Uinta Mountains approximately 40 miles (64 km) northwest of Duchesne.
Type of event: Foundation movement
Date of event: Summer 1988 - During initial filling
Date of construction: 1985 thru 1987

Loading: The first reservoir filling, during the fall of 1987, provided the downstream hydrostatic force which instigated the foundation movement. Unusually high piezometer readings were also recorded early in the filling process.

Description of site, structure and materials: Upper Stillwater Dam is the Bureau of Reclamation's first roller-compacted concrete (RCC) dam (see figures UU-1, UU-2, and UU-3). The dam is a straight gravity type with a maximum height of 290 feet (88.4 m), although over much of its length it is about 220 feet (67.1 m) high. The crest of the dam is approximately 2,680 feet 816.9 m) long and 29.2 feet (8.9 m) wide at an elevation of 8177.5 feet (2492.50 m). Concrete parapet walls extend above the crest on both the upstream and downstream sides. A central overflow spillway allows active storage to an elevation of 8172 feet (2490.83 m). The reservoir behind the dam has a total capacity of approximately 33,123 acre-feet (40.84×10^6 m^3). The spillway is a 600 foot (182.9 m) long uncontrolled ogee crest with a slip-formed concrete stepped chute and drowned hydraulic jump basin. The spillway, which separates the left and right crest roadways, has a capacity of 15,000 ft^3/s (424.8 m^3/s) at a reservoir elevation 8175.5 feet (2491.89 m). The outlet works consists of a single 6-foot (1.8-m) diameter conduit beneath the dam from a free-standing intake tower.

The dam is founded on interbedded sandstone and argillite of the Precambrian Uinta Mountain group. Interbedded sandstone and argillite is present near the crest on both abutments. A thick argillite layer is present near the base of the dam at both abutments. A lower sandstone unit, with nearly horizontal bedding structure, forms most of the foundation. A continuous 2-foot (0.61-m) thick argillite interbed, labeled Unit L, lies beneath the dam within the lower sandstone foundation. The presence of silty sand fillings was observed in some of the bedding planes and vertical foundation joints. Blanket grouting was performed over the entire foundation to a depth of thirty feet. The importance of the passive rock mass downstream of the dam was recognized and additional blanket grouting was performed in the spillway channel as a defensive measure to stiffen and strengthen the rock.

Behavior under loading: Horizontal movement of the foundation on the Unit L argillite layer was recorded by multiple point borehole extensometers (MPBX's) beginning in June 1988, when the reservoir reached elevation 8130 feet (2478.0 m) (see figure UU-4). The movement was distinct in onset, uniform in rate, and ended abruptly once the reservoir was filled and was drawn

down. Total movement recorded by MPBX C-2 was 0.4 inches (10 mm) with lesser amounts at other instruments. A distinct offset across the unit L argillite interbed was discernable in three inclinometer profiles (see figure UU-5). Piezometers reacted generally as expected except for two upstream piezometers which rose faster than the reservoir during the upper stages of filling. The recorded pressures were quite high and exceeded Reclamation criteria, even downstream of the drains.

A Reclamation analysis of the instrumentation response was documented in Technical Memorandum US-D3620-1. An independent review, provided by Mr. Robert L. James and documented in Decision Memorandum US-D3620-2, concluded:

> "Discrete permanent movement of 0.3 to 0.6 inch occurred generally along the unit L argillite bed during first filling. The amount of movement was controlled by joint closure downstream of the dam, and does not represent sliding of a foundation rock mass fully bounded by discontinuities."
>
> "Discrete movement on a foundation plane is not considered to be normal behavior, and although the dam is not judged to be in danger, very careful monitoring of the next filling is necessary."

Consequences: No notable consequences resulted from the incident. However, the downstream hazard classification for Upper Stillwater Dam is high. Failure of the dam would result in nearly complete inundation of the town of Duchesne, Utah, although the flood wave travel time would be significant.

Shrinkage/temperature cracks, some of which extend continuously through the parapets, crest, galleries, and downstream face, resulted in significant leakage in at least 15 distinct locations. These cracks, one of which was up to a 1/4-inch (6 mm) wide, was probably aggravated by the relative downstream foundation movement since some displacement could be seen at the large crack. Leakage rates at the worst crack reached 2,000 gal/min (126 l/s). while others recorded leakage rates around 150 gal/min (9.5 l/s). Extensive remedial grouting and crack repair was required to reduce the leaks.

In addition to seepage from cracks in the dam, there was also significant flow from the foundation drains. Forty additional drain holes were subsequently added to the original 270. Approximately 35 of the 310 drains had very high flows and were prone to carrying sand into the drainage galleries. Attempts to filter the sand using slotted pipe wrapped with filter fabric were essentially unsuccessful, as iron-forming bacteria clogged the fabric. During initial filling, up to 75 gal/min (4.7 l/s) was flowing from drains in one area and fine sand was observed in the discharge. Sand and iron producing bacteria had also resulted in plugging of some drain holes. A remedial foundation grouting and drainage program was completed in 1993, which resulted in significantly reduced drain flows, and acceptable uplift pressures.

Back Calculations: Finite element studies were conducted to compare the observed behavior with what would have been predicted. Linear elastic plane strain analyses were conducted using the computer program SAPIV. Deformation properties were determined from laboratory testing of the RCC and borehole jacking tests performed in the foundation rock. The foundation was modeled to the depth of the extensometers and for a distance upstream and downstream equal to the height of the dam. Nodal points were modeled at the MPBX anchor locations. Several reservoir water surface elevations were analyzed with water load applied to the upstream face and on the rock surface upstream of the dam. In general, the actual deformations were smaller than the model predicted by a linear elastic analysis until the reservoir reached elevation 8140 feet (2481.07 m). At that point the actual deformations increased dramatically and exceeded the model deformations by a considerable amount. It was concluded that horizontal shearing along the Unit L argillite had occurred, and was responsible for the large movements recorded by the angled MPBXs and the inclinometers.

Samples of the Unit L argillite were obtained and tested both prior to construction, and following the movements. Direct shear test results indicated lower strength values for samples obtained following the movements (see figure UU-6). Stability analyses were conducted, including the potential for progressive instability due to localized failure and subsequent inability to redistribute the excess driving force. The only strength which accurately predicted shearing of the Unit L argillite at reservoir elevation 8140 feet (2481.07 m) was the post-construction lower bound strength (the lowest strength measured).

Discussion: The foundation movements far exceeded the expected values, and occurred at discrete locations as yielding along the Unit L argillite. The mode of displacement was by base plane shearing limited by compression of downstream vertical joints, shears, and faults. This resulted in mobilization of the resistance provided by the downstream passive rock mass. The possibility of rigid block displacement that could lead to failure was ruled out because (1) the foundation movement proceeded at a smooth and constant rate, even though the loading increased as the square of the reservoir height, (2) the movement abruptly terminated when the reservoir reached its maximum elevation, (3) the movement was not uniform in magnitude, and (4) the direction of movement was not uniform. Selecting strengths for weak discontinuities may require selecting values lower than the average values measured on small samples. However, it is comforting to realize that the downstream passive rock mass provides significant resistance. No significant additional displacement was recorded across the unit L argillite during the second reservoir filling and the dam now appears to be performing satisfactorily, although very small permanent movements may still be occurring as the reservoir fills and empties each year.

References: Scott, Gregg A., "Deformation of Rock Foundations Under Dams", Dam Foundation Engineering Tenth Annual USCOLD Lecture, New Orleans, Louisiana, March 6-7, 1990.

Memorandum to Head, Dam Safety Inspection Section, Subject: "Examination Report for Upper Stillwater Dam - Safety Evaluation of Existing Dams (SEED) Program" , dated June 5, 1990.

Technical Memorandum No. US-3620-3, Evaluation of 1992/1993 Grouting and Drain Remediation Program, U.S. Bureau of Reclamation, 1995.

Technical Memorandum No. US-8314-1, Performance Monitoring Criteria- Upper Stillwater Dam, U.S. Bureau of Reclamation, 1994.

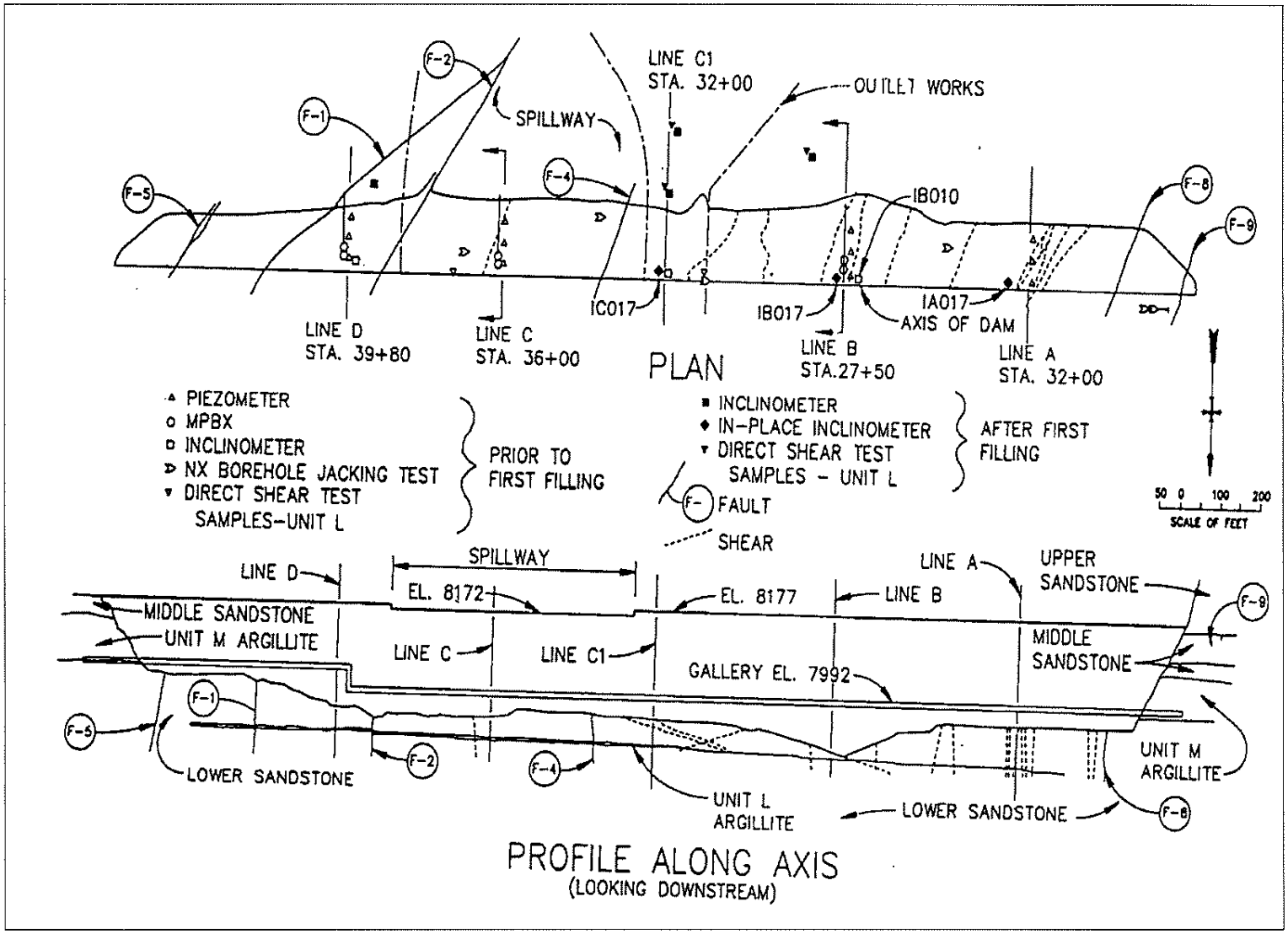

Figure UU-1. Plan of Upper Stillwater Dam with instrumentation and sample locations

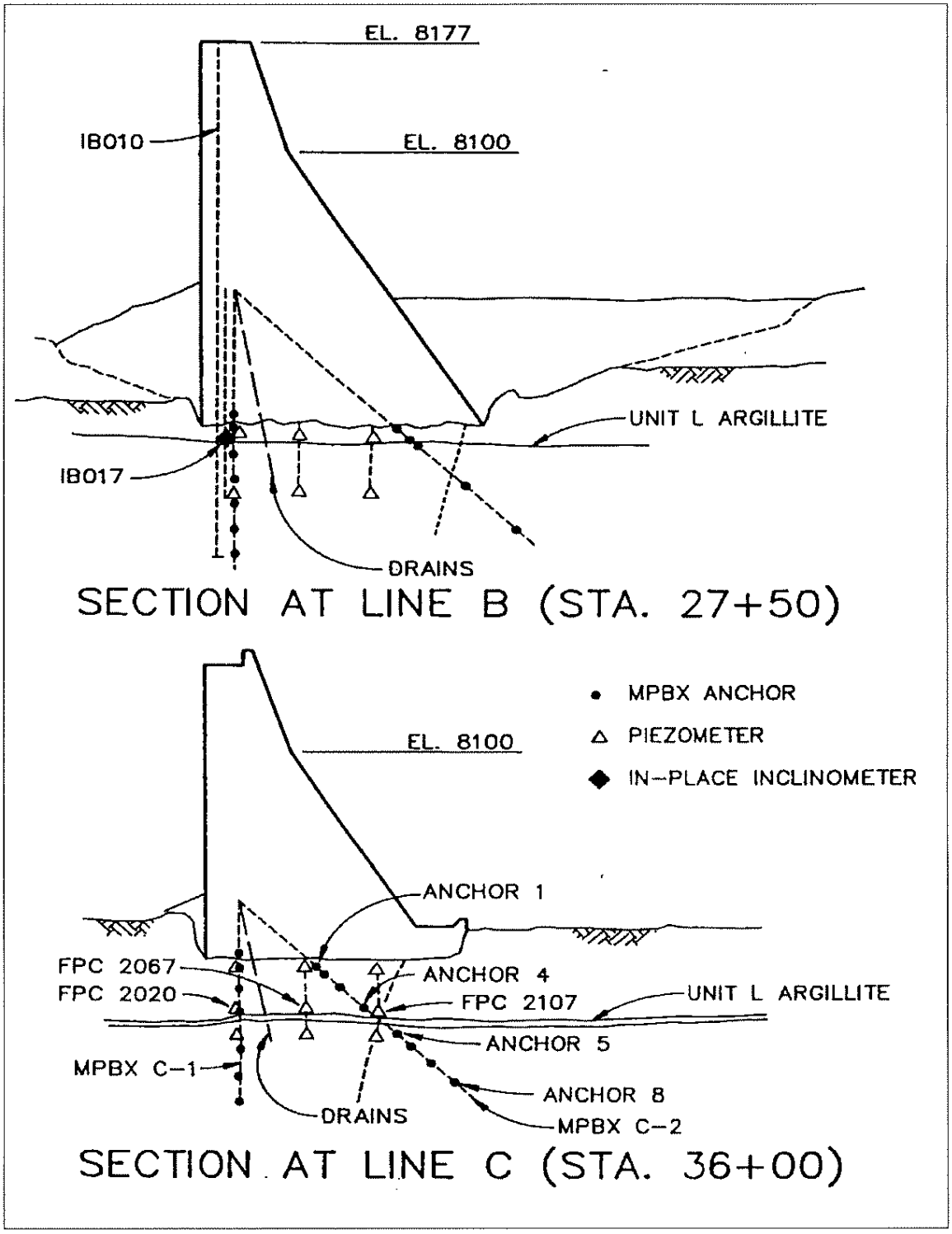

Figure UU-2. Sections through instrument locations at Upper Stillwater Dam

Figure UU-3. Photo of Upper Stillwater Dam

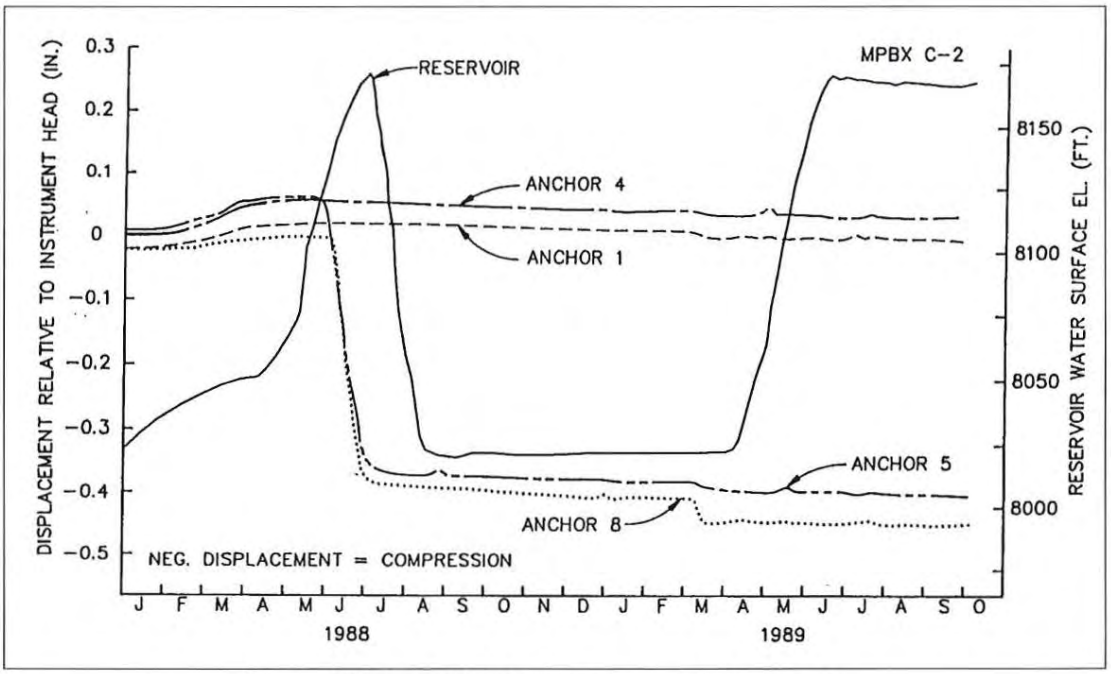

Figure UU-4. MPBX C-2 response to initial loading

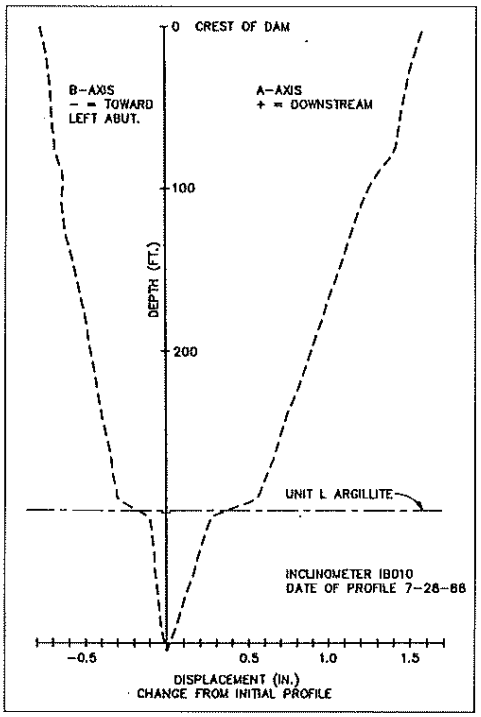

Figure UU-5. Inclinometer B010 response during initial filling

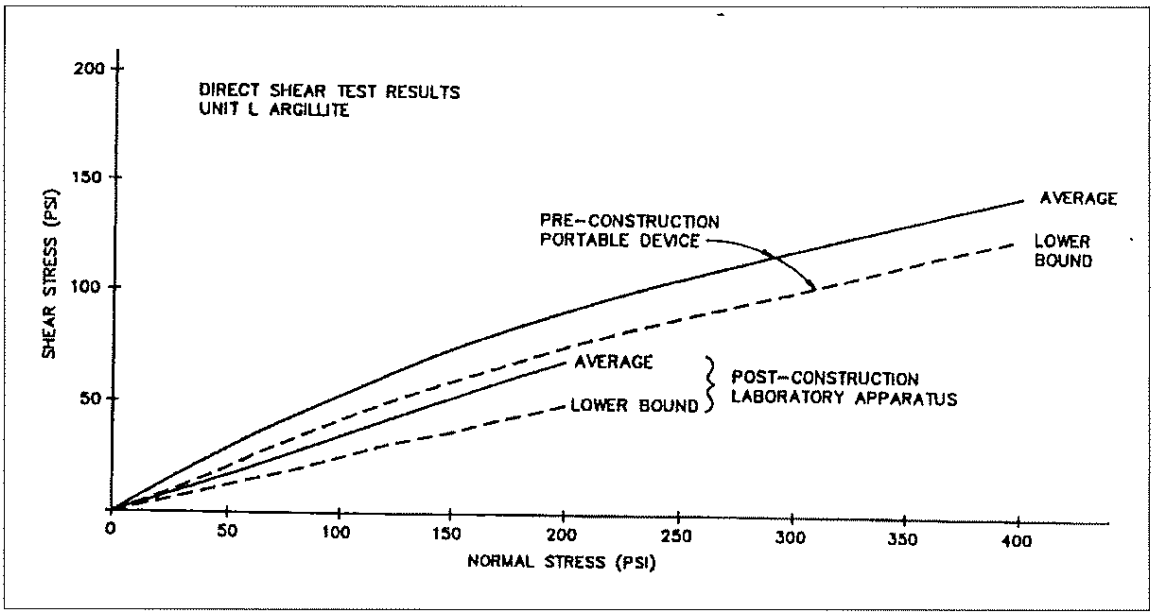

Figure UU-6. Shear strength test results for Unit L argillite

CASE HISTORY SUMMARY

Name of site or structure: Morris Sheppard Dam (Slab and Buttress)
Location: 75 miles (121 km) north of Ft. Worth, Texas, USA
Type of event: Sliding of the spillway section on the shale foundation (maximum displacement of 4-1/2 inches (114 mm))
Date of event: Probably occurred over a period of several years but was discovered during a routine inspection in December, 1986
Date of construction: Dam was completed in 1941

Loading: Normal operation

Description of site, structure and materials: Morris Sheppard Dam is founded on a shale foundation across the Brazos River and impounds a reservoir with a capacity of 570,000 acre feet (703×10^6 m^3). The dam rises 190 feet (57.9 m) above the river bed and is the highest flat slab-massive buttress type structure in the country (see figure MT-1). The total length of the dam is 2,740 feet (835.2 m) including the terminal dike. The non-embankment portion is 1,626 feet (495.6 m) long and is comprised of 40 buttresses on 40-foot (12.2-m) centers. The 720 foot (219.5 m) long spillway section is hollow between buttresses and has 12-inch (0.30 m) square holes to equalize the tailwater elevation within the hollow sections. The spillway is controlled by nine 73-foot (22.3-m) wide roof weir crest gates (bear trap). Review of data on the spillway indicated the gates were capable of passing only 60 percent of the Probable Maximum Flood (PMF).

Behavior under loading: During a Federal Energy Regulatory Commission (FERC) mandated inspection, it was discovered that the hollow spillway section of the dam had moved downstream on a slippage zone in the foundation. Metal survey points, installed twenty years earlier, had been placed on a straight axis along a catwalk positioned 90 feet (27.4 m) above normal tailwater inside the structure. The survey line indicated a downstream bow, across the length of the spillway section, with a maximum displacement of 4-1/2 inches (114 mm). The reservoir was lowered and core borings were made. Piezometers indicated that the hydrostatic uplift pressure under the spillway base slab was equivalent to 65 percent of the lake head. Observation of structural distress, such as cracks in the footings of the hollow spillway, confirmed that the dam had serious problems. When the dam was designed in the 30's, foundation stability was based on peak shear strengths and did not consider uplift pressures. Potential buoyancy of the lower portion of the hollow spillway section, coupled with the low resistance to sliding, added to the tendency to move downstream. Core drilling identified a longitudinal crack along the top the upstream cutoff, possibly caused by movement of several spillway bays during a flood shortly after completion. This could have allowed additional reservoir water to enter the foundation causing significant pressure beneath a weak shale layer about 30 feet (9.1 m) below the buttress footings along which sliding was occurring.

Consequences: Complete failure did not actually occur and releases were kept within the channel capacity as the reservoir was quickly lowered 13 feet (4.0 m) to improve the factor of safety. The structure carries the classification of "high hazard" due to its size and the magnitude of consequences predicted in the event of failure. Thousands of residents, hundreds of businesses, the Worth Boy Scout Ranch, the city of Granbury, and other substantial economic developments are located downstream. The reservoir is one of the largest in the state and is used for hydro-power production, water supply, irrigation, and recreation. Lowering the lake water surface for the two year repair period resulted in greatly reduced power production, interruption of irrigation and domestic water supplies, boat docks out of water, boat rental and fishing supply business curtailed, stores with fewer customers, and delayed residential and business development in the region.

Back Calculations: Piezometers, installed as part of an extensive geotechnical investigation, showed that uplift pressures, approaching reservoir pressure, had penetrated as much as 80 feet (24.4 m) downstream from the heel, beneath a weak shale zone located about 30 feet (9.1 m) below the buttress footings. Pressures as high 60 percent of reservoir head were measured 200 feet (61.0 m) downstream from the heel. Using the results of geotechnical tests on foundation material obtained during the piezometer installation, a coefficient of sliding friction of 0.40 was established. Stability calculations indicated that sliding frictional strength alone resulted in a factor of safety well below 1.0. An equivalent cohesion value, acting over the area of the sliding plane, of 1.69 k/ft^2 (12 lb/in^2, 0.08 MPa) was found necessary to bring the factor of safety up to near 1.0 (considered to be the appropriate value since the movements were thought to be very slow). This force was applied in future stability analyses as an effective cohesion value; this value is considered to be a combination of three-dimensional stress distribution and side-shear on foundation blocks. After drainage wells were installed, additional studies were conducted to determine the increase in stability achieved by the reductions in uplift pressures (see figure MT-2). The factor of safety against sliding was calculated as 1.5 after the first phase modifications were complete; subsequent modifications, such as addition of ballast to the spillway bays, would bring the factor of safety against sliding to 1.75.

Discussion: A network of piezometers, extensometers, inclinometers, along with a precision field survey system was installed to monitor any movement or change in uplift pressure while corrective actions were under way. Personnel compared weekly surveys with previous readings and at the height of activity, took measurements daily. An 8-foot (2.4 m) diameter opening was drilled though each of the 24 nine-foot (2.7-m) thick spillway buttresses to facilitate construction activities and to allow water to flow between critical spillway buttresses to eliminate buoyancy effects during high tailwater. Divers used a suction dredge under 30 feet (9.1 m) of water to remove 18,000 cubic yards (13,760 m^3) of mud and debris left in the bays during construction from 1938 to 1941. To create a platform for the geotechnical operations, contractors placed 66,000 cubic yards (50,450 m^3) of 3-inch (76 mm) crushed rock in the bays using a telescoping conveyor.

A series of 6-inch (152 mm) diameter relief wells, spaced every 13 feet (4.0 m) were planned, but some wells encountered pressure flows of up to 450 gallons per minute (28 l/s). This unexpected and excessive pressure called for additional wells in some bays. A total of 147 wells, some as large as 12-inches (305 mm) in diameter, were installed in an attempt to lower the hydro-static pressure causing the foundation uplift. During a subsequent construction phase, concrete ballast was planned to be added to increase the weight resisting the uplift forces.

References: Pullen, P.H., and R.A. Thompson III, "Averting the Risk of Dam Failure," Hydro Review, Vol. VIII, No. 5, October 1989, pp. 28-38.

Foster, J.L., "Evaluation of Foundations of Existing Concrete Dams on Rock," Tenth Annual USCOLD Lecture Series, U.S. Committee on Large Dams, March 1990.

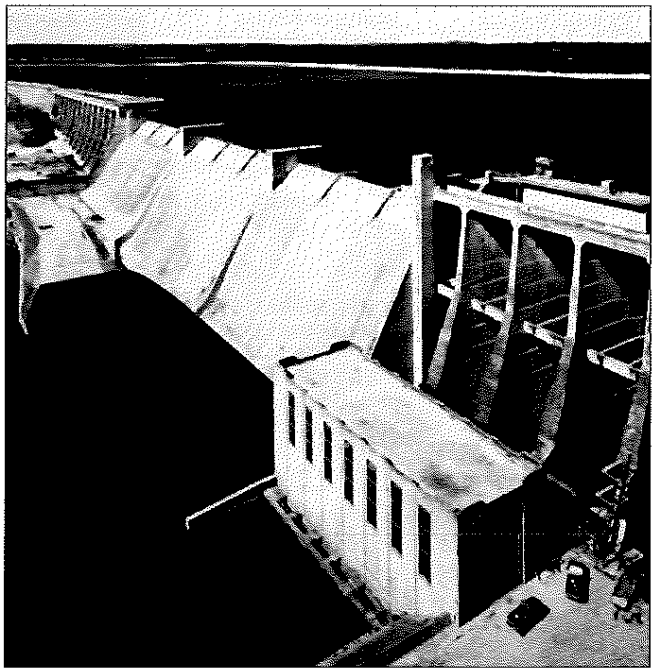

Figure MT-1. Photo of Morris Sheppard Dam (after Pullen and Thompson, 1989)

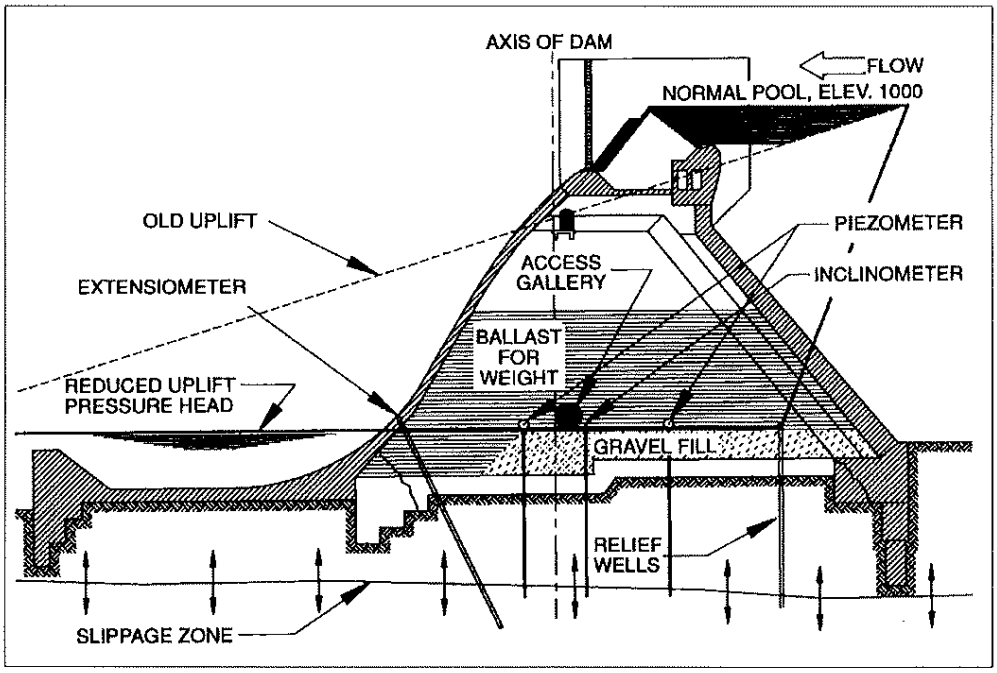

Figure MT-2. Section through spillway buttress at Morris Sheppard Dam (after Pullen and Thompson, 1989)

3.0 Concrete Dams Subjected to Earthquakes

No concrete dams are known to have failed catastrophically during an earthquake. This is comforting in itself, but also means that seismic failure modes for concrete dams are not well understood. However, several have been subjected to seismic shaking, and it is possible to learn from their response. The following table summarizes some of these dams. Detailed case summaries are included for Koyna (India) and Pacoima (California), since there is more information available for them.

Concrete Dams Subjected to Strong or Moderate Earthquake Shaking (with measured accelerations and/or estimated stresses)

Name	Location	Shaking date	Magnitude/ Distance	reservoir full?	Shaking level	Geometry of dam known?	Ground motions recorded?	Matl. properties measured?	Comments
Koyna gravity 103 m (338 ft) high 1963	India	1967	M 6.5 epicenter 13km (8.1mi)	11m (36 ft) below crest	.63 long 0.49 g trans 0.34 g vert in lower gallery	yes	yes, influenced by dam	yes sigmat = 2.4 MPa (348 lb/in^2)	significant cracking near change in section, 9.6 MPa (1400 lb/in^2)tension est. on d/s face
Lower Crystal Springs curved gravity 138 ft (42 m) high 1890	California	1906	M 8.3 fault 0.3 km (0.19 mi)	yes	no	yes	estimated 0.6 g	E_c=4.8 M lb/in^2 (33.1 GPa) E_f=0.9 M lb/in^2 (6.2 GPa)	max principal tensile stresses - 830 lb/in^2 (5.7 MPa) near abut d/s face, 590 lb/in^2 (4.1 MPa) upper center d/s face (st+dyn)
Big Tujunga arch 253 ft (77 m) high	California	1971 San Fernando	M 6.6 fault 32 km (19.8 mi)	95 ft (29 m) below crest	0.25 g on abutment	yes	seismo-scopes	E_c=4 M lb/in^2 (27.6 GPa) E_f=1.9-2.6 M lb/in^2 (13-18 GPa)	2.5 g calculated at crest
Santa Anita arch 230 ft (70 m) high	California	1971 San Fernando	M 6.6 fault 27 km (16.7 mi)	not sure	0.17 g above right abutment	yes	some	finite element analysis - no results	

Name	Location	Shaking date	Magnitude/ Distance	reservoir full?	Shaking level	Geometry of dam known?	Ground motions recorded?	Matl. properties measured?	Comments
Name	**Location**	**Shaking date**	**Magnitude/ Distance**	**Reservoir full?**	**Shaking levels**	**Geometry of dam known?**	**Ground motions recorded?**	**Matl. properties measured**	**Comments**
Pacoima arch 371 ft (113 m) high 1928	California	1971 San Fernando	M 6.6 epicenter 6km (3.7 mi)	148 ft (45m) below crest	1.25 g on left abutment (amplified by topography)	yes	some	E_c=3 M lb/in^2 (20.7 GPa) E_f=2 M lb/in^2 (13.8 GPa) & variable zones	opening of grouted joint near lt thrust block, 749 lb/in^2 (5.16 MPa) peak tensile stress est linear analysis
		1994 Northridge	M 6.7 epicenter 18km (11.2mi), fault 10 km (6.2 mi)	131 ft (40m) below crest	0.5 g at base, 2.0 g on sidewalls near crest	yes	yes		cracking, displacement of abutment rock, crest displacement 2 in (50 mm) u/s, jt open 2 in (50 mm) at lt thrust block
Hsinfengkiang diamond head buttress 105 m (344 ft) high 1959	Kuantung, China	1962	M 6.1 epicenter 5km (3.1 mi)	13 m (43 ft) below crest	no	yes	no	some analyses performed (linear elastic plane strain, and rocking of assumed top cracked part)	produced 82 m (269 ft) long cracks on 4 buttresses near change in section
		1975	M 4.5	not sure	0.61 g @ top 0.06g @ fdn		yes		
Dondo gravity 71.5 m (235 ft) high 1990	Japan	1995 Hanshin	M 7.2 epicenter 19km (11.8mi), fault 12 km (7.4 mi)	not sure	432 gal (0.43 g) center of crest, 78-157 gal (0.08-.16 g) on bedrock	yes	some	some finite element analyses - no results	temporary increased leakage, max joint displacement 0.05 mm (.002 in)

Name	Location	Shaking date	Magnitude/ Distance	reservoir full?	Shaking level	Geometry of dam known?	Ground motions recorded?	Matl. properties measured?	Comments
Name	**Location**	**Shaking date**	**Magnitude/ Distance**	**Reservoir full?**	**Shaking levels**	**Geometry of dam known?**	**Ground motions recorded?**	**Matl. properties measured**	**Comments**
Hitokura gravity 75 m (246 ft) high	Japan	1995 Hanshin	M7.2 epicenter 45km (27.9mi), fault 10 km (6.2 mi)	not sure	0.19 g at base	not sure	yes	not sure	no damage observed
Nagawado arch 155 m (509 ft) high	Japan	1984 Naganoken Seibu	M 6.8 epicenter 37km (22.9mi)	24 m (79 ft) below crest	0.04 g fdn, 0.2 g crest center, 0.25 g lt 1/4 point	not sure	yes	E_c=42.2 GPa (6.1 M lb/in^2) E_f=28.4 GPa (4.1 M lb/in^2)	
Ambiesta arch 59 m (194 ft) high	Italy	1976 Friuli	M 6.5 epicenter 22km (13.6mi)	not sure	0.33 g on abutment	yes	yes	some finite element analyses - no results	no damage

References

Hall, J.F., "The Dynamic and Earthquake Behaviour of Concrete Dams: Review of Experimental Behaviour and Observational Evidence," Soil Dynamics and Earthquake Engineering, Volume 7, No. 2, April 1988.

"Concrete Gravity Dams International Survey of Earthquake Experience (Interim)," Klohn-Crippen for BC Hydro, June 1996.

CASE HISTORY SUMMARY

Name of site or structure: Koyna Dam (Concrete Gravity Dam)
Location: India
Type of event: Subjected to large magnitude (6.5 Richter) earthquake
Date of events: December 11, 1967 earthquake
Date of construction (if applicable): Construction began in 1954 and was completed in 1963.

Loading: The dam was subjected to a nearby earthquake that registered 6.5 on the Richter scale and produced ground motion accelerations at the structure that were nearly ten times the design value. The epicenter of the earthquake was located within about 13 km (8 miles) of the dam and the focal depth was approximately 8 to 13 km (5 to 8 miles). The fault break was found to be only 2.8 km (1.8 miles) from the dam and the duration of strong shaking was approximately six seconds. Among the hundreds of aftershocks, six strong shocks with magnitudes ranging from 5.5 to 6.2 (Richter) occurred in the two weeks following the main disaster. The reservoir water level was 91.7 m (301 feet) deep (at elevation 662.6 m (2174 feet)), compared to crest elevation 673.9 m (2211 feet)) when the earthquake occurred, which is close to capacity.

Description of site, structure and materials: Koyna Dam is situated on the Koyna River, approximately forty miles inland from the Arabian Sea in the southwestern region of India. It was designed as a straight gravity dam that stands 103 m (338 feet) high and 853 m (2800 feet) long. It measures 14.8 m (48.5 feet) thick at the crest and 68.6 m (225 feet) thick at the base of the tallest section. The dam was constructed in 15.2-m (50-foot) wide monolith sections, with a 91.4-m (300-foot) wide overflow spillway located at the center of the structure. Figures KI-1 and KI-2 show the general features of the dam. Please note that 31 feet (9.44 m) must be added to the elevations shown on the figures to arrive at mean sea level values. Vertical joints between monoliths were not grouted but do contain copper water seals. Available information does not indicate that these joints were keyed, but the inferred independent response of the monoliths suggests they probably are not. Due to plan modifications that occurred during construction, Koyna Dam has an unconventional cross-sectional design for the nonoverflow monoliths, as illustrated in figure KI-2. The downstream faces of the non-overflow sections change slope from 0.153:1 to 0.725:1, at elevation 637.3 m (2091 feet), 36.6 m (120 feet) below the crest of the dam. A grout curtain spans the foundation to a depth of one-half to two-thirds of the water head over the foundation, and the presence of a drainage curtain is reported.

Koyna Dam was constructed primarily from rubble concrete. During construction, freshly-placed, nine-inch-thick layers of high-slump, air-entrained concrete were overlaid with corresponding nine-inch-thick layers of large rock rubble. The maximum diameter size of the rubble ranged from four to sixteen inches. These two layers were vibrated together to create one lift line, and then were finished according to methods established for conventional concrete lifts.

Conventional concrete was placed in 1.8-m (6-foot) thick layers along the upstream face of all monoliths to provide a more impervious barrier, and along the downstream face of the overflow monoliths for more durability.

The reservoir has a maximum storage capacity of 3.1×10^9 m^3 (2.5 million acre feet). The dam was constructed to abate monsoon flood waters, to provide irrigation water to the east, and to provide power to the west. Both the dam and reservoir are situated upon Deccan Volcanic ("Trap") terrain, which is a byproduct of extensive lava flows from the Cretaceous-eocene period. These traps are primarily basaltic in composition have a total thickness that exceeds 3000 feet at portions the dam site. Although the basalt is very compact, it contains very thin temperature cracks that are filled with clay and secondary materials. These cracks were consolidation grouted to support the structure. Below the traps lie uneven topography primarily consisting of Dharwarian metamorphites, with gneisses and granites.

Until the Koyna earthquake, the geology of the Indian Peninsula was considered stable and non-seismic. Although small ground motions were noted once the reservoir was filled, no direct correlation has been established among reservoir impoundment and seismic activity.

Behavior under loading: The accelerograph near the base of the dam in monolith 13 failed to operate. However, the Koyna earthquake was recorded by strong motion accelerographs located in the gallery of a monolith near the right abutment (monolith 1A in figure KI-1). This instrument recorded peak ground accelerations of 0.63 g in the horizontal direction parallel to the dam axis, 0.49 g in the horizontal direction transverse to the dam axis, and 0.34 g in the vertical direction. The spectrum intensity, defined as the area under the velocity response spectrum between periods of 0.1 and 2.5 seconds, ranged from a minimum of 0.42 m (1.39 feet) in the transverse horizontal direction to 1.34 m (4.39 feet) in the longitudinal, horizontal direction. The acceleration spectrum intensity, defined as the area under the acceleration response spectrum between periods of 0.1 and 0.5 seconds, was about 0.375 m/s (1.23 ft/s). Since the accelerogram was placed near the base of a securely-grounded monolith, that data were assumed to represent ground motion at the site. Further evaluations of the data have indicated the motions were probably influenced by the structure.

Although the structure was only designed to withstand seismic accelerations of 0.05 g, Koyna Dam survived the earthquake without any loss of water. It was, however, significantly damaged. Deep horizontal cracks formed throughout the upstream and downstream faces of most nonoverflow monoliths. A majority of these horizontal cracks on both faces were located at the elevation of the downstream slope change (36.6 m (120 feet) below the crest), where a stress concentration is expected to occur. The most severely cracked monolith was asymmetrically designed. Half of the section served for overflow and the other half for non-overflow. Unfortunately, the depths and thicknesses of the cracks were not reported. However, severe leakage was noted on the downstream face of a monolith to the left of the spillway, and traces of seepage were observed throughout the left side of the structure. The overflow monoliths were generally not damaged.

The visual inspection of the vertical joints revealed that relative movement had occurred between adjacent monoliths. The concrete had spalled and seepage increased significantly along these joints. Such differential movement was expected, since the monoliths have different heights and thus different periods of vibration and motion. Cores that were extracted post-disaster at the foundation-concrete interface exhibited good bond. Uplift pressures did not significantly change after the earthquake.

Consequences: Although the dam safely withstood the seismic activity, the reservoir was lowered for inspection and repair and the entire dam was strengthened. Large cracks were grouted with epoxy resin injections. The upper portion of the taller non-overflow monoliths were vertically prestressed with cables. Concrete buttresses were added to the downstream face along the non-overflow monoliths that increased almost the entire width of the cross-section by 6.1 to 9.1 m (20 to 30 feet).

Although not related to the dam, approximately 180 people were killed and 2200 were injured from the catastrophe. The nearby town of Koynanagar, which served as the construction camp for the dam, was demolished.

Back calculations:

Methods of analysis

Two-dimensional finite element analyses were conducted at the University of California, Berkeley that modeled the performance of Koyna Dam (Chakrabarti and Chopra, 1972). Since the length of the actual structure measures eight times the height, horizontal movement parallel to the dam axis was considered negligible and thus omitted. During the earthquake, the vertical joints between monoliths, which were not grouted or bonded, moved somewhat independently. Thus, a two-dimensional model that utilized the transverse and vertical components of individual monolith vibrations adequately represented actual movements. One set of analyses was based on the mode-superposition method, including the first four modes of vibration. the dynamic interaction between both the dam and reservoir and dam and foundation were excluded. Accordingly, the foundation was assumed to be rigid. Static and dynamic stresses were superimposed. Both the nonoverflow and overflow monoliths were modeled. A second set of analyses accounted for hydrodynamic interaction and the compressibility of water (presumably using a frequency domain solution) considering only the first mode of vibration and the transverse ground motion. Only the nonoverflow monoliths were modeled for this latter analysis. Finally, as part of the efforts to complete this report, a state-of-the art two-dimensional analysis was performed, accounting for hydrodynamic interaction and full foundation interaction, using computer program EAGD-SLIDE (Chavez and Fenves, 1994).

Material properties

Although different concrete mixes were used in the dam, the structure was considered homogenous for analysis purposes. Input parameters to the model included a five percent damping ratio, and the following material properties:

concrete modulus of elasticity:	31 GPa (4.5 x 10^6 lb/in^2)
Poisson's ratio:	0.20
unit weight:	2642 kg/m^3 (165 lb/ft^3)

The strengths of the concrete used to compare to analysis results were based on previously reported compressive strengths for the concrete mixes in the structure. Four concrete mixes were used in the dam at various elevations. However, Mix No. 1 was excluded from the study since it was used in only the lowest sections of the dam. Mix No. 3 was used in the area of the change in slope. The tensile strength was assumed as ten percent of the compressive strengths. The concrete properties are as follows:

Mix No.	Elevation Used (m (ft))	Compressive Strength (MPa (lb/in^2))	Assumed Tensile Strength (MPa (lb/in^2))
2	Up to 616 (2021)	28.3 (4100)	2.8 (410)
3	616-667.8 (2021-2191)	24.1 (3500)	2.4 (350)
4	667.8-673.9 (2191-2211)	20.0 (2900)	2.0 (290)

Results without hydrodynamic or foundation interaction

The no reservoir fixed-foundation finite element model of the non-overflow section consisted of 136 quadrilateral elements and 162 nodal points. Constraints at the base allowed for 306 degrees of freedom. The periods of the first four modes of vibration were 0.326, 0.122, 0.093, and 0.063 seconds. For the non-overflow section model, the time history of displacement at the crest produced a maximum value of 31 mm (1.23 in). The maximum calculated principal stresses were located near the slope change on the downstream face at elevation 637.3 m (2091 feet), and occurred at approximately 4.25 seconds of a 10-second analysis. The largest compressive stress in the section exceeded 8.6 MPa (1250 lb/in^2). The maximum tensile principal stresses on the upstream face and downstream face exceeded 3.45 MPa (500 lb/in^2) and 6.90 MPa (1000 lb/in^2), respectively, exceeding the assumed tensile strength of the concrete. This correlates with cracking as it was observed in the structure.

Results with fixed foundation and hydrodynamic interaction

With the hydrodynamic effects included for the nonoverflow monoliths, the vibration period of the dam due to the reservoir lengthened since the time history of the crest displacement provided a longer dominant period. The maximum crest displacement, also occurring at approximately 4.5 seconds of the 10 second analysis, increased to a distance of 45 mm (1.77 inches). Since the peak crest displacement of the full reservoir model was 45 percent greater than the that for the no-reservoir model (31 mm to 45 mm, or 1.77 in to 1.23 in), the total principal stresses were estimated as approximately 145 percent of the previously provided values. Although this estimate includes both static and dynamic stresses, the static stresses at the peak stress locations were relatively small and contribute little to the total stress. Based on these assumptions, peak

tensile stresses at the slope change were estimated to be 4.83 MPa (700 lb/in^2) at the upstream face and 9.65 MPa (1400 lb/in^2) at the downstream face, and maximum compressive stress was approximately 12.41 MPa (1800 lb/in^2). These values exceed the strength values for concrete Mix No. 3.

The overflow monolith model

The overflow monolith section was analyzed assuming a rigid foundation without hydrodynamic loadings. The periods of the first four modes were 0.205, 0.088, 0.78, and 0.051 seconds. Tensile stresses in the overflow section were also considerably smaller. A maximum tensile stress at the upstream face of 2.07 MPa (300 lb/in^2) occurred after 3.825 seconds, compared to 3.45 MPa (500 lb/in^2) for the corresponding non-overflow model. There was no concentration of tensile stress along the downstream face of the structure. A maximum tensile stress of 1.72 MPa (250 lb/in^2) occurred at this face after 2.90 seconds, compared to 6.90 MPa (1000 lb/in^2) for the non-overflow model. These values are slightly less than the tensile strength of Mix No. 3. If roughly the same percentage increase is assumed to estimate the effects of hydrodynamic interaction as was used for the non-overflow model, the results indicate that the maximum stress would be slightly less than the strength of the concrete on downstream face , and slightly greater than the strength of the concrete on the upstream face. Accordingly, little or no cracking would be expected at the non-overflow monoliths, which is consistent with the actual damage to the structure.

Results from EAGD-SLIDE

The nonoverflow monoliths were modeled using the same geometry and material properties as used by Chopra and Chakrabarti (1971). The foundation was excavated up to 18 m (60 ft) to found the dam entirely on massive basalts. Laboratory tests on core samples from this rock type yielded modulus values in excess of 69 GPa (10x10^6 lb/in^2). Heuze (1980) indicates the in situ rock modulus should be between 0.2 and 0.6 times this value, with the average multiplier being about 0.4. Thus, it is reasonable to use the same value for the foundation modulus as the concrete modulus of 31 GPa (4.5x10^6 lb/in^2). This value was used in the analyses along with a Poison's Ratio of 0.33 to take advantage of an existing impedance matrix. The reservoir wave reflection coefficient, a, was taken as 0.8, as a reasonably conservative value based on field testing at other sites.

The results of the analysis are shown in figure KI-3. As can be seen from the stress contours, the maximum tensile stresses just reached 2.75 MPa (400 lb/in^2) on both the upstream face and downstream face near the change in section. This is less than one-third the value estimated from the previous finite element studies. Although these stress values exceed the estimated concrete tensile strength by a small amount, clearly less damage would be expected based on these results than the previous finite element studies.

Discussion: These dynamic analyses support the theory that the critical stresses due to earthquake loading occur in the upper portion of gravity dams. It is common practice in India to gradually decrease the strength of concrete in gravity dams at higher elevations, in accordance

with the theory that static loading decreases with increasing elevation. This practice is not recommended when considering seismic loading conditions.

Chopra and Chakrabarti performed some additional analyses to determine whether the unique cross-sectional design of the non-overflow sections of Koyna Dam made the structure more susceptible to damage. The cross-sectional design of Pine Flat Dam near Fresno, California, which is about 20 percent higher than Koyna and has no abrupt corners, was selected for this purpose. It was modeled with a rigid foundation and no reservoir, and the Koyna ground motions were applied. The maximum tensile stresses were similar to what was calculated for Koyna Dam (about 0.69 MPa (1000 lb/in^2) less on the downstream face, with a similar increase on the upstream face). Strengthened Koyna Dam was also modeled in a similar fashion. The results indicated the maximum tensile stresses were reduced to about 60% of the stress level without the strengthening.

In comparing traditional finite element analyses with EAGD-SLIDE analyses, the latter produces tensile stresses much lower based on reasonably conservative assumptions. Both approached estimated tensile stresses in excess of the concrete tensile strength, but the level of damage would be expected to be much less when using the results from EAGD-SLIDE. There is no real way to tell which analyses produce the more correct results. One should be careful when using results of EAGD-SLIDE analyses, as calculated stresses in excess of the tensile strength could indicate the potential for significant cracking.

Several papers have been written describing nonlinear analysis of Koyna Dam in an attempt to model the observed cracking. These studies are not described here because they involved specialized procedures that would require lengthy discussion to describe, and because Reclamation is not routinely performing nonlinear analyses for concrete dams at this point in time.

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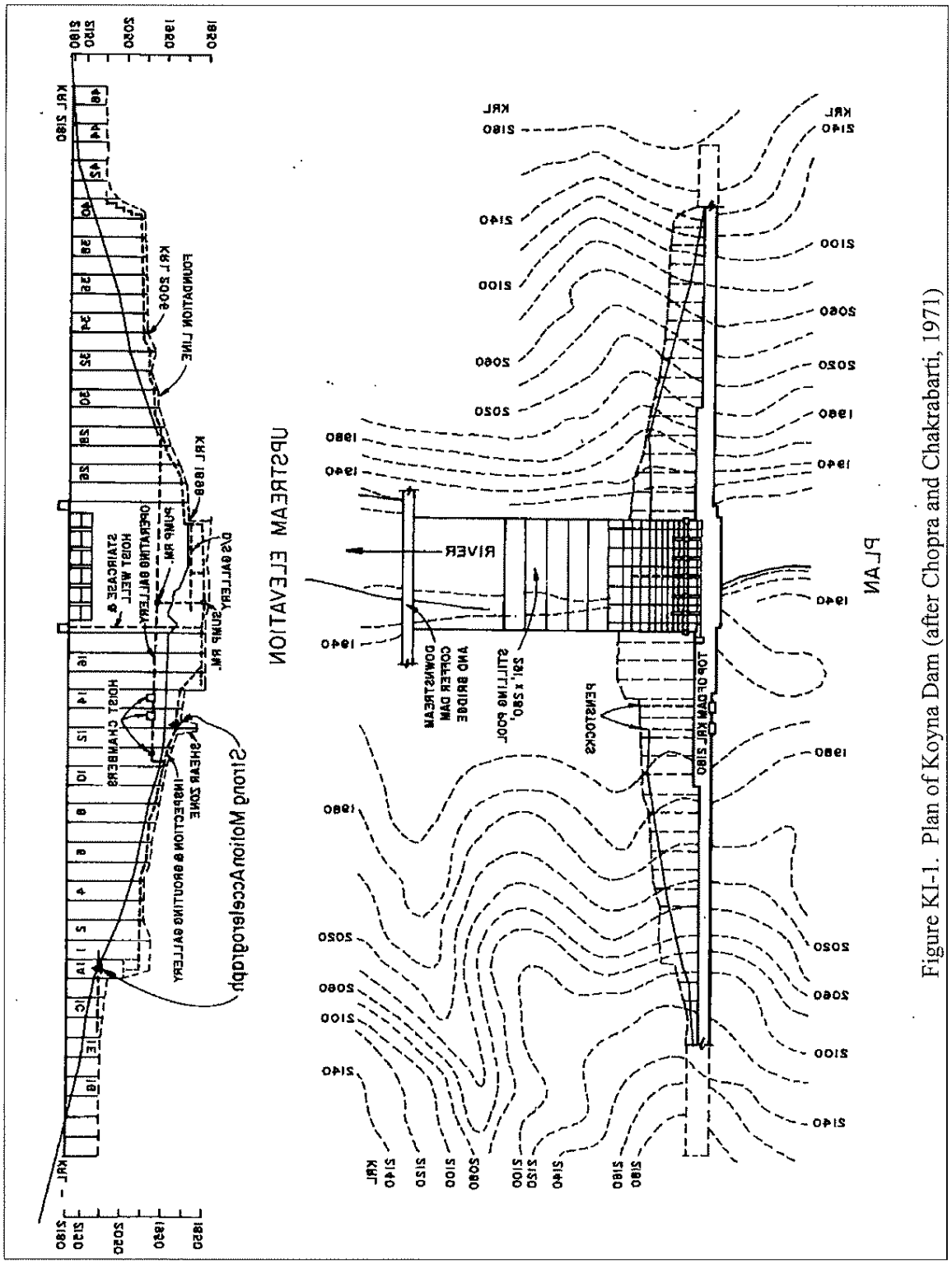

Figure KI-1. Plan of Koyna Dam (after Chopra and Chakrabarti, 1971)

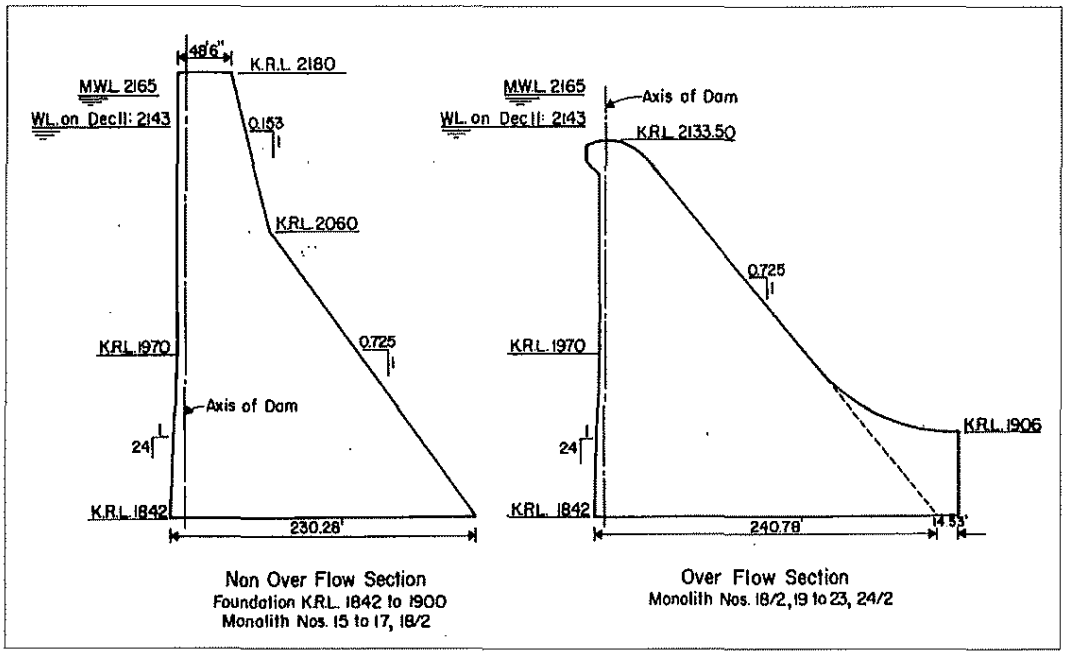

Figure KI-2. Sections of Koyna Dam (after Chopra and Chakrabarti, 1971)

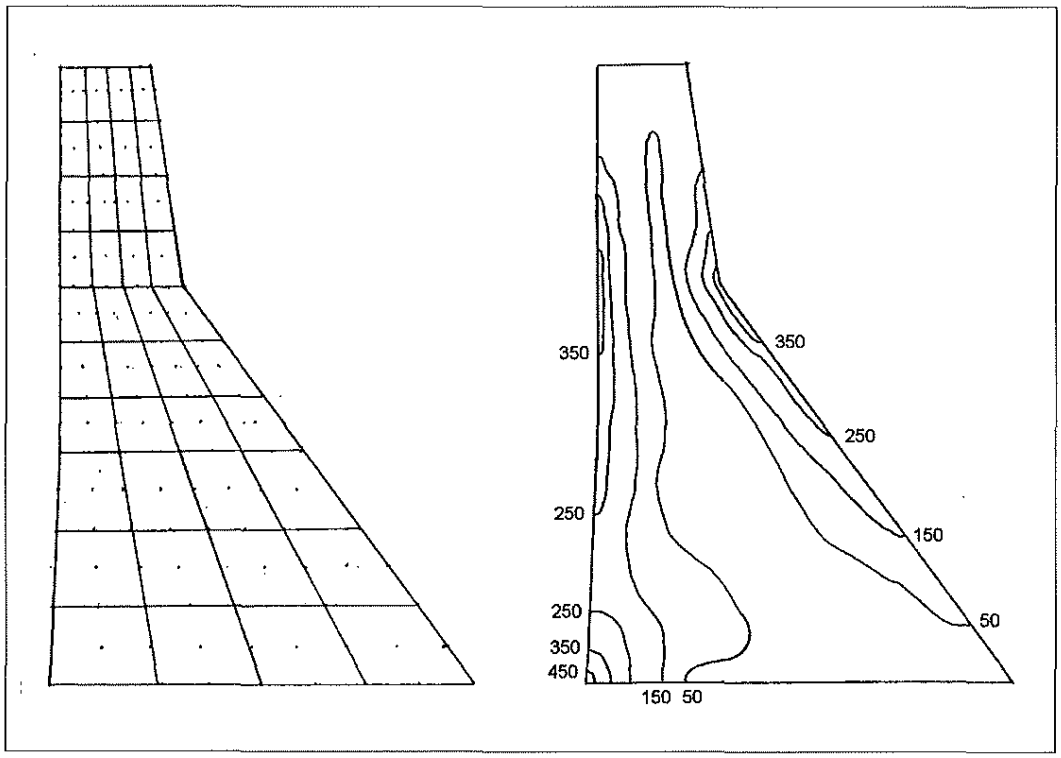

Figure KI-3. EAGD-SLIDE finite element mesh (left) and calculated envelope contours of maximum static + dynamic principal (tensile) stresses (right - lb/in^2)

CASE HISTORY SUMMARY

Name of site or structure: Pacoima Dam (Arch Dam)
Location: Southern California, USA - In San Gabriel Mountains, north of Los Angeles
Type of event: Subjected to large magnitude earthquakes (6.7) at small epicentral distances.
Date of events: February 9, 1971 San Fernando Earthquake and January 17,1994 Northridge Earthquake

Date of construction: Construction was completed in 1929.

Loading: The dam survived two nearby earthquakes with limited observable damage. However, the dam is a flood control dam and the reservoir was low in both cases

1971 San Fernando earthquake
The 1971 San Fernando earthquake, which was generated by thrust faulting, registered a Richter magnitude of 6.6. The epicenter was four miles (6 km) north of the structure and focal depth was approximately 8 miles (13 km). It originated on a fault plane that passes three miles below the dam and lasted approximately eight seconds. During the disaster, the reservoir water level was approximately elevation 1867, 148 feet below the crest of the dam.

1994 Northridge earthquake
The magnitude of the 1994 Northridge earthquake measured as M_S 6.8. The epicenter was located approximately eleven miles (18 km) southwest of the structure. The reservoir level was similar to that during the San Fernando earthquake, at about elevation 1884 feet (574.2 m) or 131 feet (39.9 m) below the crest.

Description of site, structure and materials: Pacoima Dam is situated on Pacoima Creek in the San Gabriel Mountains, approximately 4.5 miles northeast of San Fernando, Los Angeles County, California. The dam is a 370-foot (112.8 m) high flood control, concrete arch dam, measuring 10.4 feet (3.2 m) thick at the crest and 99 feet (30.2 m) thick at the base. The length along the dam axis at the crest is 589 feet (179.5 m) (see figure PC-1). The dam was constructed in twelve vertical cantilever monoliths that are separated by eleven evenly-spaced contraction joints and interlocked with 12 inch (0.3 m) deep beveled, grouted keys. The left abutment is supported by a low gravity, concrete, thrust block, 60 feet (18.3 m) tall at the contraction joint abutting the arch. Core tests indicate that the 28-day compressive strength of the concrete was 2600 lb/in^2 (17.9 MPa). Pacoima Dam was only designed for full-reservoir loading. Earthquake loads were not considered.

A 40-year physical assessment of the structure conducted in 1967-68 indicated an average compressive strength of 4900 lb/in^2 (33.8 MPa) for the concrete. No indication of alkali-aggregate reaction or other deterioration was found. Fractures and weak zones discovered in the foundation below the left abutment raised concerns for stability under peak reservoir and severe seismic loading conditions.

The reservoir has a capacity of 10,000 acre feet (12.3×10^6 m^3) at maximum water surface and its principle roles are flood control and conservation. It is located in an extremely narrow, steep-walled canyon of predominantly metamorphic-gneissic formations. The region is prone to faults and shearing in all directions, though most lie roughly parallel to the mountain ranges.

Behavior under loading: Throughout both earthquakes, Pacoima Dam maintained functional capabilities and remains in use today. In both cases, ground motions triggered subsequent rockslides that slightly impacted the structure.

1971 San Fernando earthquake

Accelerometers located 52 feet (15.8 m) above the dam crest on the left abutment recorded earthquake movement. Unusually high accelerations, 0.7 g in the vertical direction and 1.25 g in both horizontal directions, were recorded. These results most likely include the amplification of seismic waves due to canyon topography. Convolution studies of the damsite which included 80 modes of vibration, each damped at 10% of critical, and measured concrete and foundation properties produced base accelerations to be applied to the finite element models in the range of 0.4 g horizontal and 0.2 g vertical.

The only significant visual damage to the dam resulting from the earthquake was an opening of a previously grouted contraction joint and a large crack in the thrust block. The contraction joint separated the arch from the left thrust block at the crest with a 0.38 inch (10 mm) gap. The arch and thrust block separation began at the crest and extended downward along the arch-thrust block interface approximately 50 feet (15.2 m), where it ended at a horizontal contraction joint. The crack in the thrust block initiated along a horizontal joint and extended along this joint toward the abutment for about five feet (1.5 m), and then angled down approximately 55 degrees to intersect the abutment rock at elevation 1965 feet (598.9 m).

The left abutment of the dam was severely damaged. Extensive cracks were found in the gunite cover and the downstream slope slumped over a 72,000 square foot (6690 m^2) area, moving about 8 inches (203 mm) vertically and 10 inches (254 mm) horizontally. Slight cracking and spalling of the gunite cover also occurred along the right abutment.

Despite the intensity of the 1971 acceleration measurements, no visible cracks were detected on the dam and no differential movement was detected between adjacent blocks. Laboratory tests performed on concrete core samples extracted from the dam indicated no change in the physical condition of the concrete.

Seismic surveys indicated that a part of the San Gabriel mountain block to the left of the dam had been thrust up 4.2 feet (1.28 m) vertically and 6.6 feet (2.01 m) horizontally in a southwesterly direction. This geographical deformation resulted in an overall narrowing of the canyon that imposed new compressive forces on the dam. The distance between the left and right abutments decreased 0.49 inch (12.4 mm) and the axis of the dam rotated 30 seconds clockwise relative to the baseline. The entire structure tilted downward from the crest of the right abutment 0.68 inches (17.3 mm) relative to the left abutment.

1994 Northridge earthquake

Accelerometers, provided by the California Division of Mines and Geology after the San Fernando earthquake, recorded accelerations of the 1994 earthquake. Along the crest at the left abutment, peak accelerations of 1.6 g in the horizontal direction and 1.2 g in the vertical direction were observed. Acceleration in the radial direction along the crest reached 2.3 g. These accelerations, among the highest on record to date, also include some amplification due to canyon topography. Significantly smaller peak accelerations of 0.5 g in the horizontal direction and 0.4 g in the vertical direction were recorded at the base of the structure.

Post-earthquake inspections revealed the unusually clean appearance of the vertical contraction joints, indicating that they temporarily opened during movement, and then closed under static forces. Permanent vertical offsets appeared along most of the vertical joints at the crest of the dam, with the elevation of each block crest slightly declining from the left to right abutment. One joint separation remained at the arch-thrust block interface along the left abutment, measuring two inches (50 mm) at the crest and one-fourth of an inch (6 mm) at the base. The thrust block and the accompanying rock foundation tilted approximately three degrees downslope from the dam, creating a long, diagonal crack in the thrust block near the thrust block joint connection to the arch. A one half-inch (13 mm) horizontal offset was noted 48 feet (14.6 m) below the crest, with the upper section moving downstream relative to the lower section. Surface cracks were also noted in the concrete throughout the arch and the thrust block. It appears the thrust block and underlying rock mass moved away from the arch. Small vertical offsets were observed at each joint at the crest, indicating the right blocks probably dropped down relative the left blocks. Surveys indicated the dam crest moved upstream by as much as 0.5 inch (12 mm) in addition to the 0.6 inch upstream movement that occurred during the 1971 earthquake. The left side of the arch moved slightly to the left as a result of loss of support from the thrust block.

Piezometers indicated there was a redistribution of pore pressures. The seepage increased immediately after the earthquake, but reduced with time. There was extensive cracking and displacement of the gunite cover on the left abutment rock surface. The left abutment moved as much as 16 to 19 inches (400 to 480 mm) in a southwesterly direction, and 12 inches (300 mm) downward at the surface, and 3 to 4 inches (75 to 100 mm) at depth indicating movement of rock blocks. Elongation and overstressing of the tendons near the thrust block probably occurred. Near vertical stress relief joints parallel to the canyon downstream of the right abutment were open to a width of several feet. It was found that the concrete spillway tunnel lining was severely cracked and displaced in a zone about 20 feet (6 m) long, along an apparent geologic

discontinuity. Extreme landslide damage resulted from the Northridge earthquake. Rock falls damaged the spillway and the shotcrete covering the dam abutments. Hundreds of thousands of cubic meters of rock debris slid into Pacoima Canyon and temporarily blocked the access road to the dam.

Consequences: Minor repairs were made to the dam after each earthquake. Repairs after the San Fernando disaster included grouting of the open joint on the left abutment, additional grouting of the left abutment, patching of the gunite cracks, removal of loose debris, and installation of relief drains downstream of the dam. Post-tensioned rock anchors were also installed to stabilize the rock mass at the thrust block. Repairs to damage sustained from the Northridge earthquake included grouting of surface cracks and joints, removal of debris, and additional stabilization of the left abutment and thrust block. Although much of the local infrastructure was destroyed and many lives were lost during both earthquakes, no loss of life or local damage can be directly attributed to the performance of the dam.

Back calculations: Foundation and finite element structural analyses have been performed to simulate conditions at Pacoima Dam during shaking and to evaluate the performance of the structure during both earthquakes.

<u>1971 San Fernando earthquake</u>
Boundaries of the disturbed zones on the left abutment were established through an exploratory program. Idealized "planes" representing irregular fracture zones along which movement defined three potentially unstable rock masses, one of which underlies the thrust block. Back estimates using three-dimensional stereographic techniques and pseudo-static dynamic loads (corresponding to 0.3 g horizontal and 0.15 g vertical) indicate the shear strength of these planes must have corresponded to a friction angle of about 47°. Since movement had indeed occurred, tendons were designed and installed for future large seismic loading events.

A linear elastic stress analysis was initially performed to determine the severity of stress in the dam during the earthquake (Sharma and Swanson, 1979, Hall 1988). The three-dimensional finite element (3-D FEM) model consisted of hexahedron elements that conformed as closely as possible to the actual dimensions of the structure. The thickness of the dam was modeled by one row of elements for the top thirty feet of the structure (elevation 1985 - 2015 feet ((605.0 - 614.2 m)), and by two rows of elements below (elevation 1650 - 1985 feet (502.9 - 605.0 m)). Extensive portions of the abutments and foundation were included in the model (presumably using massless elements) to account for foundation-dam interaction.

Analyses were conducted using recorded accelerations and assuming the reservoir level at the time of the earthquake. The previously reported acceleration amplitudes that were recorded on the ridge of the left abutment were scaled to 2/3 of their measured values since they appeared to be influenced by topography. Ground motions were uniformly applied to the boundaries of the foundation that was provided in the model. Water was assumed as incompressible and was represented by lumped added masses at the upstream nodes calculated by Westergaard's equation. Calculations were performed using five percent damping.

The following moduli of elasticity, based on the results of a 1971 field program, were used:

modulus of elasticity (dam) = 3.00×10^6 lb/in^2 (20.7 GPa)
modulus of elasticity (below el. 1650) = 2.00×10^6 lb/in^2 (13.8 GPa)
modulus of elasticity (right abutment) = 1.75×10^6 (12.1 GPa)
modulus of elasticity (lower left abutment) = 1.50×10^6 lb/in^2 (10.3 GPa)
modulus of elasticity (upper left abutment) = 0.5×10^6 lb/in^2 (3.4 GPa)
modulus of elasticity (disturbed portion of left abutment) = 0.05×10^6 lb/in^2 (0.34 GPa)

The ultimate average compressive strength of the concrete, determined from cylindrical samples, was approximately 4200 lb/in^2 (29.0 MPa). The allowable strengths of the Pacoima Dam concrete used for comparison were 900 lb/in^2 (6.20 MPa) for static compression and 180 lb/in^2 (1.24 MPa) for static tension (based on f_c' = 3600 lb/in^2 (17.93 MPa)). The dynamic tensile strength would probably be in the range of 600-800 lb/in^2 (4.14-5.52 MPa) although this was not actually determined or discussed in any detail.

Given these parameters, the 3D-FEM analysis method produced the following maximum total (static excluding temperature plus dynamic) stresses:

compressive stress (arch): 915 lb/in^2 (6.31 MPa) (u/s face near crest at crown)
tensile stress (arch): 750 lb/in^2 (5.17 MPa) (u/s face near crest at crown)
compressive stress (cantilever): 480 lb/in^2 (3.31 MPa) (u/s face at el. 1750 ft (533.4 m) near crown)
tensile stress (cantilever): 250 lb/in^2 (76.2 MPa) (u/s face at el. 1925 ft (586.7 m) near left 1/4 point)

The tensile arch stress values probably exceed the allowable tensile strength of the structure. According to this result, cracking could have been expected. However, none was observed. This could be due to a number of reasons: (1) the contraction joints opened and relieved the tensile stresses without unduly increasing the cantilever stresses, (2) applying uniform ground motions to the entire base of the model is known to be conservative and the actual base input motions were not known, and/or (3) the canyon walls converged slightly which may have produced additional static stresses not accounted for in the finite element analysis.

A nonlinear analysis that accounted for potential predetermined horizontal cracking planes and opening of contraction joints was later performed (Hall, 1988). This finite element model considered nonlinearities due to cracking and joint openings. The dam was modeled with shell elements which were interconnected by special joint elements. The foundation elements were assumed massless and the reservoir was represented by an incompressible water domain. The analysis used input parameters for ground motion and damping as described for the previous study. The parameters used in the analysis are as follows:

modulus of elasticity (dam) = 3.00×10^6 lb/in^2 (20.7 GPa)
modulus of elasticity (foundation) = 2.00×10^6 lb/in^2 (13.8 GPa)
tensile strength of grouted contraction joint = 300 lb/in^2 (2.06 MPa)
tensile strength of horizontal cracking planes = 450 lb/in^2 (3.10 MPa)

These modulus values produced better agreement with the fundamental frequencies measured by forced vibration tests than the values used in the first finite element analysis. The computed nonlinear response exhibited pronounced opening and separation of the contraction joints in the upper half of the dam, reaching a maximum of 0.9 inch (23 mm) at the crest. Although the compressive stresses went up on the contact side, the maximum compression was within the linear range (1840 lb/in^2 (12.69 MPa)). The joint opening also resulted in some load transfer to the cantilevers indicating potential for minor cracking at one location and a maximum cantilever compressive stress of 915 lb/in^2 (6.31 MPa).

1994 Northridge earthquake

Although additional foundation studies were performed for this event, no documentation could be located. The response of Pacoima Dam to the Northridge earthquake was analyzed with a modified version of the ADAP-88 finite element computer program developed at the University of California, Berkeley (Fenves and Mojtahedi). The program was modified to allow for standard 3-D solid elements throughout the model, to include nonlinear joint elements, and to permit application of non-uniform free-field motion through displacement histories at the dam-foundation interface so that the strong motion records obtained by the California Division of Mines and Geology accelerographs on the canyon walls and dam could be used.

The Pacoima Dam model consisted of 588 eight-node, 3-D elements that provided approximately 3600 degrees of freedom. Only five vertical contraction joints were included in the model, one of which represented the dam-thrust block interface. Although the dam has eleven of these joints, it was determined that it was not necessary to include all joints in order to capture the earthquake response and effects of joint opening. Two horizontal joints, located at 97 feet (29.6 m) and 202 feet (61.6 m) above the base, span the entire width of the model structure between abutments. The dam was modeled with three 3-D elements through the thickness. This also required 3 joint elements through the thickness to simulate the contraction joints. To provide for keyed joints, tangential slippage of the joints is not permitted. However, this assumption may not be valid for the horizontal and abutment joints. Ten percent Rayleigh damping was assumed for the dam-foundation system.

The following material properties for the Pacoima Dam concrete, determined from tests on core samples, were used for input parameters:

modulus of elasticity = 2.40 x 10^6 lb/in^2 (16.5 GPa)
Poisson's ratio = 0.20
unit weight = 150 lb/ft^3 (2403 kg/m^3)

To account for the effects of dam-foundation rock interaction, the model included a foundation rock region that extended to a depth approximately equal to the height of the dam. The foundation rock was considered massless. Cases were run assuming both uniform and non-uniform free-field motion. For analyses assuming uniform free-field ground motion, the ground motion was applied at the rigid base of the foundation model. For the cases of non-uniform free-field motion, free-field ground displacements were provided at the nodes along the dam-foundation rock interface.

The material properties for the foundation rock, extracted from previous studies, used in the analysis were:

modulus of elasticity = 2.00 x 10^6 lb/in^2 (13.8 GPa)
Poison's ratio = 0.20

The hydrodynamic pressure of the water on the dam is represented by an added mass matrix that considers water incompressible based on a fluid element formulation. Accelerations recorded from accelerograms in the radial direction near at the upper left abutment (at 80 percent of the dam's height) and dam base were applied to the model for the case of non-uniform ground motions. To account for topographic effects of the canyon at various elevations, a linear interpolation of acceleration between recorded motions at the base and the left abutment was used to predict motion at various elevations of the canyon. Additional accelerographs on the abutments were triggered, but the accelerations were so high that the acceleration traces overwrote each other and could not be interpreted. The calculated accelerations were compared to measured accelerations for an instrument on the dam at the left quarter point and about 80% of the dam height.

With these input parameters, several case loads were analyzed to compare the behavior of the dam during the 1994 Northridge Earthquake relative to: (1) uniform vs. non-uniform input motions, (2) allowing the joints to open vs. constraining the joints to be closed, and (3) including the support from the thrust block vs. neglecting this effect. Static loading excluding temperature was included.

In the case of uniform motions, using 2/3 upper left abutment motions matched the measured dam accelerations best, although a large amplitude acceleration cycle not present in the recorded dam response was calculated. When the vertical contraction joints are allowed to open under uniform input motion, three large acceleration spikes not present in the recorded motion were

calculated, probably due to impact as the contraction joints in the model closed. Envelopes of maximum tensile stresses (looking downstream) are shown in figures PC-2 (uniform motion, no contraction joints) and PC-3 (uniform motion, with contraction joints). The opening of the contraction joints significantly reduced the maximum arch tensile stresses, but increased the cantilever tensile stresses.

Using non-uniform motions, the measured dam accelerations most closely matched the recorded values. Non-uniform motions without joint opening resulted in a significantly different stress distribution than with uniform motions. With the non-uniform motions, tensile arch stresses exceeding 3000 lb/in^2 (20.68 MPa) were calculated near the upper right abutment on both faces, and tensile cantilever stresses reaching 1400 lb/in^2 (9.65 MPa) were calculated in the lower right portion of the dam. The large stresses near the abutment were caused by the relative displacements of the non-uniform free-field motion at the interface. Of the cases studied, the response of Pacoima Dam during the Northridge earthquake was most realistically modeled with non-uniform free-field motion, open vertical contraction joints, and open lift joints. Envelopes of maximum tensile stresses are shown in figure PC-4 for this case. The arch stresses were reduced by the opening of contraction joints, but were not affected by the opening of the horizontal joints. The cantilever stresses on the upstream face were not affected by the opening of the horizontal joints due to the small magnitude of these stresses. However, the cantilever stresses on the downstream face were significantly reduced to the opening of the horizontal joints.

Analyses performed with and without the inclusion of the thrust block indicated that the thrust block had little effect on the acceleration histories of the structure and the magnitude of joint openings.

The reservoir was about 233 feet (71.0 m) above the base of the dam during the Northridge earthquake. The effect of hydrodynamic interaction was evaluated by performing additional analyses representing higher reservoir water surfaces at (1) the spillway crest (300 feet (91.4 m) above the base) and (2) a flood level with 5 feet (1.5 m) of freeboard (360 feet (109.7 m) above the base). The computed crest accelerations and stresses were similar for cases with the reservoir at the spillway crest and at the level of the Northridge earthquake. However, with the reservoir representing a flood condition near the crest of the dam, significantly more joint opening occurs, and the cantilever stresses on the downstream face near the center of the dam increase to about 600 lb/in^2 (compared to 200 lb/in^2 (1.30 MPa) with the reservoir at the spillway crest).

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Figure PC-1. Aerial photo of Pacoima Dam (survey monuments shown)

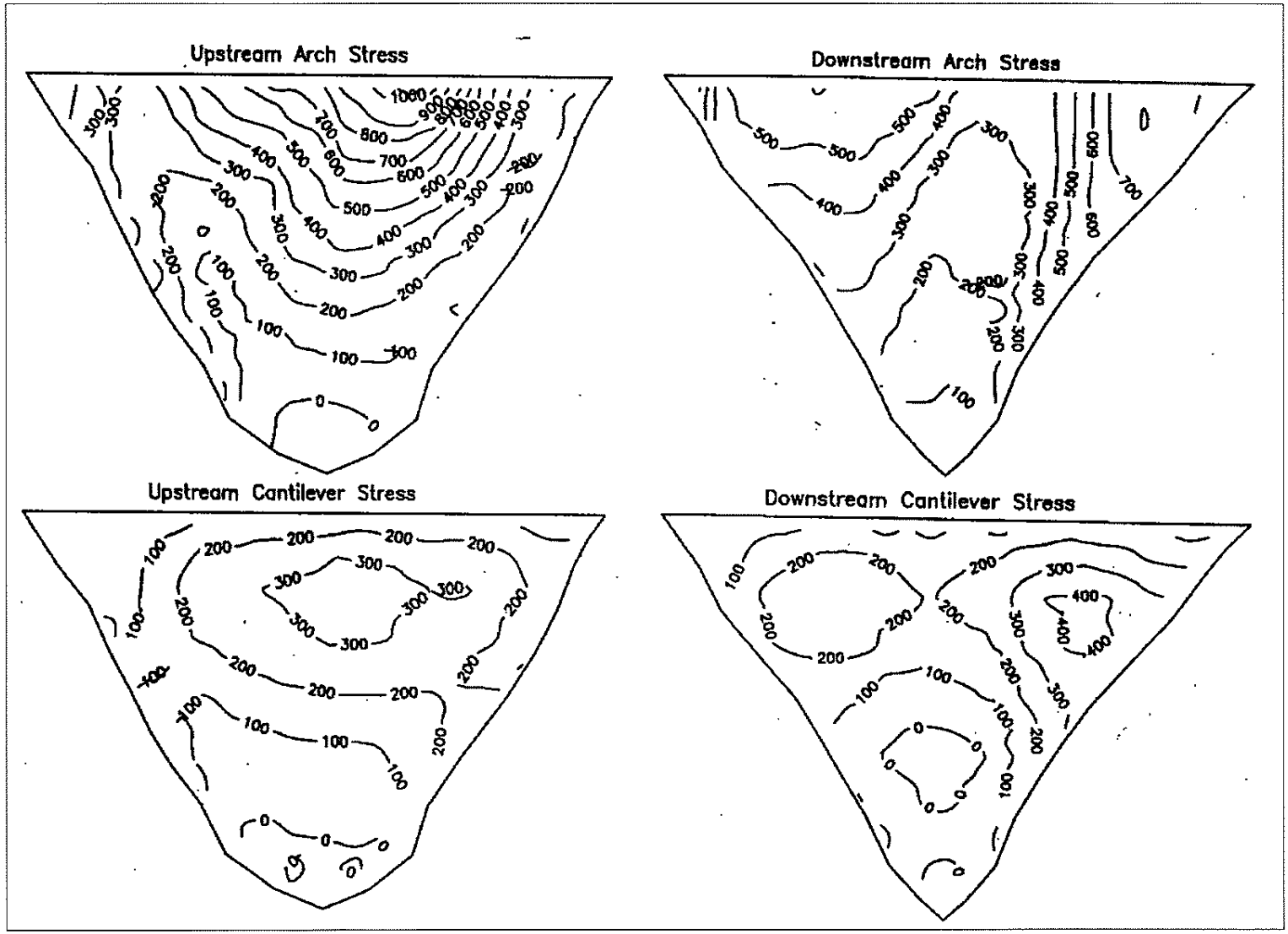

Figure PC-2. Envelopes of maximum tensile stresses for uniform motions and no joints

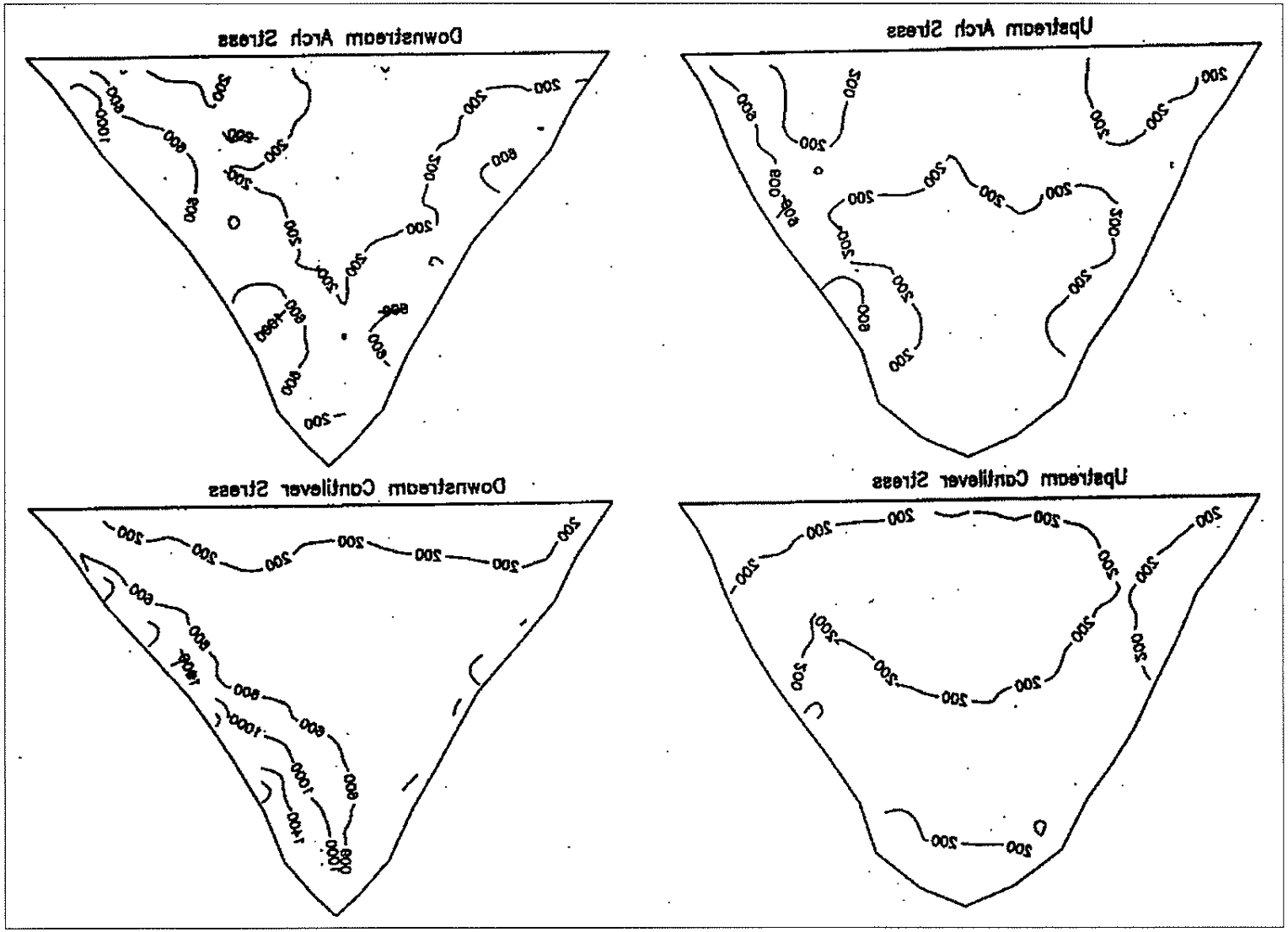

Figure PC-3. Envelopes of maximum tensile stresses with uniform motions and open contraction joints

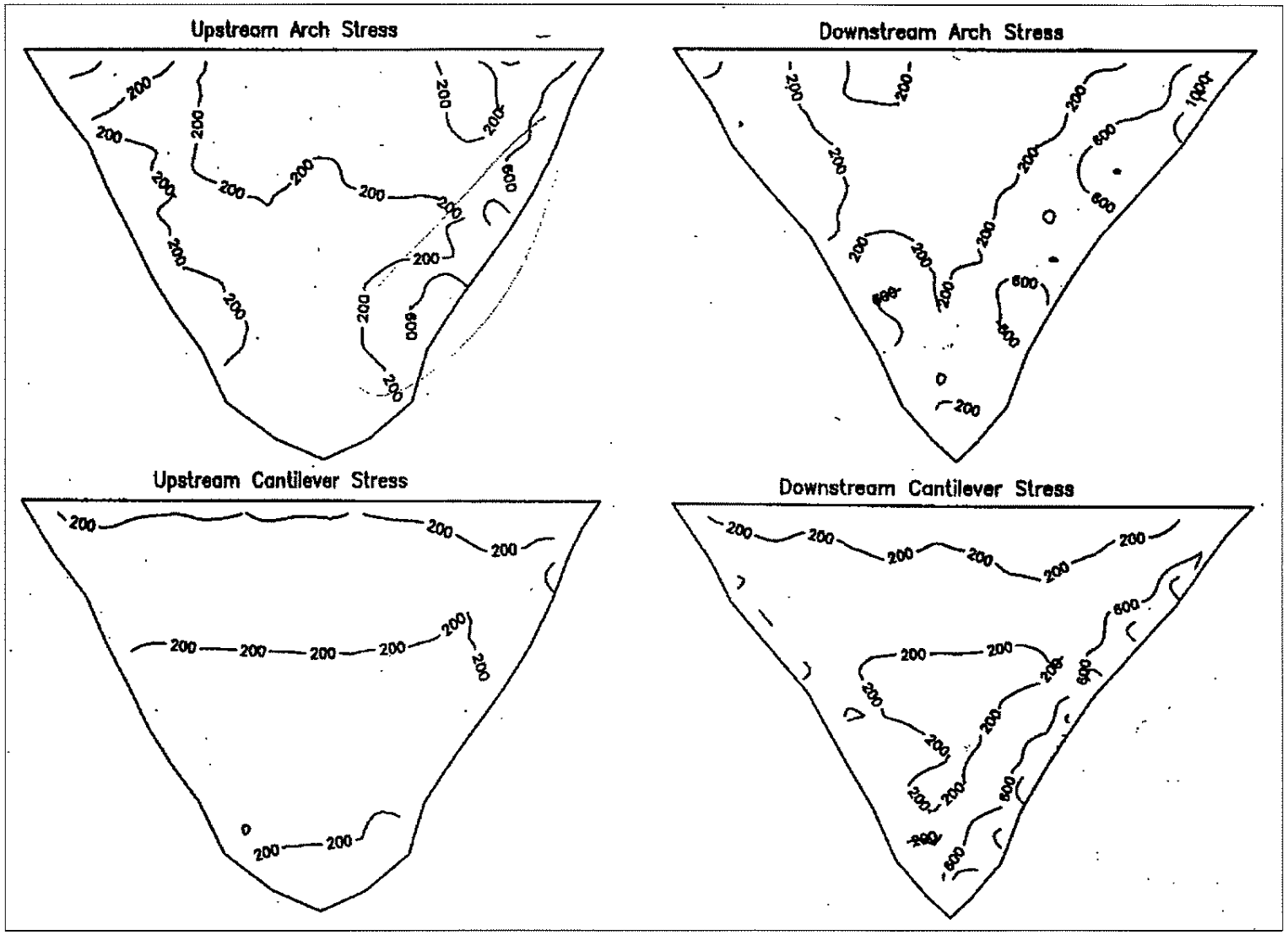

Figure CP-4. Envelope of maximum tensile stresses with non-uniform motions and open vertical and horizontal joints

4.0 Arch Dam Failures

Arch dams seem to be inherently stable structures provided the abutments are stable. The known arch dam failures have been due to foundation/abutment deficiencies and occurred upon first filling. The most notable are Malpasset Dam, France, and St. Francis Dam, California. These are summarized in this section.

CASE HISTORY SUMMARY

Name of site or structure: Malpasset Dam (Thin Arch)
Location: Southern France - Approximately 32 km (20 miles) north of St. Tropez on the Riviera
Type of event: Left abutment failure leading to sudden complete destruction of dam.
Date of event: December 2, 1959
Date of construction (if applicable): Construction completed in the spring of 1954

Loading: The dam failed with a full reservoir after an extended period of heavy rain over the entire watershed.

Description of site, structure and materials: Malpasset Dam was a 66-m (216-foot) high thin arch dam situated on the Reyran River 10 km (6 miles) upstream of Frejus in the Cannes District, France. The arch was 1.5 m (5 feet) thick at the crest and 6.7 m (22 feet) thick at the base (see figure MF-1 and MF-2). The right abutment consisted of massive rock, but the left abutment depended on a thrust block protected by a wing wall due to the topography of the site. An overflow spillway, 30 m (98 feet) long and capable of discharging 178 m^3/s (6,300 ft^3/s) was located in the central portion of the crest. At the time of completion, the dam was reported to be the thinnest of that height. The reservoir had a capacity of 51.4×10^6 m^3 (41,700 acre feet) at maximum water surface. The foundation contact was grouted with blanket holes to a depth of about 5 m (16 ft). A grout curtain was not considered necessary due to the low permeability of the rock below the 5 m (16 ft) depth. No drainage had been provided in the dam or foundation, and no instrumentation, other than surface measurement points, was installed.

The foundation was described as a "synclinal carboniferous zone enclosed by the metamorphic horizons of the base Massif of the Esterel". Exposed at the surface and in the investigation tunnel were numerous stratified mica schist layers which could be crumbled with bare hands. A fault zone was present in the left abutment, unidentified until after the failure. See figure MF-3 for geologic details.

Behavior under loading: Heavy rainfall occurred during the fall of 1954 and by mid-November the reservoir was within 5.2 m (17 feet) of the normal maximum level. At that time operators discovered seepage (a trickle of clear water) at the right abutment about 20 m (65 feet) downstream of the dam at elevation 80 m (262 ft), in an area that was not disturbed by the failure. A few weeks before, cracks had been seen the concrete apron at the toe of the dam, but no one knew when they first appeared. Beginning on November 28, there was another intense rainstorm and by November 30, the reservoir had risen another 1.8 m (6 feet) and the seepage had increased. By noon on December 2, the reservoir was at normal maximum and at 6:00 p.m., the outlet valve was opened to lower the reservoir. Engineers visiting the site at the time did not notice anything abnormal in the dam. At 8:45 p.m., the caretaker left the dam without observing anything unusual, and went to his home on a hillside about 1 mile downstream from the dam. At approximately 9:10 p.m., he heard a loud cracking noise and at about the same time the windows and doors were blown open by a violent blast of air. The caretaker also observed a bright flash

which was probably caused by the break in the power line that occurred at 9:13 p.m.. Although there were no witnesses to the actual collapse of the dam, the loud sound and rush of air indicate that the failure was sudden and complete. About a mile downstream, people reported feeling a trembling of the ground, followed by a loud brief rumble, and then a strong blast of air. Finally, the water arrived in two pulses, a wave that overflowed the stream banks and then a wall of water which the survivors barely escaped.

Analysis of the displacements of the dam remains show that the thrust block at the left abutment was first dislodged, removing support at the left abutment. The entire dam and several feet of the underlying foundation then lifted and rotated downstream as a monolithic, cantilevered structure about a vertical axis where the crest met the right abutment. The displacement at the left abutment, at the maximum distance from the axis of rotation, was 2 m (6.7 feet). The rotation occurred in two distinct movements. Both the dam and foundation rotated as a unit for a maximum displacement of 1.2 m (3.9 feet). The thrust of the rotation forces caused the structure to tilt forward, which drove the foundation material at the downstream toe of the structure into the ground. With these forces, the foundation eventually disbonded from the concrete structure. The dam alone then slid and additional 0.8 m (2.6 feet) about the cantilevered end. This slippage was determined from the displacement of an anchor bar, which remained embedded in both a standing remnant of the dam and the rock foundation. Throughout the rotations, the construction joints maintained their bond. The water released during these movements corresponds to the first pulse of water that flooded the riverbed. Shortly after the rotation, portions of the structure sequentially collapsed, beginning at the top of the left abutment, and released what was described as the second pulse of water that resulted in the devastation.

A photograph (figure MF-4) shows the remains of the base of the dam sheared off horizontally and with a wedge of rock, comprising most of the left abutment, completely gouged out along upstream and downstream dipping shear planes. The left abutment thrust block was pushed back into the hillside and downstream a distance of over 1.8 m (6 feet). The block which was originally located immediately next to the anchor block is shown lying where it had tumbled from the crest down the abutment to a point nearly half-way to the river. Companion photos (figures MF-5 and MF-6) shows a remnant of the dam still attached to the stronger right abutment rock mass but broken in stair-step fashion, in approximate 9 m (30 foot) increments, with alternating sloping shear failures and horizontal breaks at construction joints.

Consequences: The sudden failure let loose a flood wave that caused total destruction along a 11 km (7 mile) course to the Mediterranean Sea. The police at Frejus were reported to have received a telephone warning of the approaching flood but they did not have a means of passing on the warning in time. The number of deaths attributed to the failure was 421.

Back Calculations: Post-failure review uncovered no problems with the arch structure. The design and construction of the dam were well-executed and in accordance with current standards. Material tests and trial load analysis indicated the dam structure could easily withstand loads associated with the reservoir filling. Further investigation, however, determined that the failure

resulted from particular attributes of the foundation material and the geologic structure (discontinuities) that were not recognized.

Several conventional back calculations were performed following the failure. Concrete stresses were determined using a wide range in the ratio of $E_{concrete}/E_{rock}$ up to about 10 (based on concrete core tests and in situ foundation jacking tests). The predicted stresses were well within strength parameters, and did not explain the failure. The mechanism of arch buckling was also re-analyzed after the failure. These analyses indicated there was ample margin of safety for this mode. Buckling is also not consistent with the large downstream movement of the thrust block, or measured deflections of the structure under filling conditions. Sliding of the left abutment thrust block was also evaluated. Analyses showed that the friction coefficient required at the base of the thrust block to resist the thrust from the arch at the time of failure was about 0.58, if no cohesion is assumed. This is certainly not a high number for the given rock conditions, and thus sliding of the thrust block probably did not initiate the failure. Sliding at the contact between the arch and foundation was also evaluated. Even when it was assumed there is no cantilever resistance at the central base of the dam, arch action took over and resulted in a reasonable stress distribution. This explanation was therefore also discarded.

Preconstruction investigations indicated insignificant permeability through the foundation material. Accordingly, the foundation was considered stable and further investigations were limited. However, the dam site is located on Tanneron Massif which varies in strength and stability from the right to left abutment. The right abutment consists of massive gneiss. The left abutment and thrust block were seated on a schistose zone with numerous shear planes and faults parallel to the schistocity. Directly below the left bank thrust block, a large fault plane dipping approximately 30 to 50 (averaging 45) degrees to the southwest (upstream) intersected with an orthogonal foliation shear dipping to the northeast. The junction created a clearly visible (after failure) rock dihedron in the foundation. The downstream face of the rock mold (fault plane) was planar, clayey, and persistent over the whole abutment. The friction angle of the fault material was measured to be 30 to 35 degrees. With no pore pressure, the resultant force was practically normal to the this fault plane. With a pore pressure distribution determined from a seepage flow net, assuming uniform permeability, the factor of safety for sliding approached 2. Thus, even if the fault had been recognized, it is likely it would have been considered of no consequence.

However, all measured movements and post-failure evidence pointed to sliding on the fault as the cause of failure. Three-dimensional rigid block limit equilibrium analyses were conducted for a dihedral wedge formed by the foliation shear, the downstream dipping fault, and a third release plane. Only frictional resistance on the planes was considered to account for the large size of the planes (scale effect), and the potential for progressive instability if cohesion were to be lost with time. Dead weight, uplift forces on each plane, and thrust from the dam were considered loads. The analyses concluded that failure was possible when the uplift on the upstream face of the wedge is equal to full reservoir head, and the uplift on the other two faces are equal to 30 to 50 percent of the full reservoir head. The sliding would then occur on the downstream fault, and the friction angle required for equilibrium is 30 degrees. So, apparently

the only way that foundation sliding failure could be explained is if full reservoir head was assumed at depth in the foundation. Therefore, research was conducted to investigate ways in which such large pore pressures could be developed. First, it was discovered that as compressive stresses in the rock increased, the permeability decreased. Since the abutment downstream was in compression this had the effect of reducing the rock permeability and thus the ability to drain downstream of the dam. Studies on stress distributions in discontinuous media indicated that compressive stresses can penetrate to significant depths in discontinuous media such as the gneiss at Malpasset. These stresses were aligned parallel to the schistocity on the left abutment and could have penetrated to great depths, perhaps decreasing the rock permeability 100 to 1000 times at the level of the fault zone. This would create a groundwater barrier that acted as an underground dam within the foundation. The thrust of the arch at the right abutment acted perpendicular to the schistocity, and thus the compressive stresses dissipated more quickly. A crack was observed in the rock at the upstream face of the dam. It was postulated that tensile stresses at the upstream heel were relieved by opening of natural joints in the rock mass, rather than cracking of the concrete. This same phenomenon was measured at many other dams, and perhaps allowed reservoir pressures to more freely enter the foundation at the upstream face of the dam. Another factor that may have contributed to the large uplift forces is the potentially lower permeability perpendicular to the foliation shear, if impermeable materials were present in this discontinuity.

Given these factors, it is believed that the failure was indeed caused by sliding on the upstream dipping fault plane. This seems to be the only explanation that is consistent with all the known facts. Large uplift forces developed on the upstream foliation shear for one of several reasons or a combination of reasons: (1) the stresses generated in the rock extended to great depths, decreasing the rock permeability and forming an under ground dam, (2) tensile and shear stresses at the upstream face opened the foliation shear and allowed water pressures to easily penetrate to great depths, or (3) clayey materials along the foliation shear created a natural dam in the foundation. As the reservoir water rose, the loading increased until the block began to move. The arch tried to redistribute the loads higher to the thrust block, but under the extreme overload, the thrust block moved about 0.8 m downstream separating from the attached wing wall. With out any place to transfer the load, the entire left side of the dam lifted and rotated downstream (all the while the hydraulic forces are increasing) until the dam ruptured.

Discussion: Professor Carl Terzaghi commented on the Malpasset failure in February, 1962 as follows:

"The left abutment of this dam appears to have failed by sliding along a continuous seam of weak material covering a large area. A conventional site exploration, including careful examination of the rock outcrops and the recovery of cores from 2-inch boreholes by a competent driller, would show - and very likely has shown - that the rock contained numerous joints, some of which are open or filled with clay."

"From these data an experienced and conservative engineer-geologist could have drawn the conclusion that the site is a potentially dangerous one, but he could not have made any positive statement concerning the location of the surface of least resistance in the rock and the magnitude of the resistance against sliding along such a surface..."

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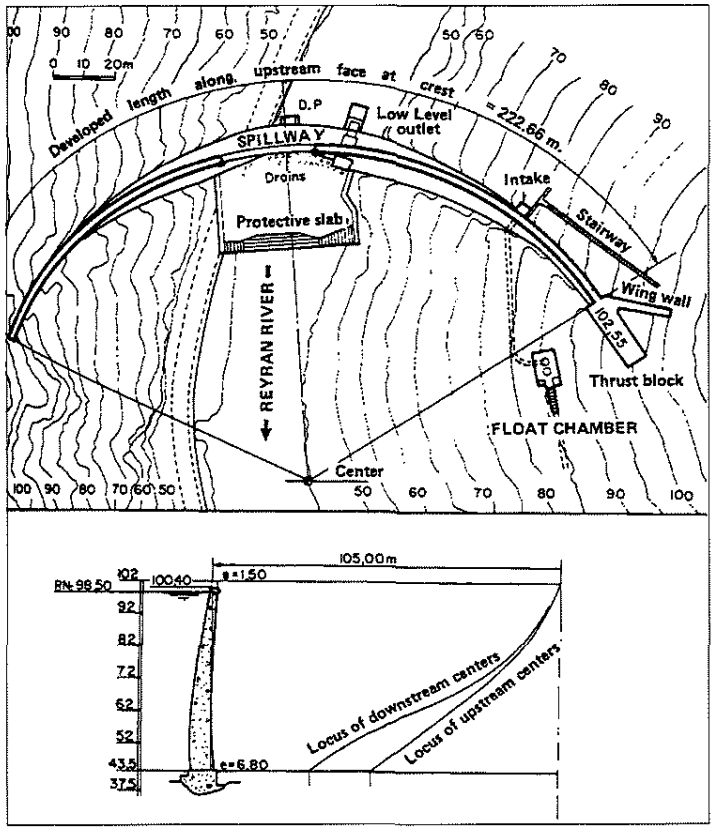

Figure MF-1. General plan and section of Malpasset Dam (after Londe, 1987)

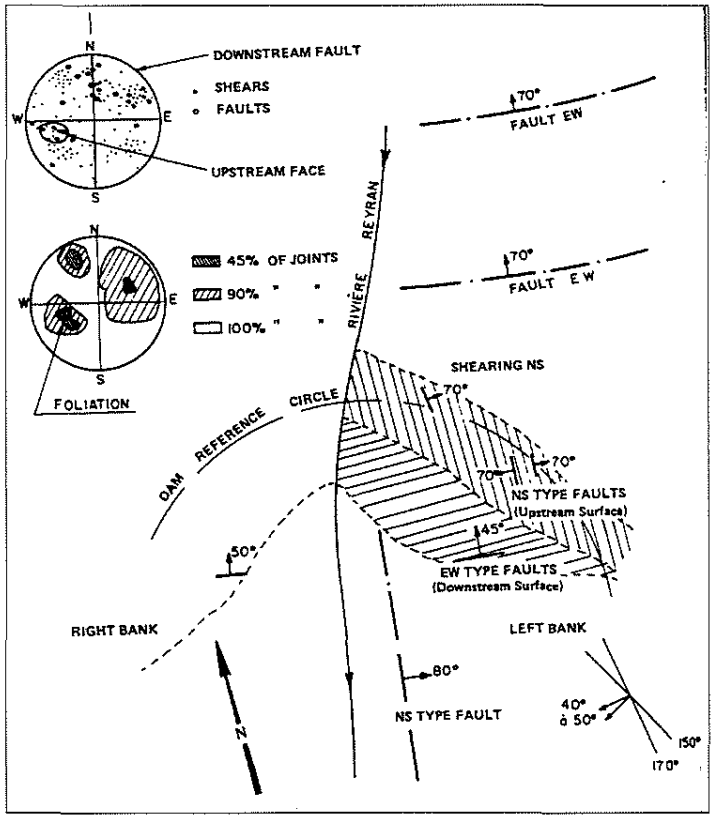

Figure MF-2. Geology of the Malpasset Damsite (after Londe, 1987)

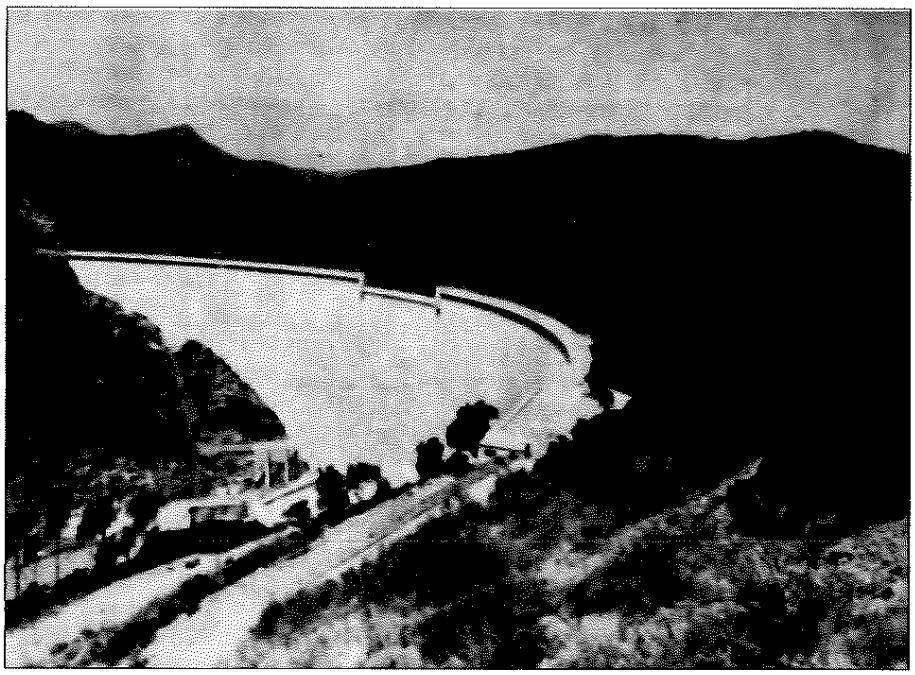

Figure MF-3. Malpasset Dam before failure (after Bellier, 1967)

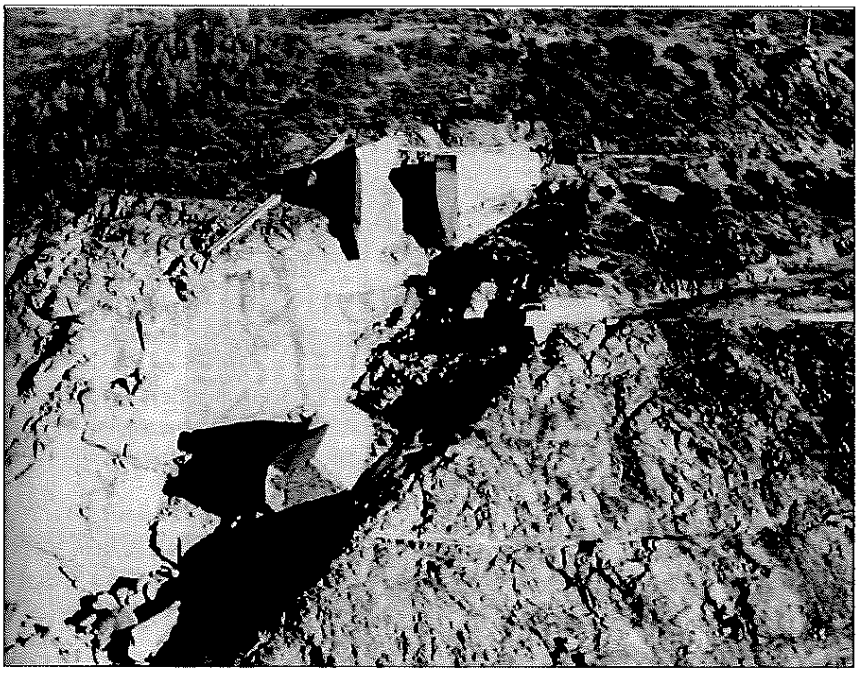

Figure MF-4. Left abutment after failure. Note upstream dipping fault and downstream dipping foliation shear (after Post and Bonazzi, 1987)

Figure MP-5. Right abutment after failure
(after Bellier, 1967)

Figure MP-6. Looking from downstream after failure
(after Bellier, 1967)

CASE HISTORY SUMMARY

Name of site or structure: St. Francis Dam (Arched Gravity Dam)
Location: Southern California, USA - In San Francisquito Canyon, approximately 45 miles north of Los Angeles
Type of event: Abutment deformation leading to sudden complete destruction of dam.
Date of event: Near midnight on March 12, 1928
Date of construction (if applicable): Construction began in April of 1924 and was completed May 4, 1926

Loading: The dam, referred to as "William Mulholland's Dam", failed suddenly under normal hydrostatic load from an initial full reservoir with no unusual weather conditions and no known seismic activity.

Description of site, structure and materials: St. Francis Dam was a 205 foot (62.4 m) high gravity arch dam situated on San Francisquito Creek, a tributary of the Santa Clara River (see figures SC-1 and SC-2). The dam was 16 feet 4.9 m) thick at the crest, 175 feet (53.3 m) thick at the base, and included a low wall extending 500 feet (152.4 m) from the main arch along a narrow ridge on the right abutment. The length of the main portion of the dam was 700 feet (213.4 m) along the curved crest. Eleven spillway openings, each 20 feet (6.1 m) wide were located in the central portion of the crest. There were five 30-inch (0.76-m) diameter outlet pipes at vertical intervals of 36 feet (11.0 m) which were controlled by slide gates on the upstream face of the dam. The dam had no contraction joints or inspection gallery. The foundation was not pressure grouted and only drained under the center section (the only section to survive). The reservoir had a capacity of 38,000 acre feet (46.9×10^6 m^3) at maximum water surface and was to be used as a backup water supply for Los Angeles in case the flow of Owens Valley water was interrupted. Mulholland, who is still famous for his success at bringing water to Los Angeles during it's early growth years, was having trouble with sabotage to his aqueduct by disgruntled Owens Valley farmers.

The foundation was comprised of two kinds of rock. The canyon floor and the left abutment was a relatively uniform mica schist known locally as "greywacke" shale. The schistosity planes were essentially parallel to the canyon wall, dipping toward the canyon at about 35 degrees. The upper half of the right side of the foundation was a red conglomerate. The contact between the two formations was a fault which, at the dam site, had a strike approximately parallel with the stream and dipped into the right abutment at about 35 degrees. The dam was placed across the fault with full knowledge of its existence.

Behavior under loading: During the initial filling in 1226-27, two sets of cracks appeared on the face of the dam, but Mulholland dismissed them as a natural result of concrete curing. Large tension cracks were noted in the schist on the left abutment on Saturday, March 10. On the day of the failure the reservoir had stood three inches from the spillway for five days, and a brisk wind created waves that splashed over the crest and soaked the downstream face. That morning

the damtender called Mulholland to report muddy water leaking from the west (right) abutment. Mulholland and his assistant hurried to the dam, but discovered the seepage was clear, picking up sediment only as it ran down the abutment. They spotted another leak on the left abutment, which they also inspected before pronouncing the dam safe. A gauge attached to the only still-standing monolith section (located between two of the early developed vertical cracks, see figure SC-3) recorded a sharp 3.6-inch (91mm) drop in the reservoir level a few hours before the collapse. One of the caretakers reportedly was seen on the crest of the dam at 11:00 p.m., just one hour before the collapse, possibly alerted to some high velocity orifice leakage situation. That caretaker disappeared in the flood and no witnesses to the dam's failure survived, although several people drove upstream past the dam just minutes before the failure, and did not notice that anything significant was occurring. One person interviewed told of having crossed a landslide scarp approximately 12 inches high cutting across the road just upstream of the dam. This would seem to correspond to an upstream lateral landslide scarp identified after the failure.

At 11:47 p.m., the operator of a power plant above the reservoir logged a call to the power plant below the dam and nothing unusual was reported by the staff on duty. At 11:57 p.m. the lights of Los Angeles flickered momentarily and at 11:58 p.m. the Southern California Edison power transmission line in the canyon downstream of the dam was broken. On a hill above the lower power plant, the home of one of the workmen was shaken. The stunned residents waited for a moment as the windows rattled. Then rumbling became more ominous, the entire house began to vibrate strongly, and then...the lights went out. Down the canyon, another employee of the powerplant was awakened by a thunderous sound. He hurried outside just in time to see a tremendous flood wave approaching. A roof from a demolished building washed toward him and he jumped up onto it for a short and turbulent ride. He was able to jump from the roof to the canyon slope and climb to safety. He spent the remainder of the night searching unsuccessfully for his family. By morning he had found a woman with her son, the only other survivors from the power plant settlement.

The dam had failed very suddenly and catastrophically a few minutes before midnight (see figures SC-3 and SC-4). Within 70 minutes the entire 38,000 acre foot (46.9×10^6 m^3) reservoir was emptied. The flood wave reached an estimated maximum height of 125 feet (38.1 m)in the first mile

Consequences: An immense wall of water was released which totally destroyed the concrete and steel "Powerhouse No.2" and its associated community just downstream of the dam. As the water swept down the canyon, it wiped out the Frank LeBrun Ranch, the Harry Carey Ranch and Trading Post, and killed six members of the Ruiz family. The Ruiz family had farmed the area since the mid 1800's. At a Southern California Edison construction camp, 16 miles (26 km) below the dam, more than 80 of the 140 people at the site died. Broad areas of the valley, where ranches and livestock had flourished, were buried in mud, rubble, and debris with extensive damage to roads, bridges, and railroads.

The flood waters met the Santa Clara River at Castaic Junction and headed west toward the Pacific Ocean. The communities of Piru, Fillmore, Santa Paula, Saticoy, and much of Ventura were devastated before the water, mud, and debris completed the 54 mile (87 km) trip to the ocean at 5:25 a.m. on March 13th. It has been estimated that 470 lives were lost but the exact count will never be known. Skeletons, buried under several feet of earth, were still being found throughout the 50's. Most recently, some remains, believed to be those of a dam disaster victim, were found in 1994.

Back Analysis: There were many investigations into to the cause of the failure shortly after the disaster. Most agreed that the failure was caused by an inadequate foundation, and cited the lack of defensive measures. Experts noted the tendency for the red conglomerate to swell, soften, and slake upon wetting, and many homed in on this as a potential cause of failure. A commission appointed by the Governor and a panel appointed by the Los Angeles district attorney both concluded the dam had failed due to water percolation and erosion near the fault zone, followed by flow toward the left abutment causing erosion and landslides on that side. Another panel hired by the Santa Clara Valley ranchers concluded that failure initiated by sliding of an ancient landslide upon which the left abutment was built.

J. David Rogers recently performed forensic investigations using modern techniques, by analyzing the abutment conditions and reconstructing the large failed concrete blocks based on their location after the flood subsided. The left abutment of the dam was indeed unknowingly constructed on a large but old paleo-landslide. Keyblock analyses were performed for the left abutment to identify potentially unstable rock blocks formed within the dam foundation by the shcistocity landslide planes, joints, and excavated abutment surface. The loads acting on the blocks were estimated and their stability analyzed. If two-thirds of the full reservoir hydraulic pressure were to have developed beneath the postulated rock blocks, the dam abutment would have been lifted, throwing parts of the sloping abutment into tension. Even with 50 percent uplift, the blocks would be predicted to be unstable (no shear strength estimates were provided). As the dam filled and uplift pressures developed in the abutment, the slide probably began to move (as evidenced by the initial cracks and the scarp observed prior to the failure). As the reservoir pool rose, the abutment would become increasingly unstable, and the movements would have loaded the dam obliquely creating tension on the upstream face. Arch action was important in redistributing stresses when the reservoir reached 7 feet (2.1 m) below the spillway. If arch action were lost at this point, stress analyses indicate excessive heel tensile stresses and toe compressive stresses would be generated. This could have resulted in lifting and tilting of the dam downstream (accounting for the sudden change in the reservoir gauge).

Several pieces of evidence suggest the left abutment failed first. The blocks found furthest downstream came from the lower left abutment area. These were probably carried by an initial flood wave laden with landslide debris, described as "liquid mud" by the only two adult survivors of the downstream powerplant community, who noticed the sound of rushing water and a "foggy haze", and scrambled up the canyon to safety just in time to see the debris wash past. Several blocks from the left abutment moved across the canyon, shearing off the toe of the dam

at the center section as they moved. A high water mark on the reservoir 4 feet (1.2 m) above the reservoir elevation suggests that a large (landslide) wave was generated while the reservoir was still fairly high. The reservoir gauge pipe was buckled in a direction that indicated water poured out through the left side of the dam first. Surveys indicated the remaining monolith had shifted 0.52 feet (0.16 m) east (toward the left abutment) and 0.46 feet (0.14 m) downstream, indicating a clockwise rotation (in plan view). The movement toward the gap left at the left abutment probably then removed any remaining arch action on the right abutment, resulting in collapse of that section as well.

Discussion: Many lessons were learned from the St. Francis Dam failure. Sound geologic input must be obtained from more than one source. The concepts of uplift and effective stress were apparently not well understood by Mulholland and the design engineers, and later work brought this to the forefront. No outside consultants were retained to review the designs, which lead to legislation in California mandating this be performed. It is interesting to note that a similar recommendation (use of external consulting boards) was made following the failure of Teton Dam.

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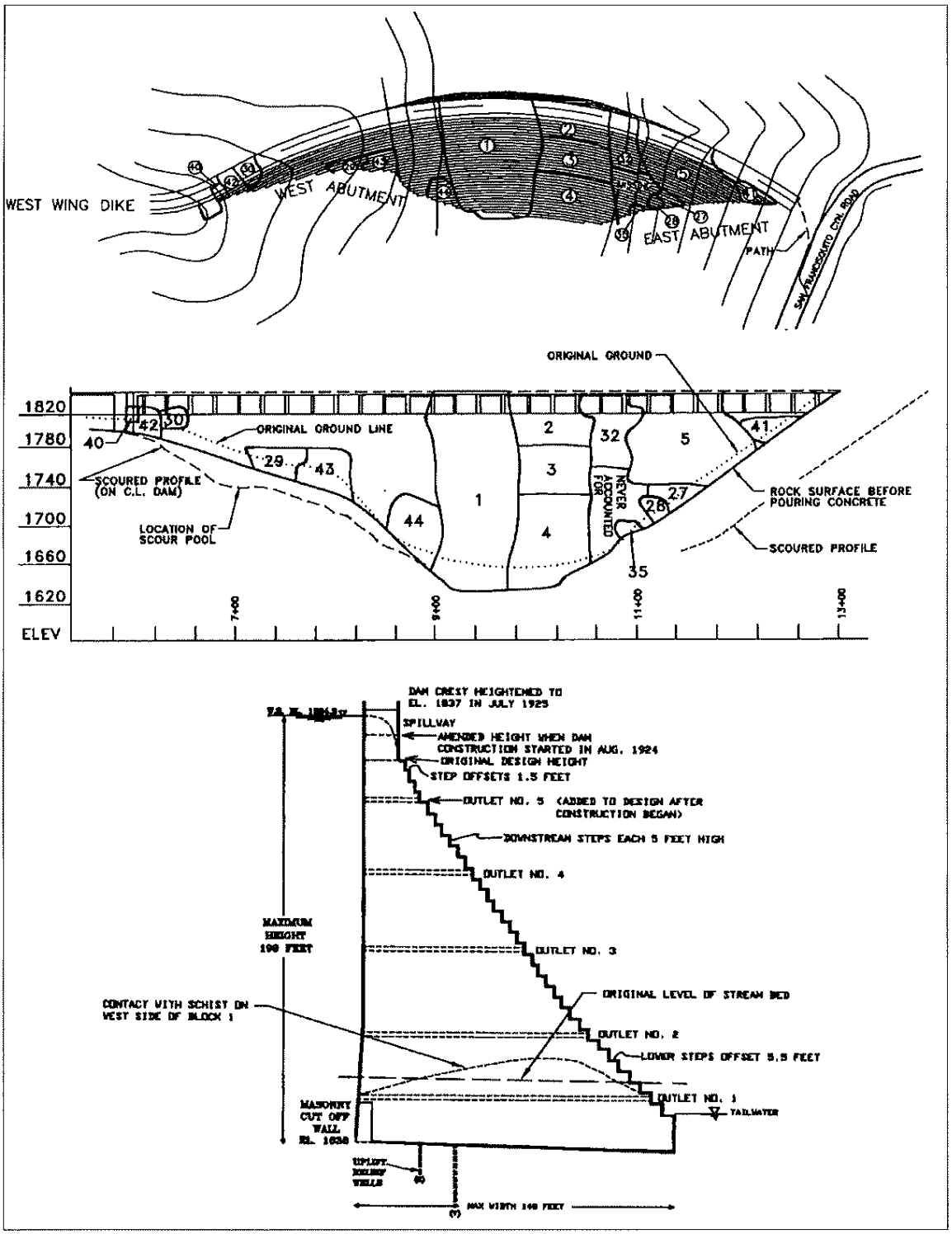

Figure SC-1. Plan, profile, and section of St. Francis Dam

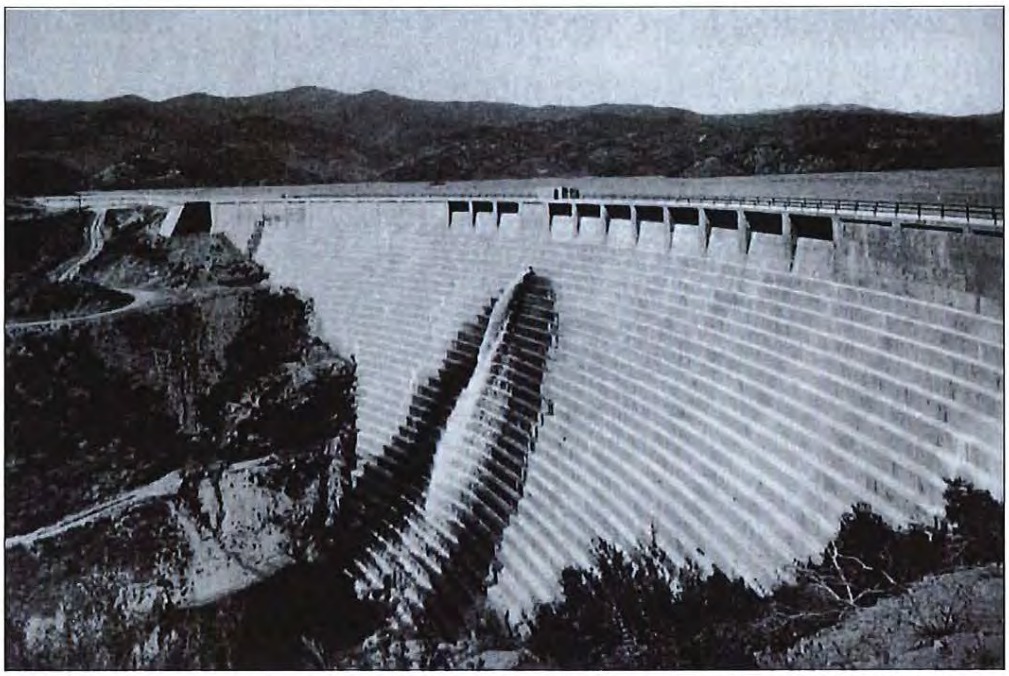

Figure SC-2. St. Francis Dam before failure (Huber Collection, U of Calif Water Res Center Archives)

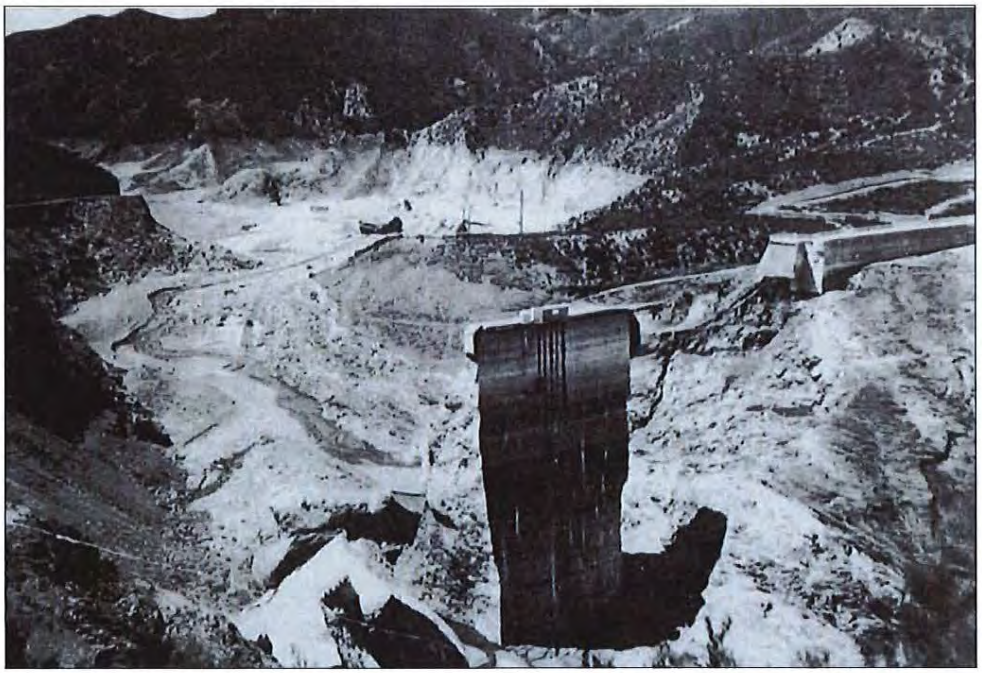

Figure SC-3. St. Francis Dam after the failure (Huber Collection, U of Cal Water Res Center Archives)

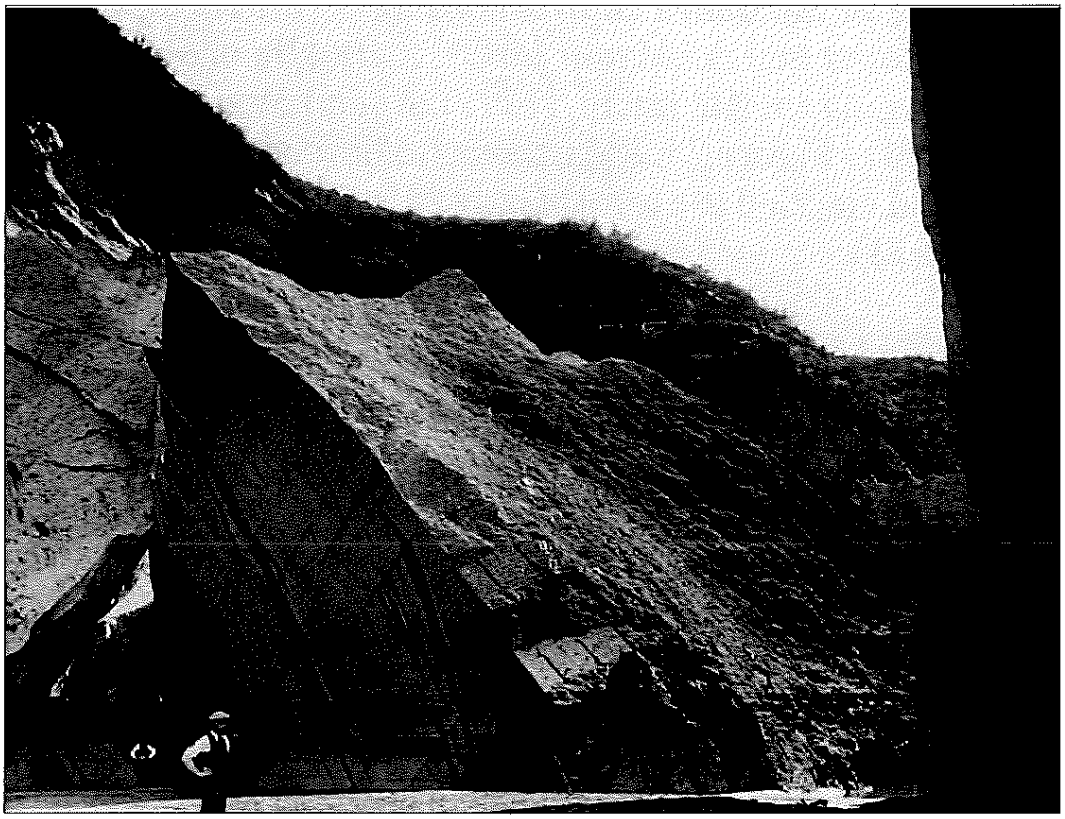

Figure SC-4. Blocks of concrete after failure, note size of men for scale (Huber Collection, U of Cal Water Res Center Archives)

5.0 Landslides

Landslides in the reservoirs of either concrete or embankment dams that are large and capable of moving at great speeds can generate dangerous waves. The grand-daddy of these was the Vaiont slide in Italy. This case is presented in this section. While this was an unusual situation, understanding of the geometry, geology, and other conditions associated with the slide is useful in evaluating other potential slides.

CASE HISTORY SUMMARY

Name of site or structure: Vaiont Dam (Very high, narrow, and thin arch)
Location: Northern Italy - Near Longarone and Belluno in Veneto Province
Type of event: Massive reservoir landslide causing overtopping
Date of event: October 9, 1963
Date of construction (if applicable): Construction completed in the fall of 1960

Loading: The dam was overtopped by a wave estimated to be 100 m (330 feet) above the crest of the dam caused by a huge nighttime rockslide which hit the reservoir close to the dam with tremendous speed and force. The dam remained intact, except for the destruction of the bridge over the crest spillway and other minor damage, even though estimates of the load sustained were as high as 4 million tons from the slide and overtopping pressures.

Description of site, structure and materials: Vaiont Dam is a 265 m (869 foot) high thin arch dam situated on the Vaiont River near Longarone in Veneto Province, Italy (see figure VI-1). The arch is 3.4 m (11.2 feet) thick at the top and 22.7 m (74.5 feet) thick at the base. An unusual feature of the dam is its division into 4 sections by horizontal joints. To account for the extreme height and narrow width, three horizontal joints were coated with a material to prevent bond and force the loads to be carried by pure arch action in the upper sections. A two lane concrete bridge was supported on piers across the length of the crest. The crest length is 189 m (620 feet) and includes 16 sluices, each 2.7 m (22 feet) wide, in the center of the crest. An underground power plant and three additional outlets are located in the left abutment. Prior to the slide, the reservoir had a capacity of 150×10^6 m^3 (122,000 acre feet).

The foundation consists mainly of oolitic limestone and Liasic sliliciferous limestone with thin layers of organic silica. Excavation for the dam disclosed many fractures paralleling the river in the upper portions of the foundation. The steep gorge walls had to be reinforced by 4570 m (15,000 feet) of post-tensioned rods or grouted wire anchorage and 30,000 m^3 (36,000 square yards) of wire netting.

Behavior under loading: Survey reference points were installed on the left slopes of the Vaiont Valley in May of 1960, and the dam was completed in September of that year. By October 1960, the reservoir had filled to elevation 635 m (2083.3 ft), and about the same time approximately 500 mm (19.7 in) of rain fell. The benchmark surveys indicated accelerating movements. An average of 1 m of movement occurred, and a crack over 2 km (1.24 mi) long formed in the upper slopes above the reservoir.

On November 4, 1960, when the reservoir water had risen to a depth of 129.8 m (426 feet) (elevation 645 m (2116.1 ft)) during first filling, a slide carrying 700,000 m^3 (916,000 cubic yards) came down the left slope of the canyon very near the dam. There is no documentation of damage or loss of life caused by the 1960 slide, but cracking was observed above the slide scarp indicating that a larger mass was still in motion. A 2-m (6.6 ft) high wave was generated by the

slide. After the 1960 slide, the lake was lowered and restricted to the 600 m (1968.5 ft) level, and reservoir bank stability studies were expanded. Drainage adits were driven at about elevation 920 to 950 m (3018.4 to 3116.8 ft), an expanded network of survey monuments was established on the slope extending 4 km (2.5 miles) upstream, a bypass tunnel was constructed on the right bank in case a slide were to divide the reservoir, and piezometers were installed in the slide. The reservoir was raised again to elevation 650 m (2132.5 ft) in early 1962, with generally small movements. The reservoir continued to rise throughout 1962 until it reached elevation 700 m (2296.6 ft) in November 1962, a month when 414 mm (16.3 in) of rain fell, and the rate of movement increased to 12 mm (0.47 in) per day. The reservoir was again lowered to elevation 650 m (2132.5 ft) by March 1963. Over the next year, movement tended to decrease and the reservoir was allowed to fill within 24 m (80 feet) of the crest (elevation 710 m (2329.4 ft)). Intense rain during August and September of 1963 caused heavy runoff and recharged the water storage in the rock mass. This increased the weight and the internal hydrostatic pressures on the planes of weakness. By mid-September 1963, many of the geodetic monuments on the lower parts of the slope were creeping at a rate of about 35 mm (1.4 in) per day, and by the end of the month similar rates were observed on the upper parts of the western portion of the slide.

Heavy rainfall resumed on September 28 and continued on into October. The reservoir rose to within 26 m (85 feet) of the crest and animals grazing on the slopes of Mount Toc abandoned the area, presumably sensing the hazard. A slow drawdown was begun the beginning of October. On October 8, those responsible for monitoring the geodetic grid recognized that a tremendous mass was in motion, embracing an area five times as large as they had assumed was affected. Efforts to quickly lower the reservoir were essentially nullified by the heavy inflows. At 10:39 p.m. on October 9, a gigantic slide 2 kilometers (1.2 mi) across and moving at speeds estimated at over 30 m (100 feet) per second plunged into the reservoir. The duration of the entire disaster, from the initiation of the slide to the complete downstream destruction, was estimated at seven minutes.

The volume of slide material has been estimated at 270×10^6 m^3 (350 million cubic yards). The material covered an area 1.8 km (1.1 miles) wide by 1.6 km (1.0 mile) high along the left abutment, just upstream from the dam (see figures VI-2 through VI-4). The elevation of the slide area surface ranged from approximately 550 m (1800 feet) at the toe to 1200 m (3937 feet) at the top. The reservoir level was at elevation 700.4 m (2330 feet) when the slide occurred.

Consequences: A resident of Casso, on the right canyon wall 260.0 m (853 feet) above the lake, reported that at about 10:15 p.m., he was awakened by the sound of moving rocks. He was not alarmed since surficial slides had become a common occurrence. Then at 10:40 p.m., an air blast hit his two story building, breaking the windows. The roof then lifted enough for rocks and water to spray into his bedroom. He had scrambled to the door just as the roof collapsed onto his bed and the wind abruptly subsided.

Others at the site included 20 technical personnel in the control building on the left abutment and about 40 occupants of an office and hotel building 55 m (180 feet) above the crest on the right

abutment. None of these people, who may have actually seen the mountain collapse, were counted among the survivors. After demolishing the hotel high on the right bank, the displaced water apparently surged back across to the left abutment and rose there to a height of 100 m (328 feet) above the dam crest. Giant waves converged at the dam and went over the crest in a massive spill. The flood wave was more than 230 feet high where the Vaiont River enters the Piave River about 1 mile downstream of the dam. The sudden huge flood wave erased any sign of most of the many substantial masonry buildings in the village of Longarone and caused the death of more than 2600 people. Surviving witnesses living higher on the valley slope said the flood wave hit at 10:43 p.m. causing strong earth tremors and an air blast that blew out windows. By 10:55 p.m., the flood had passed and the valley was again silent.

The slide had totally filled the reservoir for a distance of over a mile upstream from the dam. In places the material lay at heights of 150 m (492 feet) above the lake level. Even in the underground power plant, steel doors had been ripped from their hinges and structural steel beams and columns were twisted or sheared. The flood also caused devastation for many miles on down the Piave valley. Figures VI-5 and VI-6 show the town of Longarone before and after the disaster, respectively.

Back Analysis: The instability of the slopes of Mount Toc on the south side of the reservoir was a source of some controversy during design and construction. The geologists postulated that the left reservoir bank represented an old prehistoric landslide that had moved down Mt. Toc in a northeast direction. Since small slides were regularly experienced on the left side of the basin prior to construction of the dam, a seismic survey was initiated in 1959 to assess whether this rock mass was naturally in place or rather the remains of a previous slide. The rock layers were found to have high seismic velocities and thus it was proposed that the foundation was in place and stable. Exploratory borings conducted in 1960 encountered fractured rock through which water frequently circulated and disappeared, indicating underlying fissures and open material. A variety of soils were discovered in the form of intermittent layers of hard rock and uncemented clay-like materials. During reservoir filling it became apparent that the reservoir bank was sliding. Post-failure studies indicate that the left bank was indeed composed of a rock mass that had slid to its current location in the post-glacial period, explaining the discontinuities and voids in the soil structure. The base of the 1963 slide planes was found to correspond to a prehistoric slide surface.

Analysis of the data indicated a direct correlation between precipitation, reservoir level, and rate of movement at the slide location from 1960 to 1963. All major slide movements were preceded by periods of heavy rain and reservoir filling, as can be seen in figure VI-7. The excess pore pressures created by the reservoir and rain was enough to trigger sliding, and eventually the catastrophic failure.

Although the massive slopes that failed were predominantly limestone and dolomite, they contained prominent and continuous clay interbeds and clay layers (gouge from past movements). It is clear that multiple layers of weak clays were present along much of the sliding

surface. Testing of these layers that occurred shortly after the slide indicated very low strengths (residual friction angles between 5 and 22 degrees), but back-calculated angles of shearing resistance were much higher (17 to 39 degrees), even assuming water pressures that appear to be too low (i.e. even higher friction angles would be needed to account for calculated factors of safety if appropriate water pressures were used). There were apparently significant geometric effects that were not accounted for by the initial simple two-dimensional analyses. More detailed analyses were conducted by Patton and Hendron (1985). The base of the slide was assumed to correspond to a pre-historic slide surface. The pronounced upstream (eastward) dip of the failure surface along the base of the seat of the slide, and shearing resistance developed along the east side of the slide was considered in the analysis (see figure VI-8).

Laboratory analyses of the physical properties of the soil along the slide plane determined that the clay content is about 50 to 80 percent, predominantly calcium montmorillonite. The in-situ appearance of the clay also indicated that the microscopic bonds of the clay had deteriorated under previous loads. It was soft, sticky and slightly cracked due to repeated cycles of wetting and drying. Laboratory plasticity tests indicated the presence of both inorganic clays of low plasticity (liquid limit = 33-60, plasticity index = 9-27), and clays of high plasticity (liquid limit = 57-91, plasticity index 30-61), indicating low shear strengths, no cohesion, and high swelling potential. The residual angle of shearing resistance, was on average 8-12 degrees as measured in the laboratory. However, 10-12 degrees was thought to be appropriate for analysis because (1) there were localized areas of shearing across bedding planes, (2) there were areas where clays did not occur, (3) there were areas where clays were squeezed and forced into voids that developed as a result of displacement of irregular surfaces on either side of the clay beds, and (4) small increases in shear strength could result from introduction of brecciated rock fragments into the clays along the sliding surface. Friction angles were assumed to be 30 to 40 degrees for discontinuities not parallel to bedding.

The piezometric head acting on the sliding surface was assumed to equal the reservoir level where the reservoir was in contact with the slide surface. Away from the reservoir, the piezometric pressure was assumed to be artesian below the slide plane, based on an assumed groundwater flow system, to match water levels measured in a few piezometers above the reservoir for low rainfall conditions. For high rainfall conditions, higher levels were assumed.

The main goal of the stability analyses was to try to understand the periods of movement that were recorded, and the difference between unstable behavior observed in October 1960 with the reservoir at elevation 650 m (2132.5 ft) (high rainfall) and the stable conditions in January 1962 with the reservoir at elevation 650 m (2132.5 ft) (low rainfall). Three-dimensional slope stability calculations best modeled the slide history and ultimate failure of the left reservoir slope. Using a base plane friction angle of 12 degrees, an eastern wall boundary friction angle of 36 degrees, and a friction angle of 40 degrees along vertical rock surfaces between slices used in the calculations, the factors of safety on the following page were calculated (Hendron and Patton). These calculations generally support the observed movements and failure. High reservoir and high rainfall would be expected to trigger a failure. Marginally stable conditions would be

present under high reservoir and low rainfall, or under reservoir elevation 650 m (2132.5 ft) with high rainfall. The movements occurring in October 1960 corresponded to even higher rainfall amounts than were probably modeled in the analysis. With the reservoir at elevation 650 m (2132.5 ft) and low rainfall (January 1962) the slope would be predicted to be relatively stable. It can also be seen that even without the reservoir, high rainfall conditions could bring the slope to a marginally stable condition, perhaps inducing movements before the dam was built.

Reservoir Elevation	Rainfall Condition	Factor of Safety
710 m (2329.4 ft)	High	1.00
710 m (2329.4 ft)	Low	1.10
650 m (2132.5 ft)	High	1.08
650 m (2132.5 ft)	Low	1.18
None	High	1.12
None	Low	1.21

The downstream destruction was caused by the wave which went over the top of the dam. Prediction of the height of landslide generated waves is often useful in deciding the hazard posed by landslides in the reservoir of a dam. Very few tools are available for this estimate. The Bureau of Reclamation performed some laboratory studies for Morrow Point Dam, and developed empirical relationships for that geometry. The wave height can be estimated from the following equation:

$$\frac{?}{D} \bullet 0.14 \frac{\sqrt{\frac{V}{D^3}}}{10^{\frac{L}{58D}}}$$

where ? is the wave height at the dam, D is the reservoir water depth at the landslide, V is the volume of water displaced by the landslide, and L is the distance from the landslide to the dam. Given that the depth of the reservoir near the dam was about 240 m (787 ft), the volume of displaced water was about 240 million cubic meters (314×10^6 yd^3), and the slide was as close as about 100 m (328 ft) to the dam, a wave height of about 95 m (312 ft) is predicted. This under predicts the actual wave height by about 25 percent, which was about 125 m (410 ft) since the reservoir was 25 m (82 ft) below the top of the dam when the slide occurred. If the volume of the slide is used instead of the displaced water volume, the wave height is predicted to be about 146 m (479 ft). Using a volume of 700,000 cubic meters (916,000 yd^3) to represent the November 4, 1960 slide results in an estimated wave height of about 7 m (23 ft), as opposed to

about 2 m (6.6 ft) actually observed. Therefore, it appears that this equation will produce about the right order of magnitude for other dams, but probably won't predict the actual number extremely closely. If the entire slide volume is used in the equation, it appears the results will be on the conservative side.

Discussion: The selection of this site for Vaiont Dam was apparently strongly influenced by a rare opportunity to build the world's highest arch dam. Indeed, the dam withstood unbelievable pressures from the reservoir surge and overtopping and is still in place today, which is a tribute to the dam's structural design. The very narrow and extremely deep canyon with strong rock walls was so attractive, that the designers seem to have missed indications of the dangers of the unstable reservoir banks. Evidence of a major problem was misinterpreted. The karst terrain allowed rainfall to enter freely into the landslide mass. The connection between rainfall, reservoir level, and slide movements was missed. In fact, it was observed that the movements only occurred when the reservoir was raised to new levels. This led the authorities and technicians to the conviction that a gradual stabilization of the moving mass would be brought about by raising the water level in individual small steps.

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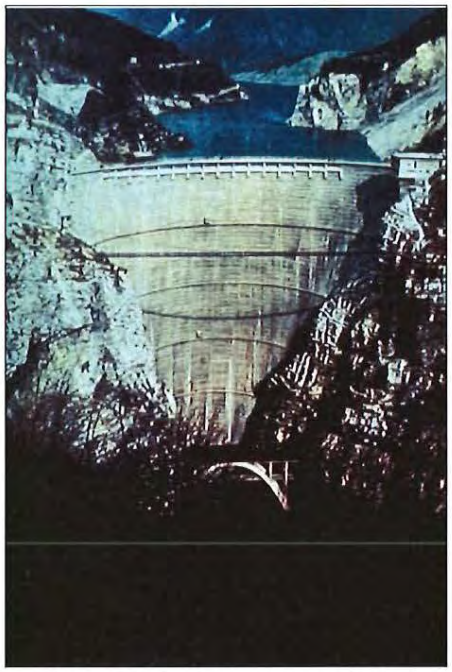

Figure VI-1. Vaiont Dam before Landslide

Figure VI-2. View of left reservoir bank following failure

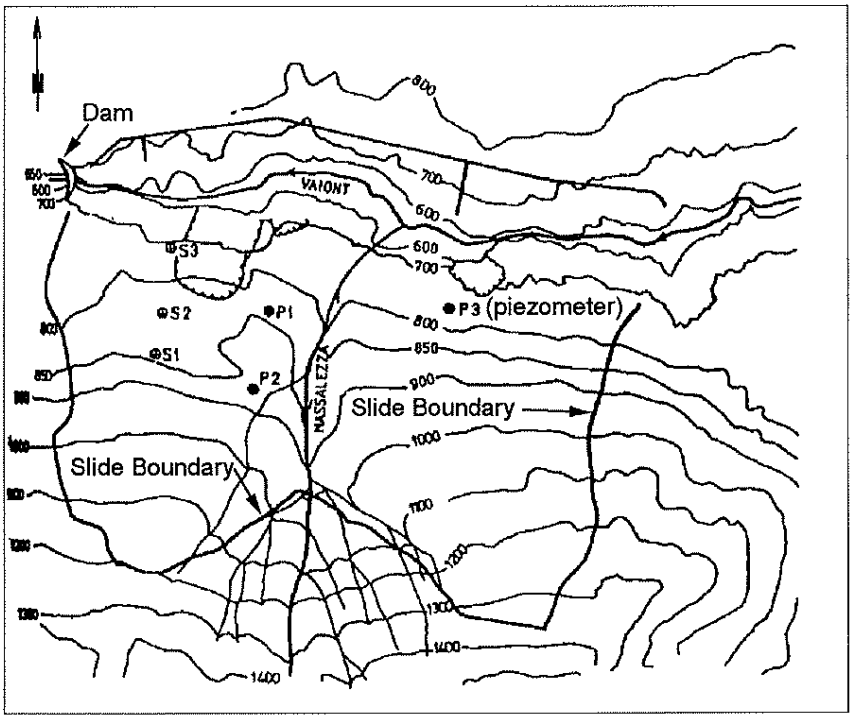

Figure VI-3. Plan view of landslide and piezometer locations (after Nonveiller, 1987)

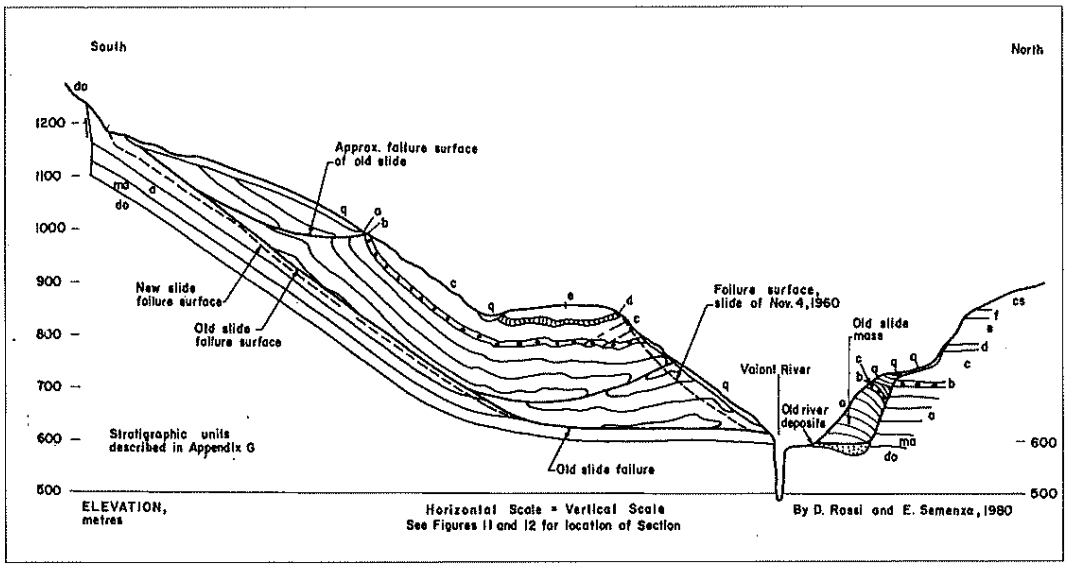

Figure VI-4. Section through slide near dam (after Hendron and Patton, 1985)

Figure VI-5. Village of Longarone before failure

Figure VI-6. Village of Longarone after the failure

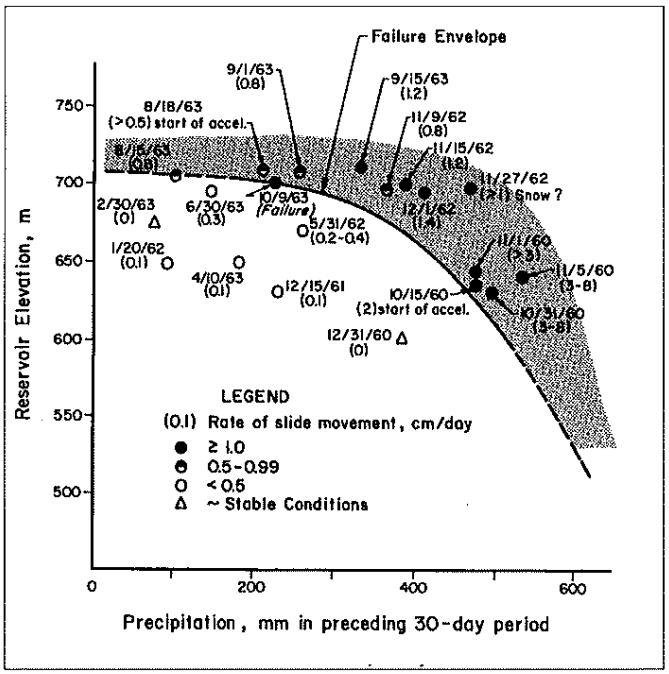

Figure VI-7. Evaluation of reservoir elevation, precipitation, and landslide movement (after Hendron and Patton, 1985)

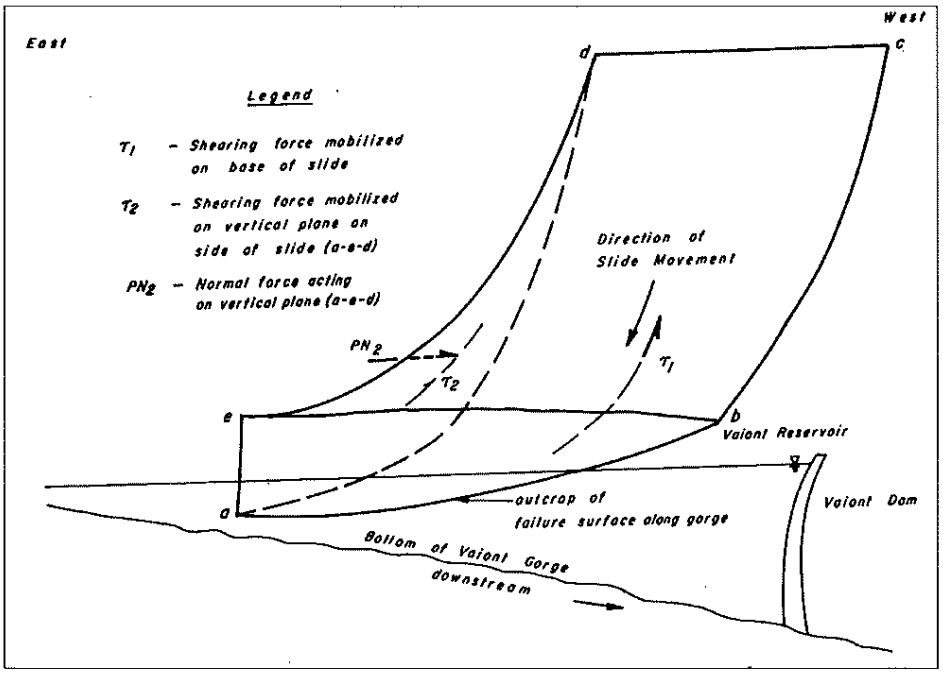

Figure VI-8. Schematic of landslide mass (after Hendron and Patton, 1985)

6.0 Overtopping During Floods

Several smaller concrete dams have failed due to overtopping during floods. The main cause being erosion of the foundation or abutment materials and loss of support. Determining whether significant erosion would occur has generally been a matter of judgement. New empirical procedures are allowing perhaps a better prediction of erosion potential. Information is not available to apply these methods to known overtopping failures. However, some larger dams have withstood overtopping. One such notable case, Gibson Dam, is presented in this section.

CASE HISTORY SUMMARY

Name of site or structure: Gibson Dam
Location: Montana, USA - 30 miles (48 km) northwest of Augusta on the North Fork of the Sun River
Type of event: Overtopping during extreme storm
Date of event: June 8, 1964
Date of construction: 1926 thru 1929 with spillway modifications in 1938 and abutment overtopping protection with crest aeration piers added in 1980.

Loading: Reservoir inflows reached "unimaginable" levels due to a combination of sustained up slope winds and unusually heavy moisture from the Gulf of Mexico. These conditions caused a rainstorm over an area 100 miles (161 km) long on the eastern slope of the continental divide and produced 30 hour rainfall amounts of from 8 to 16 inches (203 to 406 mm). The shallow soils along the Rocky Mountains and foothills area were already saturated with spring snowmelt and there was very little capacity for retaining the flows. By 2 P.M. Monday, June 8, when the overtopping began, inflows reached an estimated maximum of 60,000 ft^3/s (1700 m^3/s) and remained at this rate for 3 hours. A high water mark inside the spillway control house indicated the dam was overtopped by 3.23 feet (0.98 m). By 8 A.M. Tuesday inflows had dropped to 30,000 ft^3/s (850 m^3/s) and by 10 A.M. water stopped flowing over the parapet. The overtopping event lasted 20 hours.

Description of site, structure and materials: Gibson Dam is a 199 foot (60.6 m) high concrete thick arch dam with a crest length of 960 feet (292.6 m) (see figure GM-1) and was one of the first few dams (along with Pathfinder and Buffalo Bill Dams) in the world to be designed using the trial-load method of analysis. The dam crest width is 15 feet (4.6 m) and the maximum base width is 117 feet (35.7 m). The spillway is a drop-inlet, discharging into a shaft and 29.5 foot (9.0 m) diameter tunnel in the left abutment, controlled by six 34 by 12 foot (10.4 m by 3.7 m) radial gates. The foundation is crystalline (Madison Group) limestone in regular beds which have a strike normal to the river and an upstream dip of approximately 75 degrees. The only access to the dam is on a road along the North Fork of the Sun River, downstream of the dam.

The reservoir has a capacity of 99,100 acre-feet (122×10^6 m^3) with the water surface at El. 4724 feet (1439.9 m). The service spillway capacity at that elevation is 30,000 ft^3/s (850 m^3/s) with the outlet works providing an additional 3,050 ft^3/s (86 m^3/s). The drainage area for the reservoir is 575 square miles (1489 km^2) and in 1970 a new inflow design flood was derived which has a peak discharge of 155,000 ft^3/s (4390 m^3/s) and a five day volume of 365,000 acre-feet (450×10^6 m^3). The dam is the primary feature of the Sun River Project which supplies water for irrigation of more than 90,000 acres (364×10^6 m^2) of land downstream.

Behavior under loading: Huge volumes of water fell over the dam and washed down the entire extent of the dam abutments. As the overtopping began at 2:00 P.M., water was flowing over the

upstream parapet and was conducted to the right, along the crest roadway, to an area approximately 300 feet (91.4 m) downstream of the dam where the substantial stream of water flowed down the right side canyon wall. The water continued to rise rapidly and by 3:00 P.M. the downstream parapet was overtopped and the entire dam crest became a huge waterfall (see figure GM-2). Maximum discharge over the parapet was estimated to be 18,500 ft^3/s (524 m^3/s) with reservoir storage at an unprecedented 116,400 acre feet ($144x10^6$ m^3). Fortunately, the abutment rock and the canyon walls were quite durable and were not susceptible to the destructive force of the falling water or the erosion from the heavy flow down the canyon walls.

Consequences: Remarkably little damage was caused at Gibson Dam by the overtopping. No structural damage could be found and only three 1 ½ -inch (38 mm) diameter pipes leading to the Valve No.1 hand controls were broken and leaking water. Water had broken the entry door to the valve house and the windows were broken out. The pipe hand railing along the walkway to the valve house was about 50% destroyed. Routine clean-up and typical O&M activities restored the dam to normal operating condition. An access road bridge, a large storage building, and much of the access road downstream of the dam were destroyed by a combination of the dam outflows and the additional heavy flow entering the river from Beaver Creek just downstream of the dam.

It should be noted that operating personnel were unable to get to the dam during the event because of the loss of the access road early in the flood. On May 28th, the day before water started over the spillway crest, the operators had left the river outlet discharging 1,800 ft^3/s (51 m^3/s), spillway gates No.2 and 5 fully open, No.3 and 4 completely closed, No.1 open 9 feet, and No.6 open 11 feet (3.4 m). With this gate configuration, outflow could reach 32,200 ft^3/s (911 m^3/s) at a water surface elevation of 4729 feet (1441.4 m). This would have passed the greatest previous flow of record, the flood of June 1916, without overtopping the dam. Later analysis indicated that the dam would have been overtopped by the 1964 flood even if all gates were fully opened as early as June 1st.

The consequences downstream of the dam were much more damaging. The very large uncontrolled releases, combined with flows from other tributaries, caused heavy flooding at nearly all rural and urban areas in the entire Sun River Valley. Especially hard hit were the low lying areas from the town of Simms on down to the western suburbs of Great Falls where the Sun River enters the Missouri River. The only access to many areas, including Gibson Dam, during and immediately after the flood was by helicopter due to the many bridges and highways that were totally washed away.

Back calculations: The erodibility of the abutment rock was investigated prior to the design of the 1980 modification for overtopping protection. Although the rock strength and jointing appeared to be quite resistant to erosion during this event, a 3 to 5 foot (0.9 to 1.5 m) thick concrete overlay with anchor bolts was placed where the overtopping flow impinged on the right abutment and foundation (see figure GM-4) to protect against even larger floods up to the probable maximum flood (PMF). The steeper left abutment was treated with rock bolts and a

concrete cap in major surface fracture zones (see figure GM-3). This modification was deemed prudent given the large degree of uncertainty associated with determining erodibility.

A simplified empirical approach to determining erodibility was subsequently used to determine if it would accurately predict the observed response. This approach is based on the stream power of the impinging jet, and the erodibility index of the material being hit. Recent relationships suggests that erosion is possible if the available stream power is greater than the erodibility index raised to the 0.75 power ($P_r >= K^{0.75}$).

The erodibility index is computed as $K=(K_m)(K_b)(K_d)(K_s)$. K_m is an evaluation of the mass (intact) strength of the foundation. This varies depending on whether the foundation is a granular soil, a cohesive soil, or rock. The majority of the foundation rock at Gibson Dam is limestone and dolomite (referred to as limestone in much of the documentation). For the evaluation at Gibson, the value of K_m is the unconfined compressive strength in MPa. The average value from laboratory tests was 22,900 lb/in^2 (158 MPa). Some weaker intensely fractured beds (about 6 to 10 feet (1.8 to 3.0 m) thick) are present, particularly on the left abutment. The rock in these beds would have a lower strength, perhaps by a factor of 2 to 4 (40-80 MPa). Laboratory testing performed on the concrete during original construction of the dam resulted in an average unconfined compressive strength of about 2940 lb/in^2 (20 MPa).

K_b is an index related to the mean block size. It can be estimated as the rock quality designation (RQD) divided by the number of joint sets. The dam foundation limestone varyies from thin beds a few inches thick to massive beds, 8 to 10 feet (2.4 to 3.0 m) thick. The rocks were found to be broken by several fissures, which followed the bedding surfaces very closely. Another prominent joint set was mapped on each abutment, and there were other minor joints. This corresponds to a joint set number of 2.24. The RQD was not logged for holes drilled on the downstream right abutment, but in general the rock was recovered in long sticks with a few fractured zones. Based on core recovery numbers and field observations, the average RQD is probably about 90-95%, with isolated areas ranging down to about 80%. This results in K_b values between about 35.7 and 42.4. The intensely fractured beds would have an RQD of about 17% based on field measurements. This corresponds to a K_b value of about 7.6. The concrete was placed in 4-foot (1.2m) lifts using large blocks generally encompassing the entire thickness of the dam. The contraction joints are widely spaced (35 to 60 feet (10.7 to 18.3 m)) and keyed. Although some lift lines exhibit minor seepage at high reservoir elevations, the lifts were cleaned well and also keyed. The value of K_b for the concrete should be high, say about 80 or higher.

K_d represents the interblock frictional resistance. It can be estimated as the ratio of the joint roughness number over the joint alteration number (J_r/J_a), which is roughly equivalent to the tangent of the friction angle. Based on field observations, the limestone bedding planes are rough and planar, while the joints are very rough and irregular. The bedding strength would likely control the removeability. Therefore, the joint roughness was assumed to be rough/planar ($J_r = 1.5$). The J_r for concrete would be described as "stepped" since all the joints are keyed, resulting in a value of about 4.0. The majority of foundation joints were reported as calcite

healed or clean and tight, increasing in tightness with depth from the surface (although one joint open up to 3 inches (76 mm) wide at the surface was observed on the right abutment). This results in a J_a of about 0.75 to 1.5. Thus, K_d would range from about 1 to 2. For the intensely fractured beds, the joint roughness number would tend toward the value for rough/planar (1.5), and the joint alteration number could be as high as 2.0, resulting in a K_d value of about 0.75. For the concrete, the joints would be considered to be healed (lift lines) or tight and clean (contraction joints), resulting in J_a between 0.75 and 1.0, and K_d between 4.0 and 5.3.

The relative ground structure number (K_s) represents the orientation of the discontinuities relative to the impinging water, and takes into account the block shapes (long and narrow or roughly cubic). The orientation of the beds is extremely regular, striking 5 to 8 degrees west of north (about cross-canyon) and dipping to the east at angles ranging from 70 to 86 degrees west. The abutments give the appearance that the open bedding planes are spaced roughly twice as close as the open joints. Although the apparent dip of the bedding changes with respect to the plunging jet in relation to the curvature of the dam, an angle of 70 degrees against the flow (beds dip upstream) was assumed on the average. This results in K_s of about 0.9. This value would also apply to the intensely fractured zones. For the concrete, a K_s value of 1.0 would be appropriate.

In summary, the following values of erodibility index (K) are estimated:
Foundation Rock: 5100 - 12,000
Intensely Fractured Beds: 200-400
Concrete: 6400 - 8500

The streampower (defined as the rate of energy dissipation) is low as flow just comes over the top and impinges directly onto the upper abutments, and becomes larger as the fall height increases toward the channel when the reservoir reaches its peak (reflected in the energy available). At maximum overtopping, the depth over the crest (3.2 feet (1.0 m)) corresponded to roughly 19.2 cfs/ft (1.78 cms/m) of dam crest. Calculation of stream power for this case is fairly complicated, and the numerical model developed as part of an ongoing research effort was used for these conditions. These computations indicated the stream power ranged from a low value of 43 kW/m^2 at the upper abutments (fall height of 9 feet (2.7 m)) up to 258 kW/m^2 near the central part of the dam (fall height of 180 feet (54.9 m)).

The results are plotted in figure GM-5. The foundation rock and concrete would not be expected to erode based on these results. In general it was felt that the rock and concrete performed extremely well, indicating they should fall well below the threshold for erosion. This is particularly true when it is recognized that the parameters used to estimate erodibility index are based largely on the conditions remaining after the 1964 overtopping event. The intensely fractured zones would be expected to erode, except perhaps near the crest. Although the observed amount of erosion in these zones was not excessive, this is believed to be consistent with the observed behavior.

The results of this study support the conclusion that there probably was some erosion of weak rock (intensely fractured beds) or damaged concrete in areas where there was little dissipation of energy from the tailwater. There was some surficial erosion and scouring of loose material during the experienced overtopping, but not much, as judged from the condition of the foundation after the overtopping. The concrete should not be vulnerable to erosion provided that the concrete remains intact and there isn't degradation of the concrete by cracking, freeze-thaw action, or vandalism. The decision to protect the intensely fractured beds appears to be sound under any scenario. The areas of abutment rock most susceptible to erosion for higher overtopping flows have been protected.

Discussion: Every tributary to the Missouri River, from the Canadian border in the North to the Great Falls area in the South, experienced record flows that dramatically exceeded the peak annual flows previously recorded. This storm overtopped and destroyed Lower Two Medicine Lake Dam in Glacier National Park and totally obliterated Swift Dam on Birch Creek. The failure of Swift Dam caused a peak flow in Birch Creek, near the town of Dupuyer, to be 881,000 ft^3/s (24,900 m^3/s) where the previous record peak flow was 7,000 ft^3/s (198 m^3/s). Every highway or bridge that crossed any of the river valleys in the northern half of Montana was washed away and many of the small towns along the foothills area were inundated with very little warning. Electric power disruptions prevented radio warnings from being effective and the rapidly moving flood waves caught many residents with almost no time to evacuate. With less than 1.5 hours of warning, at least 27 people are known to have lost their lives in the flooding associated with this storm. Total storm economic damage in Montana was estimated to be $55 million (1964 dollars). Flood damage in the Sun River Basin was $12.3 million and required the evacuation of approximately 3000 people.

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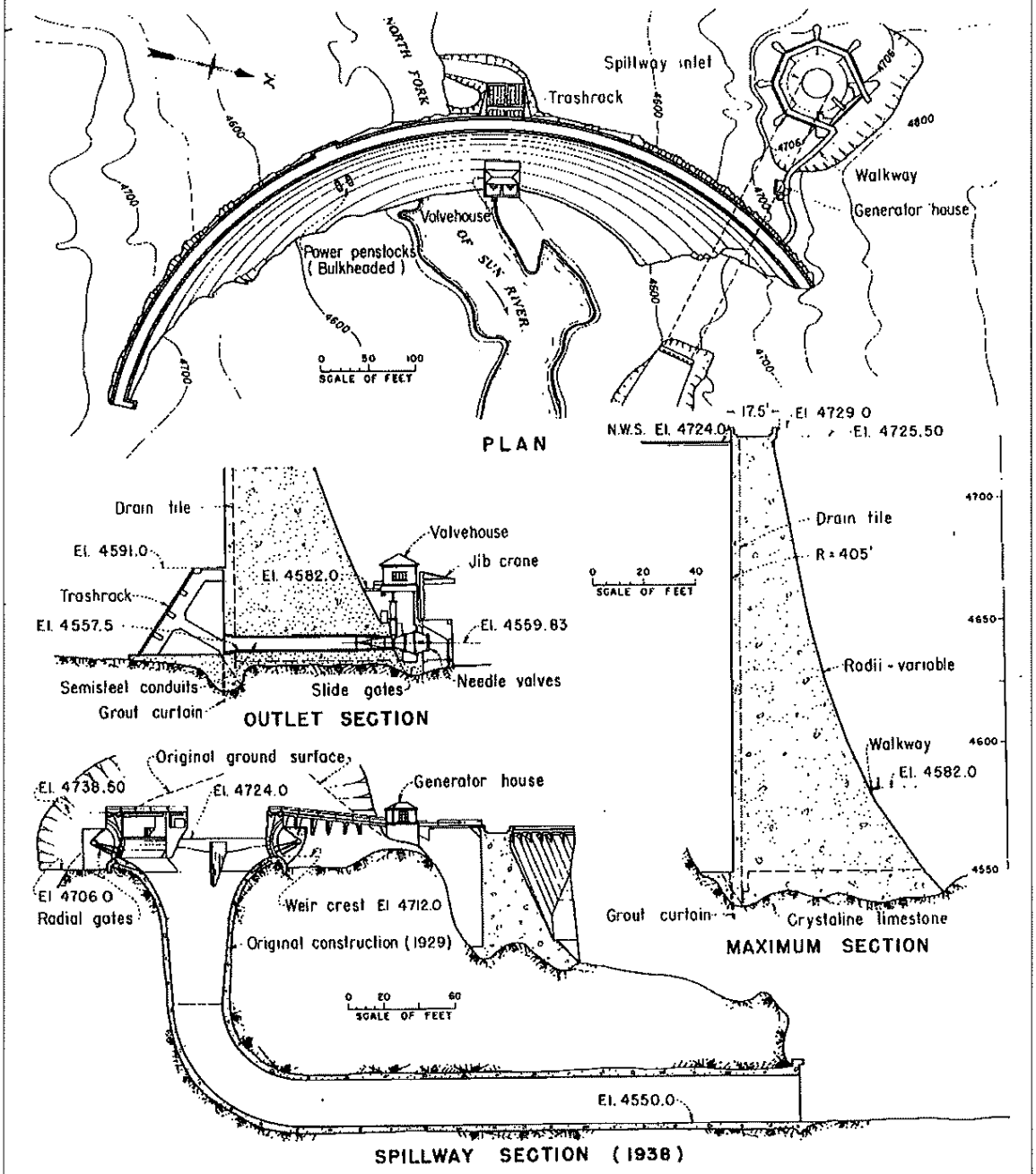
Sun River Project



Gibson Dam, Plan and Sections

Figure GM-1

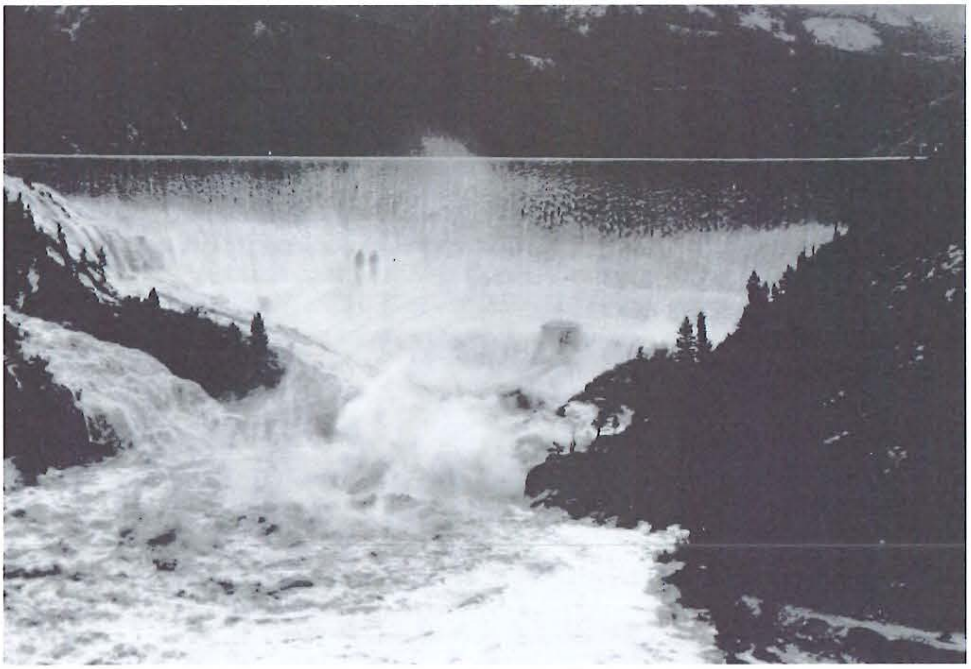


Figure GM-2. Overtopping of Gibson Dam in 1964

Figure GM-3. Overtopping protection on left abutment

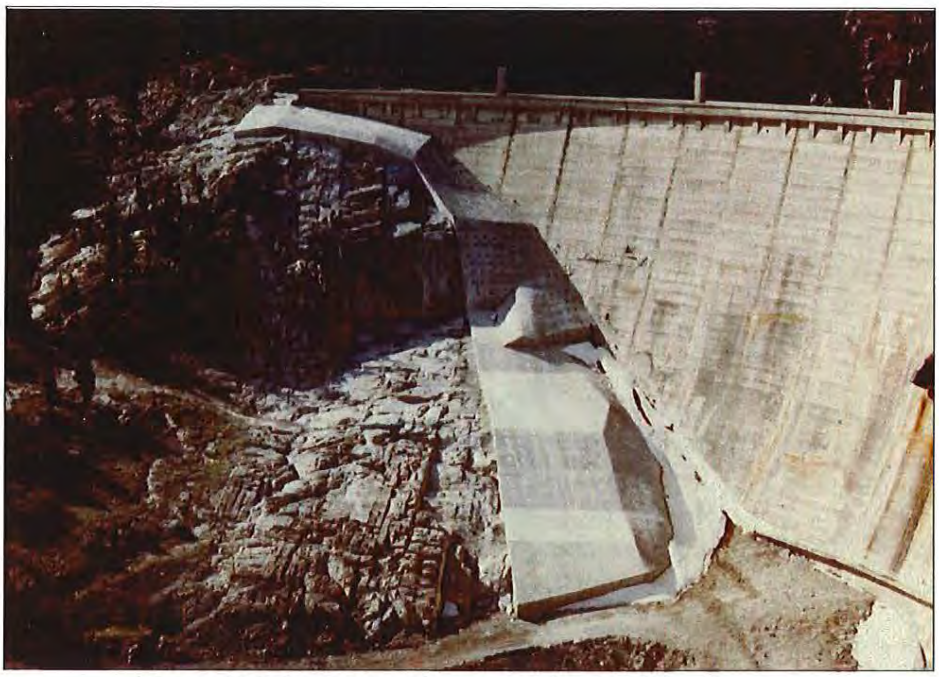


Figure GM-4. Overtopping protection on right abutment

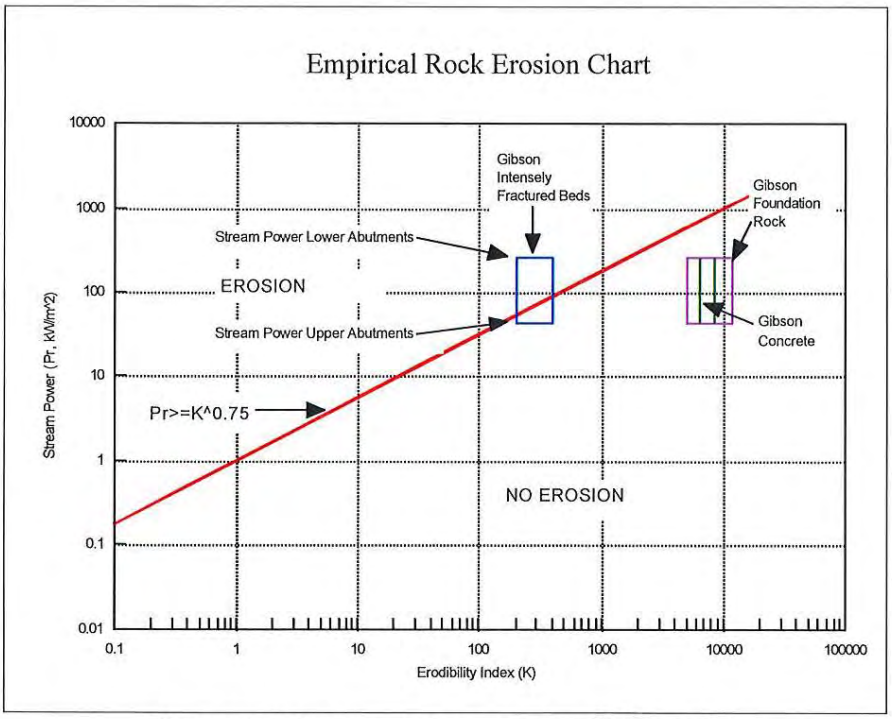


Figure GM-5. Stream Power vs. Erodibility Index