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I have read the paper by Kan et al. (2017) with interest because of our involvement in assessing seismic stability and deformations of embankment dams located in seismically active regions of the western United States. We also check validity of our analysis methods and procedures with case studies. Generally, we find a reasonable comparison between the computed results and observed performance in the field. However, such is not the case for Kan et al. (2017) with their case study of Zipingpu Dam – the computed deformations using about twelve different Newmark-based procedures listed in Table 3 of the paper differ significantly (i) amongst themselves, and (ii) from the actual observed values. Moreover, computed deformations from eleven of the twelve procedures are significantly smaller (under-prediction) than the actual deformations, and those from the one exception are significantly higher (over-prediction).

We re-analysed the Zipingpu Dam for permanent displacements along the critical slip surface using the Newmark rigid sliding block analogy – the results compare favourably with the field data; sensitivity of computed displacements to variations in yield acceleration is included. Additional items included herein pertain to: (a) selection of earthquake component for use in Newmark-based analysis, and (b) natural vibration characteristics of the dam.
Dam orientation and seismic data

Kan et al (2017) used the E-W component of the earthquake as recorded in Mao Town scaled to 0.55g for computing the displacements of the dam cross section which is primarily oriented in the N-S direction.

Figure D1(a) shows the directional orientation of Zipingpu Dam and that of the Min River; Fig. D1(b) shows a perspective view of the dam and appurtenant structures. Application of the N-S component, which is in the transverse direction to the dam axis, seems more appropriate.

During the May 12, 2008 event: (i) monitoring stations at the stream level at the Zipingpu Dam site had failed to record peak ground acceleration (PGA), and (ii) the monitoring stations at the dam crest had recorded peak crest acceleration (PCA) of 1.65g along the stream (N-S) direction. This PCA reduces to 0.8g after filtering the high frequency components of the acceleration response. Thus for an estimated PGA of 0.55g at the dam site, the acceleration amplification factor at crest (AFC = PCA/PGA) lays between 1.45 and 3.0; it is listed as 1.6 in Yu et al. (2012).

During the November 6, 2008 seismic event, the monitoring stations at the Zipingpu Dam had recorded a PGA = 0.034g and PCA = 0.08g. This results in AFC = 2.35, Yu et al. (2012).

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1 Figure D1(a) is adapted from Google Earth site: 31.0348° N, 103.5739° E. Figure D1(b) is adapted from one of the images of the dam at the web site: https://www.bing.com/images/search?q=directional%20layout%20of%20zipingpu%20and%20min%20river&qs=n&form=QBIR&sp=-1&pq=directional%20layout%20of%20zipingpu%20and%20min%20river&sc=0-48&sk=&cvid=4A3CB0C0DA4F64987C93855E0C853E.

2 Rationale for filtering a recorded seismic data at the dam crest to remove high frequency components is not clear – considering that the dam material acted as a filter to the propagating wave and the recorded data should be usable as is. As such, we attached a greater credence to the PCA of 1.65g than to the PCA of 0.8g.
Figure D2(a) shows a plot of the three components of motion recorded at the Mao Town station (Taiebat, 2017) that were used for scaling to PGA of 0.55g for the deformation analyses of the dam. Peak acceleration values in the source data components are: E-W: 307 cm/s$^2$; N-S: 302 cm/s$^2$; U-D: 256.0 cm/s$^2$. Thus, the scaling factors for the E-W and N-S components are: 1.7575 and 1.7866, respectively. The U-D component is left unscaled. Figure D2(b) shows a plot of the scaled data in units of g. Figures D2(c and d) show response spectrum and Fourier power spectrum, respectively of the acceleration data in Fig. D2(b). Table D1 summarizes the computed characteristics of the earthquake data used.

There are significant differences in the seismic data used in Kan et al. (2017) and the plots shown in Figure D2: (a) the time-acceleration plot for the E-W component shown in Fig. 12 of the paper differs from the E-W component plot shown in Fig. D2 – in principle, they should be identical but they are not; (b) predominant period of the earthquake is listed as 0.12 s whereas it is calculated to be 0.146 s as shown in Fig. D2(c); (c) mean period of the earthquake is listed as 0.21 s where as it is calculated to be 0.309 s as shown in Table D1. Rationale for using the E-W component rather than the N-S component for analyses needs to be explained.

Newmark rigid sliding block analysis

Kan et al. (2017) used twelve different versions of the Newmark procedure (Newmark, 1965) to demonstrate that the computed displacements for the dam differ greatly (i) amongst themselves,

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3 This confusion on which of the two components (N-S or E-W) was actually used in Kan et al. (2017) is also observed in Kan and Taiebat (2016). Therein, the text reads use of N-S component but the Fig. 13 caption therein showing the plot of data reads E-W component. There is also a mention of filtering frequencies > 20 Hz, and performing base-line correction. However, rationale for filtering and its effects on computed results are not given.
and (ii) from the displacements observed in the field. Average induced accelerations for the sliding blocks were based on continuum-mechanics based FLAC analyses.

In our re-analysis, we used the Newmark rigid sliding block procedure for computing permanent deformations – details of the procedure and its implementation in a computer program are given in Chugh (1980, 1982). In brief, the procedure requires: (i) time-acceleration data at the base of the dam; (ii) yield acceleration for the slip surface with lowest computed static factor of safety; (iii) inclination angle, \( \alpha \), of the base of one of the slices used to discretize the slide mass in the static slope stability analysis; (iv) inclination angle, \( \alpha_1 \), of the adjacent slice in the direction of movement; and (v) an amplification factor for the base motion to the selected slice. The calculated displacement, \( \delta \), is a translation of the slice along the planar surface with inclination \( \alpha \), and an angular rotation \( (\alpha - \alpha_1) \). The displacement of any other point on the slide mass can then be computed by geometry.

For the Zipingpu Dam, we used the shear surface with minimum FS and yield acceleration of 0.265g shown in Figure 14(a) of Kan et al. (2017). Since, our interest herein is to calculate the vertical displacement of the crest of the dam, we selected the slice adjacent to the crest of the dam and scaled the inclination angle, \( \alpha \), with the horizontal to be \( \sim 45^\circ \) and \( \alpha_1 \) to be \( \sim 44^\circ \). For the earthquake data, we used (i) the N-S component scaled to 0.55g; and (ii) the E-W component scaled to 0.55g. For application to the selected slice, the accelerations in (i) and (ii) were multiplied by an amplification factor from 1.0 to 3.0 in increments of 0.1 – specifically, the values used are: 1.0, 1.1, 1.2, 1.3 ..., 3.0. In addition, the above analyses were repeated with inclusion of U-D component. Computed displacement results are shown in Figure D3(a).
For the N-S component and AFC in the range of 2.4 to 3.0 (AFC = 2.4 is from the November 6, 2008 seismic event and AFC = 3 is from the May 12, 2008 seismic event), the computed displacement of the dam crest along the 45° slip surface is in the range of 1.0 to 1.9 m without the U-D component, and in the range of 1.1 to 2.1 m with the U-D component included. Therefore, our computed permanent vertical and horizontal displacement of dam crest is in the range of 0.8 to 1.5 m each with a best estimate of 1.1 m corresponding to AFC of 2.7 (average of the AFC of 2.4 and 3.0). The time-history of computed displacement, $\delta$, of the dam crest is shown in Fig. D3(b). The measured displacements at the crest of Zipingpu Dam are: 1 m vertical and 0.6 m horizontal. This compares well with the computed displacements – knowing that the PGA of 0.55g is an estimated value and that the actual displacements in the field are of a complex three-dimensional (3-D) facility, Fig. D1(b).

For the E-W component, the computed results are included in Figure D3(a). No interpretation in terms of comparison with the field data are included herein because AFC value is not known. It suffices to say that the computed displacements are significantly higher than those for the N-S component (with and without the U-D component).

*Sensitivity analysis*: Newmark’s step-by-step integration procedure is essentially a summation process for areas under curves. In this sense, the procedure is unconditionally stable and there are no restrictions on its numerical stability due to magnitudes of displacements. However, an engineer needs to be concerned with the physical instability of the dam that is associated with large displacements. If for an estimated decrease in yield acceleration of the slide mass, the
computed displacement does not increase asymptotically, then it is reasonable to rely on the computed displacement. Otherwise, the results are indicative of physical instability of the dam and must be investigated by other means; see Chugh (1995) for details.

For the Zipingpu Dam, a sensitivity analysis was performed to see non-linear nature of increases in displacements with decreases in yield acceleration. The results are shown in Fig. D3(c). To use this plot, one needs to know the reduction in strength of rockfill with increasing displacement – however, we do not have this information.

Natural vibration characteristics of the Zipingpu Dam

Kan et al. (2017) used Makdisi and Seed (1979) procedure to calculate the fundamental period, $T_0$, of the dam; its value is determined to be 0.753 s. This corresponds to a fundamental frequency, $f_1$, of 1.328 Hz. ($f_1 = 1/T_0$). It is worth noting that Makdisi and Seed procedure is based on shear beam theory with other engineering simplifications included. Equations 1 – 7 in the paper are also used for estimating fundamental period of a dam.

We calculate natural frequencies of a dam using a unit pulse procedure described in Chugh (2006). In brief, the procedure uses actual geometry of dam, material properties, and prevalent reservoir and pore water pressure conditions. The stress conditions in the model are simulated using incremental-construction of the dam. The model is excited by application of an impact force of prescribed magnitude in the form of a single, high frequency, sine pulse of short duration and problem solved for a finite length of time and saving displacement-time history at
select locations in the model. At the end, a fast Fourier transform (FFT) of the displacement-time history gives all the natural frequencies of the dam model. Since vibration frequencies and mode shapes are paired characteristics of a dam, it is essential to see the mode shapes associated with the natural frequencies to assure that the frequencies determined are indeed resonant frequencies. The mode shapes are determined by applying an excitation force of prescribed magnitude in the form of a sine wave of desired frequency for a finite length of time. At the end of the analysis, the deformed configuration of the dam is the mode shape associated with the specified frequency. See Chugh (2006) for details. The procedure is implemented in the computer program FLAC (Itasca 2006).

The Zipingpu Dam was constructed in three distinct stages identified as stage 1, stage 2, and stage 3. Figure D4(a) shows the 2-D numerical model of the completed dam cross section; it is discretized using a 30×24 finite-difference grid – the discretization used matches the dam configuration at the end of each of the three construction stages in the field. In the numerical analysis, the dam construction was simulated in 24 lifts, one lift per zone in the vertical (y) direction. The material properties used are: ρ = 2160 kg/m³, e₀ = 0.259, v = 0.3, c = 15 kPa, φ = 45⁰, shear modulus, G, was calculated using Eq. 8 in Kan et al. (2017) and bulk modulus, K, was calculated using the elastic relation, K = 2G(1 + v)/(1 - 2v). On the upstream face of the dam, pressure due to the reservoir water level (RWL) of 828.7 m was included. Pore water pressure in the dam body was taken to be null because the concrete facing was fully functional as an effective seepage barrier prior to the earthquake. Figure D4(b) shows the contours of computed settlements at the end of each of the 3-stages of the dam construction, and with the reservoir water pressure. For natural frequencies, the completed model with reservoir water pressure was excited using a single sine pulse of 1kN force at 100 Hz frequency for a duration of
0.01 s applied at two locations on the dam crest as shown in Fig. D5(a) and the problem solved for a dynamic time of 20 s. Figure D5(b) shows the x-displacement-time history at the crest of the dam; and Figure D5(c) shows the FFT of the displacement-time history. The frequencies corresponding to the spikes in the FFT plot correspond to the natural vibration frequencies of the dam; first natural frequency, $f_1$, is 1.70 Hz – this corresponds to the fundamental period ($T_o$) of 0.588 s which is lower than the 0.753 s used in the paper. Higher natural frequencies of vibration can be read off from the plot of the FFT results shown in Fig. D5(c). The first six natural frequencies of the dam are: $f_1 = 1.70$, $f_2 = 3.00$, $f_3 = 4.55$, $f_4 = 5.40$, $f_5 = 6.25$, and $f_6 = 7.50$ Hz; Fig. D5(d) shows the model setup for mode shapes, and Fig. D5(e) show the mode shapes associated with each of the six natural frequencies of the dam. For higher natural frequencies, one needs to increase the range of the FFT plot.

**Comparison of re-analysis results with the proposed reliability criteria in Kan et al. (2017)**

1. Figure 10 in Kan et al. (2017) suggests that computed displacements from Newmark-based analyses would be considered non-conservative if the fundamental period ($T_o$) of a dam is > 0.45 s. This criteria does not hold true in our analysis of the Zipingpu Dam because we computed the period of the dam to be 0.588 s, yet our computed displacements from the Newmark rigid sliding block analysis compare well with the field values.

2. The $K_{2\text{max}}$ for the Zipingpu Dam corresponding to our computed period of 0.588 s is approximately 155 according to the Fig. 10 in Kan et al. (2017). This compares well the $K_{2\text{max}}$ of 170 for the rockfill shells (zone 3) in Oroville Dam (a 235 m high rockfill dam with impervious
clayey core) located in California, USA, – see Banerjee et al. (1979) for details. The analysis results in Kan et al. (2017) suggest $K_{2\text{max}} \approx 90$ which is obviously too low in view of the past studies on rockfill dams.

**General comments**

1. The conceptual model envisioned in the Newmark rigid sliding block method is a dynamic equivalent of the conceptual model used in the static slope stability analysis. Thus, the Newmark procedure is applicable for a failure mode in which a rigid mass of material slides downslope on a curved slip surface without distortion. If the failure mode under study is other than that envisioned in the Newmark method, then it should not be used.

2. As engineers, we all tend to temper with the rigors of mathematical and conceptual models to include some of the entities considered important for our problems or exclude some of the entities considered not so important. This may be the reason for the development of different procedures listed in Table 3 in Kan et al. (2017). For meaningful results, it is essential for an engineer to know the scope of applicability of a procedure before using it for a problem under study.

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$K_{2\text{max}}$ value of 155 for the Zipingpuu Dam and 170 for the Oroville Dam are based on 2-D plane strain numerical model studies.
3. For the Zipingpu Dam, instrumented data during the November 6, 2008 seismic event should be used to determine its fundamental and higher frequencies of vibration. This will provide a check on the validity of different procedures being used in the dam engineering profession.

4. Since the mean period, $T_m$, and mean square frequency, $f_m$, as defined in Rathje et al. (1998), and used in Table D1 herein, do not have an inverse relation, i.e. $T_m \neq 1/f_m$, it is not clear as to how the tuning ratio, $T_0/T_m$, relates to $f_0/f_m$. Since tuning ratio is a dimensionless parameter, how one should calculate mean frequency of an earthquake which bears an inverse relation with the mean period? Engineers prefer to think in terms of frequency; seismologists prefer the term period.

5. To assess damage causing potential of earthquakes to a particular dam, we compare the natural frequencies of the dam with the frequency contents of the earthquakes. An earthquake with larger number of frequency matches is considered more damaging to the facility than another earthquake with fewer number of matches.

**Summary**

1. There are significant differences in our re-analysis results from those included in Kan et al. (2017). It will be helpful to know of the authors’ views on the items covered in this discussion.
2. Performance of the Zipingpu Dam during the May 12, 2008 Wenchuan earthquake presents an instructive case study for checking validity of different seismic analysis procedures commonly being used in dam engineering practice.

Acknowledgements

We are thankful to Prof. Mahdi Taiebat for his sending us the time-acceleration data used in our analysis of the Zipingpu Dam. Thanks are also due to Cindy Gray for her assistance in preparing the figures.
References:


Taiebat, M. 2017. Personal communication.

Figure captions

Fig. D1. Layout of Zipingpu Dam: (a) directional orientation of the dam; (b) perspective view of the dam and appurtenant structures.

Fig. D2. Seismic data recorded at Mao Town station: (a) east-west (E-W), north-south (N-S), and up-down (U-D) components of acceleration; (b) scaled acceleration data for a peak acceleration of 0.55g for the Zipingpu Dam site – U-D component is not scaled; (c) response spectrum of the seismic components in (b); (d) Fourier spectrum of the seismic components in (b).

Fig. D3. Computed displacement results from the rigid sliding block analysis: (a) computed displacements for different amplifications of ground motion (Fig. D2(b)); (b) incremental displacement history; (c) sensitivity of computed displacements to variations in yield acceleration.

Fig. D4. Discretized model of the Zipingpu Dam cross section for calculating initial stresses in the model: (a) dam cross section details; (b) settlements at the end of each of the 3-stages of dam construction and reservoir water pressure.

Fig. D5. Natural vibration characteristics of Zipingpu Dam: (a) problem setup for natural frequencies determination; (b) x-displacement history at the dam crest; (c) natural frequencies via FFT power spectrum of (b); (d) problem setup for mode shape determination; and (e) natural mode shapes (magnified).
Table D1. Computed characteristics of the earthquake data.

<table>
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<th>Earthquake component ID</th>
<th>Analysis type</th>
<th>Predominant period, $T_p$ (s); damping = 0</th>
<th>Dominant frequency (Hz)</th>
<th>Mean period, $T_m^*$ (s)</th>
<th>Mean square frequency, $f_{m*}$ (Hz)</th>
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<td>E-W</td>
<td>Response spectrum, Fig. D2(c)</td>
<td>0.146</td>
<td>1.949</td>
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<td>U-D</td>
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<td>Dominant frequency (Hz)</td>
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$*$ \[ T_n = \frac{\sum C_i^i \left( \frac{1}{T} \right)}{\sum C_i^i} \]

\[ f_n = \frac{\sum C_i^i f_i}{\sum C_i^i} \]

$0.25 \leq f_i \leq 25$ Hz; $C_i =$ Fourier amplitude; $n =$ number of points in the Fourier transform (Rathje et al. 1998)
Fig. D1

62x23mm (300 x 300 DPI)
Fig. D2

221x312mm (300 x 300 DPI)
Fig. D2 ... contd.

229x340mm (300 x 300 DPI)
Fig. D3

211x379mm (300 x 300 DPI)
Fig. D4

170x175mm (300 x 300 DPI)
Fig. D5

162x159mm (300 x 300 DPI)
Fig. D5 ... contd.

198x237mm (300 x 300 DPI)