ASPECTS OF CONCRETE DAMS RESPONSE TO NEAR-FIELD GROUND MOTIONS

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SUMMARY

Reliable analytical procedures to predict near-field effects on earthquake response of concrete dams are essential to design dams to be earthquake resistant or evaluate the earthquake safety of existing dams. Today ICOLD has adopted an approach based on two levels of earthquakes similar to the one used in nuclear industry, i.e. the design basis earthquake (DBE) and the maximum credible earthquake (MCE). But the Northridge and Hyogoken-Nanbu (Kobe) earthquakes have revealed that near-field ground motions have very damaging effects on structures. Researchers believe that at least some of these failures may have been the result of near-field effects that were not adequately taken into account in previous seismic design guidelines. It is crucial that these near-field effects be identified and thoroughly understood, and that appropriate mitigation measures be found to deal with these special ground motions. Beginning with Landers earthquake of 1992, strong motion data began to be recorded from near-field stations located within a few kilometers of the plane of fault rupture. These ground motions were observed to differ dramatically from their far-field counterparts. They were characterized by distinct large amplitude single or multiple pulses, large velocity pulses, which would be viewed as damaging criteria by engineers, forward rupture directivity and larger ratio of vertical-to-horizontal components ratio (V/H). Other records from U.S (for example Pacoima Dam site) and Japan show similar pattern.

In this investigation the effects of near-field ground motions on concrete dams are assessed using a collection of some records from actual earthquakes. All of these records exhibit main characteristics of near-field ground motions. Another record, the 1940 El Centro motion in which near-field effects are absent, is used as a reference. The effects of near-field ground motions on displacements, and stresses of upstream and downstream faces of an arch dam, and on stresses, displacements, and base sliding of a concrete gravity dam have been evaluated. Based on these results some general conclusions have been drawn.

INTRODUCTION

Concerns about the seismic safety of concrete dams have been growing during recent years, partly, because the population at risk in locations downstream of major dams continues to expand and also because it is increasingly evident that the seismic design concepts in use at the time most existing dams were built were inadequate. Since the Northridge and Hyogoken-Nanbu (Kobe) earthquakes, there has been much discussion about the adequacy of design practice of concrete dams. Such an examination occurs after every damaging earthquake, and, in fact, the seismic provisions of structures are based largely on experience from actual earthquakes.

The hazard posed by large dams has been demonstrated since 1928 by the failure of many dams of all types and in many parts of the world. However, no failure of a concrete dam has resulted from earthquake excitation; in fact the only complete collapses of concrete dams have been due to failures in the foundation rock supporting the dams. On the other hand, two significant instances of earthquakes damage to concrete dams occurred in the 1960s: Hsinfengkiang in China and Koyna in India. The damage was severe enough in both cases to require major repairs and strengthening, but the reservoirs were not released, so there was no flooding damage. This excellent safety record, however, is not sufficient cause for satisfaction about the seismic safety of concrete

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dams, because no such dam has yet been subjected to maximum conceivable earthquake shaking while retaining a maximum reservoir. For this reason it is essential that all existing concrete dams in seismic regions, as well as new dams planned for such regions, be checked to determine that they will perform satisfactorily during the greatest earthquake shaking to which they might be subjected especially in the near -field regions.

This paper deals with concrete dam response during moderately large earthquakes and focuses on near-field effects. The first section reviews current information about characteristics of near-field ground motions. Then by using a suite of ground motions recorded in the near-field of recent earthquakes, a detailed study of the response of one concrete gravity dam and of an arch dam are presented.

NEAR-FIELD GROUND MOTIONS

A lot of major dam sites is located alongside major active faults and so could be subjected to near-field ground motions from large earthquakes. Knowledge of ground motion in the near-field region of large earthquakes is limited by the scarcity of recorded data. The near-field of an earthquake (also called near-source or near-fault region) is the region within which distinct pulse-like particle motions are observed due to a coherent release and propagation of energy from the fault rupture process. For damaging earthquakes, the near-field region may extend several kilometers outward from the projection on the ground surface of the fault rupture zone and its extension to the surface, particularly in the direction of rupture propagation. The near-field ground motions are characterized by high peak acceleration (PGA), high peak velocity (PGV), high peak displacement (PGD), pulse-like time history, and unique spectral content. The nature of near-field ground motions differs significantly from that of far-field ground motions. A basic assumption in the development of design ground motions in engineering practice is that the level of strong ground motion is primarily dependent on the magnitude of the earthquake, the distance of the site from the causative fault, and the site category. Although strong motion data from recent earthquakes have confirmed the basic validity of this assumption, these data also indicate the important influence of more detailed aspects of the earthquake source, the propagation path, and the local site conditions. This is manifested in the wide variations that were observed in the levels of ground motions at different sites located at similar distances in each of these earthquakes, especially in the near-field regions. The strong motion data clearly indicate the presence of systematically larger ground motions in the fault-normal direction than in the fault-parallel direction close to faults. The ratio of fault-normal to fault-parallel motions grows with increasing magnitude, increasing fault proximity and increasing period. Further, recent destructive earthquakes have shown signs of damage occurring in preferred directions that correspond to fault-normal (north in the 1994 Northridge earthquake; northwest in the 1995 Kobe earthquake). The directional frequency content and amplitude level in near-fault strong motion have fundamental consequences in geotechnical and structural engineering. This implies that the dynamic response of structures, especially tall buildings, base isolated buildings, bridges, dams, will be influenced by their orientation relative to the ground motion and by their proximity to causative fault. Table 1 lists suite of near-field ground motion records (except El Centro record, which does not have any near-field effects) used in present study. The median peak ground acceleration and velocity from this table in the fault normal direction are 0.76g and 118 cm/sec, respectively; both are high values. Peak velocity is often viewed, as a better indicator of damage potential than is peak acceleration. Figure 1 shows the response spectra for two of the ground motion records. Except for El Centro record for which near-field effects are absent, for all other near field records considered in this study, fault-normal spectra are at least two times larger than fault-parallel spectra for a specific range of period, which is indicative of presence of forward rupture directivity in such ground motions.

RESPONSE OF AN ARCH DAM

Morrow-Point Dam an almost perfectly symmetric arch dam, 142m height, 219m crest length, 3.7m crest thickness, 16m base thickness, is considered for this part of study. The complete dam-foundation rock-reservoir system is considered in a linear analysis. The dam body and reservoir are modelled by finite element, and for foundation rock modelling boundary element is used. The reservoir is assumed to be full. Based on the available data regarding the material property of dam and foundation rock and based the on the forced vibration measurements conducted to measure dynamic characteristics of the dam, our numerical model validity has been

Table1: Suite of ground motion used in the analyses.

Earthquake	Date	Station	Rupture Distance (km)	Site Code	Peak Horizontal Acceleration (g)		Peak Horizontal Velocity (cm/s)	
					FN	FP	FN	FP
Imperial Valley	5/19/40	El Centro	10	SL	0.21	0.32	32.2	60.1
San Fernando	2/9/71	Pacoima Dam	3.3	HR	1.17	1.08	114.9	59.3
Loma Prieta	10/17/89	Los Gatos Presentation center	3.5	HR	0.66	0.44	105.5	57.4
Erzincan, Turkey	3/13/92	Erzincan, Turkey	2.0	SL	0.43	0.46	120.2	65.4
Landers	6/28/92	Lucerne Valley	1.1	SL	0.76	0.73	127.5	95.3
Northridge	1/17/94	Pacoima Dam Downstream	8.0	HR	0.5	0.24	48.5	18.9
Northridge	1/17/94	Pacoima Dam Left Abutment	8.0	HR	1.37	1.46	107.3	46.0
Northridge	1/17/94	Rinaldi Receiving Station	7.1	SL	0.89	0.39	178.4	67.5
Northridge	1/17/94	Sylmar County Hospital	6.4	SL	0.73	0.59	122.2	54.3

SL: Soil and Alluvium; **HR**: Hard Rock ; **FN**: Fault Normal; **FP**: Fault Parallel

checked and fairly good agreement was obtained between calculated and measured dynamic characteristics of the dam. All of the horizontal components of the ground motions have been rotated to fault-normal (FN) and fault-parallel (FP) directions, and time history analysis of the dam has been conducted by using three components of the records simultaneously. Table 2 lists the envelope of maximum arch and cantilever stresses on the upstream and downstream faces of the dam along with the displacement of the crest. All of the responses include static loads from structures self weight and hydrostatic pressures of the reservoir. Before discussing these results in detail, we have to bear in mind two important points. First, an analysis assuming that an arch dam is a monolithic structure invariably shows net tensile stresses in arch direction: the dynamic tensile stresses in the arch direction exceed the static compressive arch stresses. However, arch dams are constructed as cantilever monoliths separated by contraction joints, and joints cannot develop the tensile stresses indicated in linear analysis. The joints can be expected to open and close during an earthquake, producing a significant redistribution of stresses. The release of arch stresses at the contraction joints transfer forces to cantilevers. Second, because concrete does not demonstrate a linear relationship between stress and strain, except at relatively low levels of applied loading, and because most seismic evaluations are based on linear elastic analyses rather than non-linear analyses, some investigators have proposed the use of an apparent tensile strength rather than actual tensile strength for such evaluations. The apparent tensile strength is equal to the twenty percent of static uniaxial compressive strengths for the range of compressive strengths common for concrete used to construct dams, i.e. about 5.5-6 MPa. By considering all of what mentioned above, the stress level for all of the ground motions except El Centro, are well above the tensile strength of the concrete and the cracking of the dam is inevitable for such overstressed cases. Under such severe ground motions and according to our

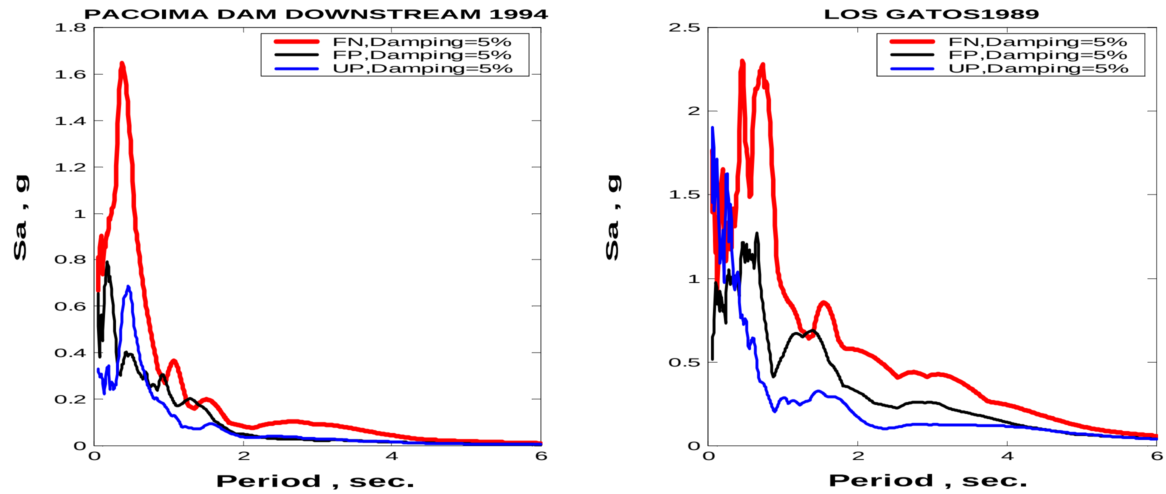

Figure 1: Response spectra for Pacoima dam downstream (1994) and Los Gatos (1989) records.

Table 2:Arch, cantilever stress, and displacement of the Morrow Point dam due to suite of ground motion

Ground Motion Record	Upstream				Downstream				Displacement at the Crest (mm)	
	Arch Stress (MPa)		Cantilever Stress (MPa)		Arch Stress (MPa)		Cantilever Stress (MPa)			
	FN	FP	FN	FP	FN	FP	FN	FP	FN	FP
El Centro	2.11	2.86	1.36	1.47	1.67	2.1	0.79	1.03	24.8	31.7
Pacoima Dam	10.74	17.44	5.94	7	9.34	14.16	2.8	3.01	112.3	112.6
Los Gatos Presentation center	8.76	4.34	4.38	3.67	8.2	3.79	1.99	1.84	64.6	60.8
Erzincan, Turkey	3.02	6.64	2.3	2.67	2.65	5.51	1.7	1.19	43.8	53.8
Lucerne Valley	4.51	6.56	2.63	2.38	4.04	4.42	2.11	2.22	45.4	44.1
Pacoima Dam Downstream	6.1	3.1	3.75	1.86	6.26	1.6	1.56	0.97	61	20.6
Pacoima Dam Left Abutment	21.51	23.67	8.17	6.64	19.43	21.72	6.16	4.74	184.6	149.2
Rinaldi Receiving Station	13.01	7.08	4.46	3.05	10.13	6.77	3.14	2.37	87.1	58.3
Sylmar County Hospital	8.62	10.61	2.84	4.69	7.64	9.87	1.48	2.12	64.8	77

analysis results cantilevers become overloaded, possibly resulting in crushing or horizontal cracking of the cantilevers. It must be reminded that our analysis does not consider joint openings. It is evident that by considering the joint openings and after load transfer to cantilevers the overloading of them will be much more than the case with closed joints. Based on the frequency contents of the ground motions and vibration frequency of the dam, the arch stresses in FN and FP directions on the upstream and downstream faces of the dam for every record show variations more than two times. The largest increase for maximum arch stresses, are 102, 97, 84 percent for Los Gatos, Pacoima dam downstream (1994), and Rinaldi Receiving Station records, respectively. In the downstream the largest increase for maximum arch stress are 291, and 116 percent for Los Gatos and Pacoima downstream records. In the case of Erzincan record maximum increase of 120 percent occurs in FP direction. The variation of the cantilever stresses in FN and FP directions is less than the arch stresses. Detailed discussion on variation of arch and cantilever stresses on the upstream and downstream face of dam could be found in [Ohmachi, and Jalali 1999]. It must be emphasized that despite of other near-field strong motions the stresses in the dam due to El Centro record, for which near-field effects are absent, remain very low and the variation of stresses in the FN and FP directions are negligible. Figure 2 shows the maximum envelope of arch stresses on the downstream face of the dam for the Los Gatos record in the FN and FP direction.

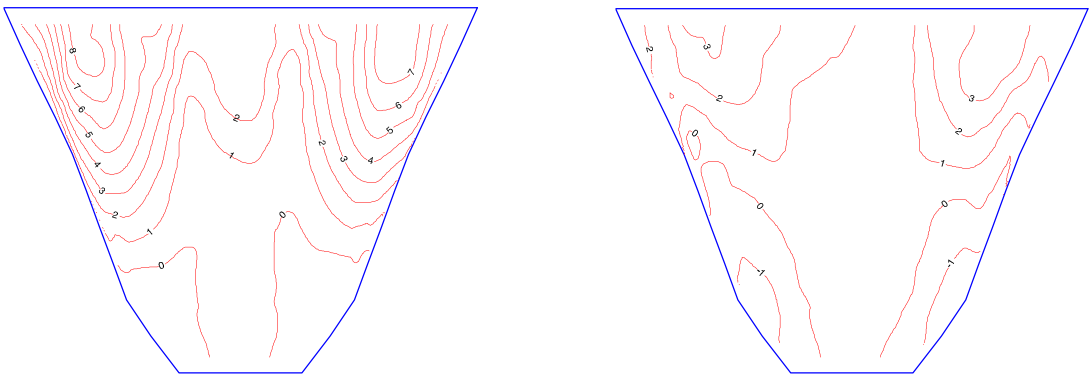

Figure 2: Maximum envelope of arch stresses (in MPa.) due to Los Gatos record in the FN (left), and FP (right) direction.

RESPONSE OF A GRAVITY DAM

Pine Flat Dam, which is a gravity structure consisting of thirty-seven 15.2m wide monoliths, with the tallest monolith 122m high, has been used in this part to demonstrate the aspects of concrete gravity dam responses to near-field ground motions. By having the forced vibration measurement results and adopting reasonable material properties for dam and foundation rock, we have tried to match our two-dimensional mathematical model with real features of the dam behavior. Good agreement was obtained between measured and calculated responses of the dam. The dam is modelled by finite element, the foundation rock is idealized as a homogeneous, half-plane for computing the impedance functions, and reservoir is idealized as a two-dimensional inviscid and compressible domain. Again similar to arch dam case we have rotated horizontal components to FN and FP

Table 3: Principal Stress, base sliding, eccentricity ratio, and displacement of the Pine Flat dam.

Ground Motion Record	Maximum Principal Stress (MPa)				Base Sliding (mm)		Eccentricity Ratio		Displacement at the Crest (mm)	
	Upstream		Downstream							
	FN	FP	FN	FP	FN	FP	FN	FP	FN	FP
El Centro	4.85	6.3	4.79	5.94	0	36.1	0.62	0.92	37.5	60.6
Pacoima Dam	11.19	9.19	12.65	9.27	702.3	59.5	1.29	1.02	126	87.5
Los Gatos Presentation center	10.17	8.37	10.29	7.22	748.3	129.7	1.46	1.04	109.4	79.9
Erzincan, Turkey	6.92	7.71	8.47	7	16.2	34.3	1.03	1.01	80.1	74.1
Lucerne Valley	6.68	6.94	7.83	7.94	0	0.85	0.69	0.66	45	56.7
Pacoima Dam Downstream	8.14	6.01	7.34	4.98	18.5	0	1.12	0.72	83.7	48.2
Pacoima Dam Left Abutment	9.9	13.5	10.61	10.82	472.5	118.6	1.8	1.67	125.3	140.6
Rinaldi Receiving Station	8.51	7.86	8.75	11.74	940.8	50.1	1.1	0.89	81.3	72.7
Sylmar County Hospital	7.01	7.59	7.39	8.64	128.2	164.9	1.05	0.99	71.4	71

directions and have applied each of these components along with vertical component to the dam separately. The response parameters considered in the analyses include principal stresses, base sliding, displacement of the dam crest, eccentricity ratio of base forces, energy due to base sliding, energy due to dam deformation, energy due to foundation displacement, and energy due to input ground motion. We have also compared the responses of two cases of dam with and without base sliding to demonstrate the effect of base sliding on dam responses.

Principal stresses

Table 3 lists the responses of the Pine-Flat Dam due to suite of ground motion records of Table 1. Regarding the stress level, they are very high except for El Centro record. The maximum principal stress of 13.5 MPa. occurs on the upstream face for Pacoima dam left abutment (1994). The stress for other records also is more than or about 10 MPa. This stress level shows that dam will crack especially near the neck region on the upstream face, on the downstream face near the stress concentration caused by the change in geometry of downstream face and near the dam-foundation interface. But as Figure 3 shows the possibility of the crack near the neck region for upstream and downstream face is very high, because the stress level in this part of the dam is the highest. This result is in agreement with shaking table results of Pine Flat dam [Donlon, 1989]. Regarding the variation

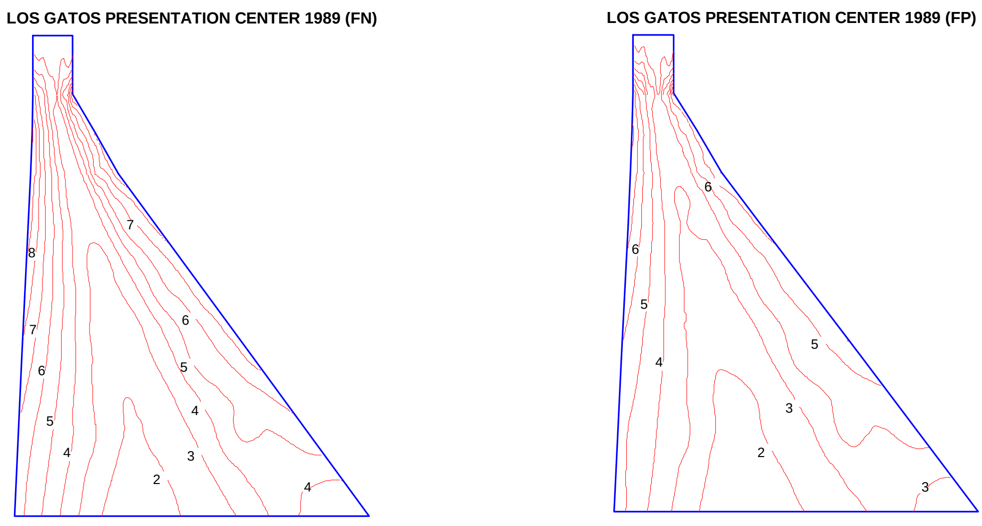

Figure 3: Maximum principal stresses (in MPa.) due to Los Gatos in the FN, and FP directions.

of principal stresses on the upstream and downstream faces of the dam with respect to FN and FP components, we can see maximum increase about 47, 42, and 37 percent in stress from FP to FN for Pacoima dam downstream (1994), Los Gatos (1989), and Pacoima dam (1971), respectively. As can be seen, despite the arch dam case, in this case the increases are not so much in comparison with more than 291 percent increase for arch dam. Again similar to what we observed in the arch dam case, the principal stresses in the upstream and downstream faces of the Pine Flat dam due to El Centro record are much lower than the other records. However, practically there are no differences for FN and FP directions. That is mainly because the arch dam case was three-dimensional linear analysis, but this case is two-dimensional non-linear one. The most interesting point of the Table 3 is the relation between stress level of different ground motions and their peak acceleration. For example the peak accelerations of the Pacoima dam (1971), and Pacoima dam left abutment are 1.8, 2.1, and 2.45, 3.32 times of Los Gatos in FN and FP direction, respectively. But the principal stresses are almost at the same level with the only exception that for the Pacoima dam left abutment, principal stress on the upstream face for FP direction becomes 1.6 times of Los Gatos. It seems that the selection of input ground motion for seismic response of the dams only by its peak acceleration could not be helpful, and we have to consider other reliable parameters especially input energy of the ground motion [Uang, and Bertero 1988].

Base sliding

As pointed out in the previous section, under the severe near-field ground motions, the development of cracks in the dam-foundation interface is inevitable. One potential failure mode of a gravity dam during an earthquake is extensive cracking and deformation in the zone between the base of the dam and the foundation rock. The interface zone is often a weak link in the transfer of seismic forces between the foundation rock and dam monoliths. Failure of the zone can result in a relative displacement between the dam and the foundation rock, a displacement which is often called a base sliding displacement. Moreover gravity structures are mainly designed to resist horizontal forces with their weight. However, under severe earthquake loads the cohesion at the interface will most likely be significantly reduced after a few vibration cycles. That is why it is usually assumed that no significant cohesion forces will develop between the dam and the foundation to contribute to the resistance against sliding under earthquake loads. In present study we have analyzed Pine Flat dam in two cases, one with considering base sliding assuming zero cohesion, and the second without base sliding. . In the case with base sliding the coefficient of friction is considered as one. The amounts of base sliding for the dam have been listed in the Table 3. As can be seen from the Table 3, the base sliding reaches its maximum (940.8 mm) for the Rinaldi receiving Station record, and the second and third largest base sliding (748.3mm, 702.3mm) occur for Los Gatos presentation center, and Pacoima dam (1971) records, respectively. By comparing different parameters of input ground motions, such as peak acceleration (PA), peak velocity (PV), peak displacement (PD), response spectra, and input energy, it seems that the main cause for such a large sliding displacement in

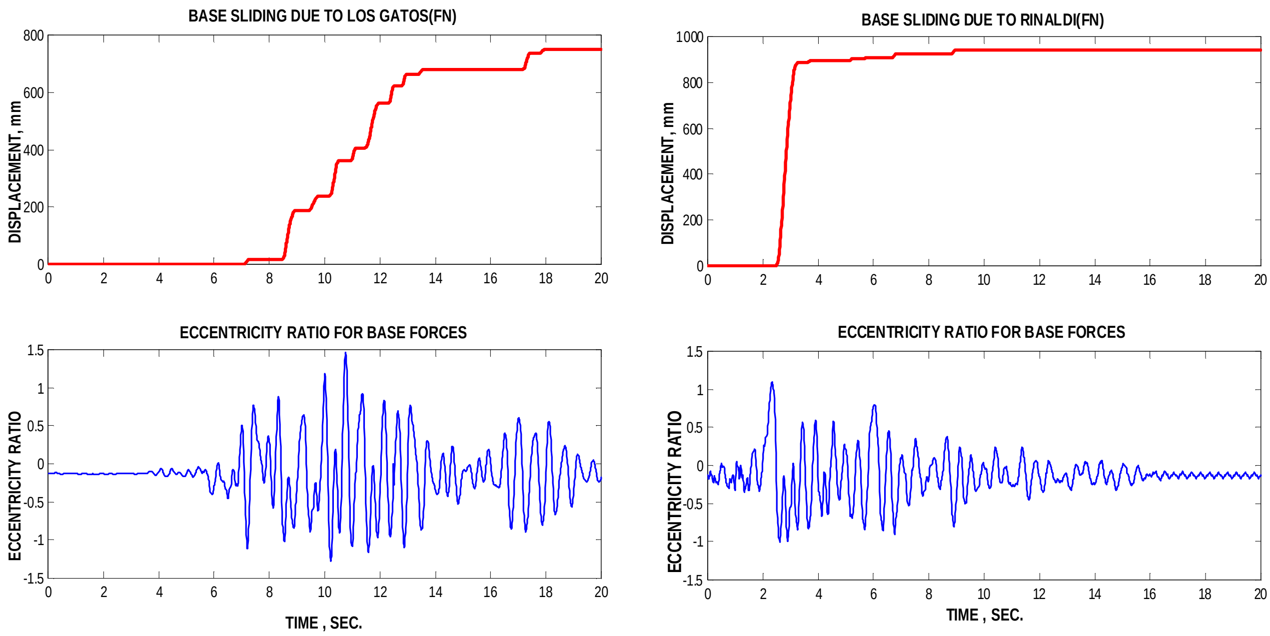

Figure 4: Base sliding and eccentricity ratio for Los Gatos and Rinaldi Receiving Station records.

Table 4: Principal stress, eccentricity ratio, and displacement of the Pine Flat dam without base sliding.

Ground Motion Record	Maximum Principal Stress (MPa)				Eccentricity Ratio		Displacement at the Crest (mm)	
	Upstream		Downstream					
	FN	FP	FN	FP	FN	FP	FN	FP
El Centro	4.94	6.87	4.97	5.88	0.63	1.09	38.2	71.5
Pacoima Dam	16.4	9.3	15.14	9.53	2.73	1.33	188.2	91.7
Los Gatos Presentation center	12.26	9.62	11.81	9.21	2.1	1.5	145	107.7
Erzincan, Turkey	6.49	8.14	7.89	7.1	1.05	1.33	76.8	89.4
Lucerne Valley	6.65	6.81	8.25	7.73	0.7	0.66	42.5	54.7
Pacoima Dam Downstream	8.07	6.01	7.47	5.08	1.24	0.71	88.4	48.2
Pacoima Dam Left Abutment	19.39	16.95	13.9	13.76	3.19	2.45	174.1	195.4
Rinaldi Receiving Station	17.5	7.87	12.67	10.45	2.88	1.2	171.2	88.4
Sylmar County Hospital	8.7	12.68	6.54	10.93	1.48	2.17	87.9	149

the case of Rinaldi Receiving Station record may be the high velocity of the record in the FN direction (178.4cm/s), the largest ground velocity has been recorded instrumentally so far. These amounts of sliding displacements are about .8, .6, and .56 percent of dam height, which are comparatively large values that do indicate unstable response. Base sliding displacement in such level will cause severe damages to keys, drainage systems, and grout curtains, and even may lead to loss of reservoir. The most interesting point is the ratio of FN and FP base sliding displacements. For example these ratios are 18.8, 11.8, and 5.8 for Rinaldi Receiving Station record, Pacoima dam (1971), and Los Gatos records, respectively. These large differences of base sliding displacement in the FN and FP directions indicate that more attention must be paid to the directional frequency contents, amplitude level and input energy of near-field ground motions. Eccentricity ratios for base forces in Table 3 are larger than one for most of the cases and even 1.8, and 1.67 for Pacoima dam left abutment record. The ratio larger than one shows the possibility of rocking of the dam along with sliding. However, under realistic conditions, sliding is the dominant motion mode, and the rocking motion will die out almost instantaneously, due to, first, the small height-to-width ratio and large size of the dam make rocking response small. Second, water pressure prevents the rocking response from building up when the dam rocks in the upstream direction around its heel. Third, large amount of rotational energy is lost in impact. Finally, downstream sliding of the dam (due to hydrostatic pressure and impact) further reduces the rocking motion [Chopra, 1991]. Figure 4 shows base sliding displacement and eccentricity ratio of base forces for Los Gatos Presentation Center, and Rinaldi Receiving Station records. As pointed out, to be able to compare the results and draw reasonable conclusions, the dam was also analyzed with base sliding prevented. The response results are listed in Table 4. Regarding the principal stresses they show very high sensitivity to base sliding in some cases. For example the stress in the FN direction on the upstream face of the dam reaches to 19.39 MPa., 17.5 MPa., and 16.4 MPa., for the case without base sliding, from 9.9 MPa., 8.51 MPa., 11.19 MPa., for the case with base sliding for Pacoima dam left abutment, Rinaldi Receiving Station, and Pacoima dam (1971) records, respectively. These are dramatic increases when we do not consider based sliding. In the downstream the stress varies from 8.75 MPa., to 12.67 Mpa., for Rinaldi Receiving Station in FN direction. In the FP direction for all of the records the stress variation remains very low compared to FN direction, with the only exception of Sylmar County Hospital record for which stress goes from 7.59 MPa. to 12.68 MPa. In most cases the stress in FP direction practically remains unchanged. Base sliding may be interpreted as an isolation and energy dissipation mechanism for dams. The response results for two cases of with and without base sliding indicate that, the base sliding displacements are large enough to reduce substantially the dam deformation and especially dam stresses. This is very clear in the FN direction, where dramatic changes in stress were observed. For example by comparing the energies due to base sliding, foundation displacement, dam deformation for the case of the dam with base sliding with that of without base sliding, it becomes clear that base sliding energy in some cases comprises about 70 percent of whole energy (Rinaldi record in FN direction). It is about 37 and 31 percent for Los Gatos and Pacoima dam (1971), respectively. Again in the most of the cases the largest amount of base sliding energy belongs to FN direction. The base sliding energy in the FP direction is very tinny amount and may be neglected very easily. Practically there is no base sliding displacement for the gravity dam case due to El Centro record. It is possible to decrease the amount of base sliding to an allowable level, e.g. 70mm, by using reinforcement steel bars of around 1.2 percent of interface area, or about 120 cm^2 steel per one square meter, for Rinaldi record. This case along with arch dam case imply that there are fundamental differences between near-field and far-field ground motions and underline the urgent need for properly addressing the near-field problems.

CONCLUSIONS

Near-field ground motions differ dramatically from their far-field counterparts, and such kind of ground motions must be treated in different ways, or even may require special processing to accurately represent their features.

It is important to select an appropriate suite of time histories not only based on instrumental parameters such as PA, PV, PD, IV, and ID, or ground motion parameters based on response spectra, but also the most reliable parameter, input energy of earthquake ground motion. Maximum incremental velocity (IV) and maximum incremental displacement (ID) seems to be better parameters for characterizing the damage potential of earthquakes in near-field region.

The directional frequency contents and amplitude level of near-field ground motions have fundamental consequences in earthquake response of dam structures. This implies that the dynamic response of dam structures will be influenced by their orientation relative to the ground motion and by their proximity to causative faults.

In the case of the arch dam for the most of the ground motions the increase in the maximum arch and cantilever stresses in the FN direction is about 100 percent. However, in some cases the maximum arch and cantilever stresses occur in FP direction. It seems the latter cases are exceptions. It may be concluded that the FN direction is the most critical direction regarding the stress level in most of near-field ground motions.

Stress level of arch dam is beyond the yield limit of the concrete commonly used in constructing the dams, and dam will crack under such ground motions, and this will make non-linear analysis of dams in highly seismic region indispensable.

For gravity dam stress level is very high, and this will lead to severe cracking of the dam basically in the neck region and interface of the dam and foundation rock, and even making dam unstable.

Base sliding displacements of gravity dam are dramatically large in FN direction, and may inflict severe damages to keys, drainage systems, and grout curtains or finally may lead to loss of reservoir.

In view of many assumptions made in the analyses performed here, the above conclusions should be regarded as preliminary, and this is strongly emphasized. Additional dam heights, configurations, and different suite of strong ground motions need to be examined. More research should be devoted to effects of large near-field earthquakes regarding duration of shaking and frequency contents, in addition to near-source effects.

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