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## INTERNATIONAL JOURNAL OF ADVANCED RESEARCH (IJAR)

Article DOI:10.21474/IJAR01/13588  
DOI URL: <http://dx.doi.org/10.21474/IJAR01/13588>



### RESEARCH ARTICLE

#### DYNAMIC RESPONSE AND STABILITY ANALYSES OF AKOSOMBO DAM USING NUMERICAL ANALYSIS

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#### Manuscript Info

##### Manuscript History

Received: 28 August 2021

Final Accepted: 30 September 2021

Published: October 2021

#### Abstract

Dams are very important in Ghana's economic development and environmental improvement. Although Ghana dams are seismically far from the active zone, accurately analysed dams should be evaluated since failure could severely impact the people in the flood environment and the region's economy on a large scale. This paper proposes a numerical procedure for the static, slope stability, and dynamic analysis of the Akosombo embankment dam. Nineteen horizontal acceleration time histories recorded data was used based on Maximum Design Earthquake (MDE), Maximum Credible Earthquake (MCE), Design Basis Earthquake (DBE) and Operating Basis Earthquake (OBE) data. The numerical results estimated showed that the Akosombo embankment dam is likely to experience moderate deformations during the different design earthquakes. The result also indicated that non-linear analysis capable of capturing dominant non-linear mechanisms could be used to assess the stability of embankment dams. The factor of safety (FS) calculated was greater than 1.5 for high reservoir, rapid drawdown condition and low reservoir condition whereas, the FS values were found to be 1.42 for slow drawdown condition.

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#### Introduction:-

Stability and crest displacements of embankment dams is a significant concern in every region. Seismic-induced deformations in a rockfill embankment dam may lead to overtopping and massive losses of built environments and people living along the dam paths. However, in recent years, the availability of seismic hazard maps and different design criteria have led to assessing the seismic safety of dams built without considering seismic-induced forces. Consequently, the adequately designed remedies have posed a challenging problem for geotechnical engineers. A standard manual on world register dams provided by the International Commission on Large Dams (ICOLD) has about 95 member countries, including Ghana, which defined dams with a measured height of more than 15 m or between 5 and 15 m whose reservoir capacities greater than 3 million m<sup>3</sup> as "large dams". Presently about 60,000 large dams have been registered (Wang et al., 2021), and 1,949 are in Africa. Skinner et al. (2009) report showed that West Africa has over 150 large dams constructed on their region's river.

Akosombo Dam is the largest dam in West Africa located on the Volta River in Ghana, with a wall height of 134 m (4th in Africa) and a capacity of 150 billion cubic metres (3rd in Africa). Akosombo dam is a rockfill embankment dam, about 82 km north of Accra city in the southeast of Ghana (Fig. 1). The dam has a crest length of 640 m, a crest width of 12.2 m, and a 114 m maximum structural height above the foundation level, completed in 1965. The

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reservoir is used for providing electricity for the host environment. The construction of embankment dams is more recognised globally with some advantages over other dams (Narita, 2000), i.e., the foundation does not require rigorous conditions. The embankment dams can be placed on alluvial deposits and pervious foundations. They can be constructed on the outskirts of the city area.

The Knowledge of different specialists (engineering, hydrology, geophysics, geology, and soil mechanics) is required for the construction of an earth-fill (embankment) dam that can provide safety and constant critical surveillance of dam reservoir and foundation during its lifetime (Ebrahimian, 2011). However, the highest responsibility belongs to the seismic designers. Dams' failure can lead to a severe loss of human and properties. The failures may be caused by structural deficiencies in the dam body due to poor design or construction, age of the dams, sliding of slope and instability, and overtopping (Shikhare et al., 2009).

However, a record shown considerable numbers of embankment dams have been damaged in different countries due to earthquakes (Gosschalk 1994). This call for a better method to enhance the stability of existing dams. The standard method used for the seismic design of earth dams was erroneous based on the assumption that dam bodies are fixed on a rigid foundation, thereby experiencing a uniform acceleration equal to the underlain-ground acceleration (Gazetas, 1985).

However, research has shown that the response of dams under earthquakes depends on the constructed materials, the geometry, and the base motion's nature. Therefore, many studies have been carried out to understand the seismic behaviour of earth and rockfill embankment dams. Newmark (1965) was the first to carry out the effects of earthquakes on dams and embankments. He used a sliding block method to evaluate the likely performance of the earth and the embankment dam. Lin and Whitman (1986) assess earthquake-induced permanent displacement using sliding blocks.

However, since the advent of more sophisticated and fast computer programmes, effective modelling, and testing, an increasing number of studies on earth (rockfill) dams using numerical methods such as finite element, finite difference, and boundary element methods with non-linear material models (Sengupta, 2010; Albano et al., (2012) Afiri and Gabi, 2018), performed seismic analysis on an existing bituminous faced rockfill dam in southern Italy using numerical analysis. The numerical simulation results compared to the small-scale centrifuge model of the dam. Selçuk and Terzi (2015) carried out a seismic response of the Ambar dam to a recorded earthquake.

This study investigated the Ambar earth dam's dynamic response and earthquake resistance in the southeast part of Turkey using the non-linear dynamic time-history of Bingol (2003) earthquake and modal analysis methods. The dam was analysed under static conditions and dynamic analysis. The PGA responses caused by maximum design earthquake (MDE) applied to the dam body, impervious core, and foundation.

They showed that the Ambar dam might likely experience moderate deformations during the design earthquake but stable after the earthquake. Afiri and Gabi (2018); Afiri et al. (2018) evaluated the slope stability analysis of Souk Tleta dam in north Algeria using the shear reduction method. They showed that large displacements occurred in the dam body and foundation at the end of construction and lowered during the reservoir filling. Also, the result showed that the factor of safety in different conditions decreases when the reservoir water level increases. Finally, Jin and Chi (2019) conducted seismic fragility analysis on high earth-rockfill dams using various ground motion records. They used a program called DYNE3WAC in combination with the Pastor–Zienkiewicz–Chan model and Biot dynamic consolidation theory to determine the vertical deformation of the dams.

This paper investigates the dynamic response and earthquake resistance of the Akosombo embankment dam located in Asuogyaman Province, Ghana, using non-linear dynamic time histories. The dam was first analysed under static conditions and then performed acceleration-time histories of 19 earthquakes.

The PGA responses caused by maximum design earthquake (MDE), Maximum Credible Earthquake (MCE), Design Basis Earthquake (DBE) and Operating Basis Earthquake (OBE) were applied to the dam body, impervious core, and foundation, and the factor of safety was calculated by drawdown reduction method. The drawdown reduction method was used by Athani et al. (2015) to determine seepage and stability analyses of earth dam using PLAXIS 3D Finite element method.

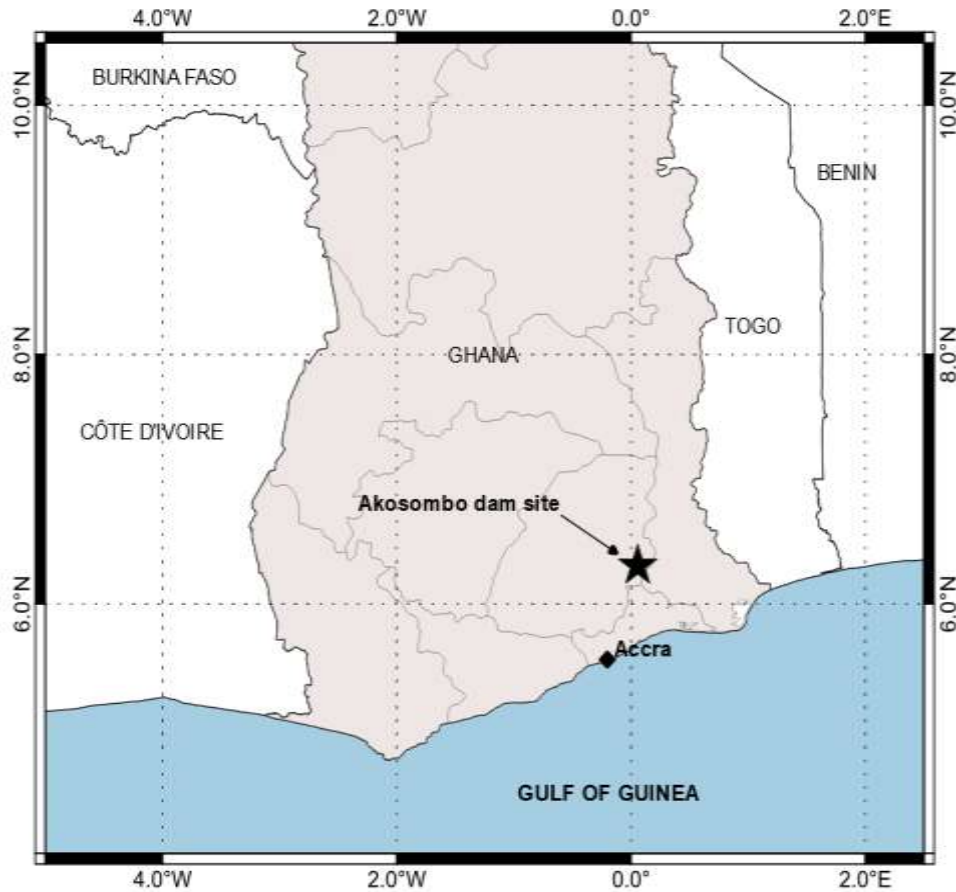


Fig.1:- Location of Akosombo dam.

### Geology of the Dam Site

The geology of the dam bodies' foundation, lithology, stratigraphy, and soil parameters need to be assessed during analysis. A high degree of judgment is required to evaluate the soil characteristics through comprehensive field observation and laboratory tests (Ozkan et al. 1996; Selçuk and Terzi, 2015). To construct the cofferdams for Akosombo Dam, the entire bedrock surface was stripped of a thick deposit of sand alluvium using machine at a depth of 200 feet below the water surface.

The upstream and downstream cofferdams formed by end dumping methods at depths varied from 30.48 m in the upstream to 60.96 m at the downstream. The clay core of Akosombo dam was compacted at moisture content slightly wet to 2 percent wet of optimum and compacted rockfill was sluiced at a water to cement ratio of 0.5 to 1.0. The field density of Akosombo core was put at 0.80 frequency of test at  $764.555 \text{ m}^3$  (Wilson and Squier 1984). The shell is made of compacted or dumped rockfill, while filter materials are of fine and coarse materials.

### Geometry of the Dam and soil Properties

When assessing the safety threshold value of any structure, the most critical factor is the deformation that occurs under seismic effects at which the accelerations change at various locations. Therefore, an accurately designed dam should detect the accelerations at the critical area of the dam under investigation (Selçuk and Terzi, 2015). The schematic cross-section of the Akosombo dam and the foundation layer used in this paper is shown in Fig. 2.

The geometrical features are provided in Table 1. The geotechnical parameters of the slope layers are determined using engineering experience obtained from the literature. The necessary parameters for the slope stability analysis and design are presented in Table 2. Only the geotechnical parameters and properties that concern the dam's slope stability analysis are specified in this study. The permeability ( $k$ ) of dam materials used the standard values were taken from the engineering perspective.

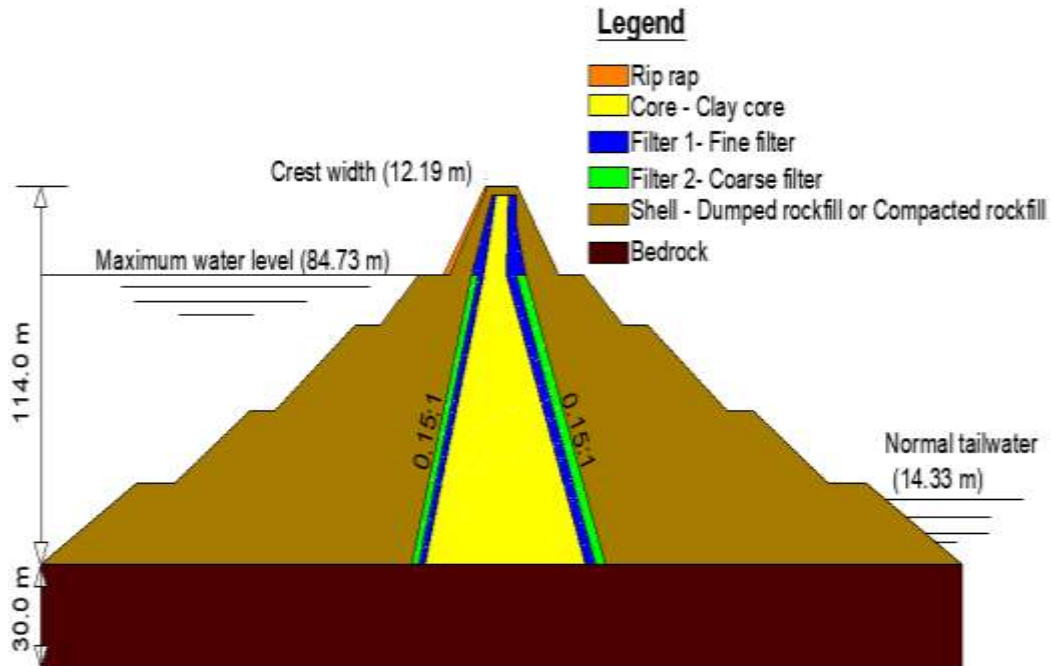


Fig. 2:- Geometry of the cross section of the Akosombo Dam.

Table 1:- Geometrical properties of Akosombo dam.

Parameter	Value
Dam's height above ground level	114 m
Crest elevation	88.39 m
Dam width	366.0 m
Dam crest length	660.0 m
Dam crest width	12.20 m
Maximum water level	84.73 m
Tail water level	14.33 m
Area of watershed	8,502 km <sup>2</sup>
Storage capacity	153,000 M m <sup>3</sup>

Table 2:- Parameters used for slope stability of Akosombo dam.

Material type	Unit weight $\gamma_h$ (kN/m <sup>3</sup> )	Saturated Unit weight $\gamma_{sat}$ (kN/m <sup>3</sup> )	Internal angle of friction $\phi$ (°)	Cohesion of soil $c$ (kPa)	Poisson's ratio $\nu$	Young modulus $E$ (MPa)	Permeability $k$ (m/day)
Core clay	18	19	20	10	0.25	25	0.0000864
Filter 1	19	20	30	0	0.23	90	0.00864
Filter 2	17	19	30	0	0.27	80	0.00864
Shell	18	21	45	15	0.30	80	0.0864
Rip rap	20	21	30	0	0.24	60	0.000864
Bedrock	21	22	29	20	0.17	150	0.000001

### Seismic activity in and around the dam site

Numerical estimation of permanent displacement of a proposed or existing earth/rockfill embankment dams requires an enormous task, mainly when information on field data is lacking. The seismic hazard assessment performed by probabilistic seismic hazard analysis (Cornell, 1968, and McGuire, 1995) considers all possible earthquake scenarios affecting the site.

The seismic hazards estimated are represented by peak ground acceleration. Therefore, to assess the dynamic response of the Akosombo Dam, it is necessary to consider the seismic risk in and around the region.

Two major active faults governed the neotectonics of the southern Ghana. They are: (1) the Akwapim fault, which is the main structural feature in the basin, (2) the Coastal boundary fault, which is a normal fault along the coastal line that strikes approximately north  $60^{\circ}$  E- $70^{\circ}$  E, at about 5 km from the coast.

No accounts of damage due to earthquakes have been observed within the dam area. However, Ghana earthquakes of 1964 and 1969 were felt near the Akosombo dam and was subjected to induced seismicity (Musson 2014). Accra's earthquakes are mainly attributed to the reactivation of faults in the Romanche fracture zone, specifically the Coastal Boundary fault and Akwapim fault (Kutu, 2013; Musson, 2014; Ahulu et al., 2018).

The Coastal Boundary fault could be active during Jurassic times and is still tectonically active (Ahulu et al., 2018). The Akwapim Fault Zone comprises a system of faults trending north-easterly from just west of Accra, along an ancient line of thrust boundaries in the Dahomeyan belt.

Neotectonics normal faults along Akwapim zone could mean that tectonic movement may be active, making faults from SW of the Akwapim likely active (Amponsah et al., 2012; Ahulu et al., 2018).

According to the available historical sources, about 35 earthquakes had occurred between 1818 and 2018 for  $M_w \geq 4$ . Based on this data, Ghana earthquake zoning Map was proposed (Fig. 3).

The map divides Southern Ghana into 4 risk zones, 70% of the total population of Ghana and 3 existing large dams currently in operations are in the proposed hazard zones. Based on the Earthquake Zoning Map; Fig. 3 shows that the dam site is in Moderate Hazard Zone Area.

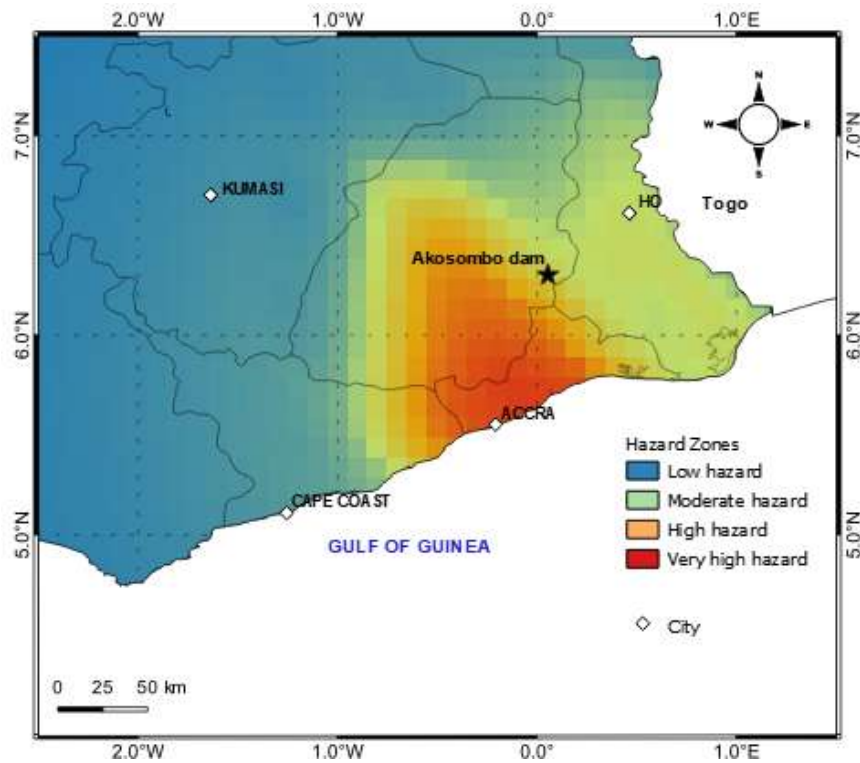


Fig. 3:- Proposed seismic hazard map for 475 years return period.

### Numerical Analysis

The dynamic performance of dams using numerical analysis to estimate the crest settlement requires a more sophisticated approach (Kan and Taiebat, 2011). For the seismic design of the dams and its appurtenant structures, the following design earthquakes were considered as recommended by Wieland (2012).

1. Operating Basis Earthquake (OBE): The OBE design is used where earthquake damage to a dam project is limited, where the mean value of ground motion is determined from a probabilistic seismic hazard analysis

- (PSHA). According to ICOLD, 2010, the OBE should be designed at an average return period of 145 years (i.e., 50% probability of exceedance in 100 years).
2. Design Basis Earthquake (DBE): The DBE is designed for a return period of 475 years for dam and its appurtenant structures. The DBE mean value of ground motion parameters is estimated based on a PSHA. (i.e., the return period of the DBE usually determined based on the earthquake codes and regulations for buildings and bridges in any project region.). The ground motions for a 2% probability of exceedance within 50 years, which is equivalent to 2475 years of return period is also considered for the design earthquake in this study.
  3. Maximum Credible Earthquake (MCE): The earthquake selected for dam structure whose return period is 2475 years. The earthquake that can reasonably expected to occur along a recognised fault or supported by known geologic and seismologic data. A hypothetical probabilistic earthquake is considered since the event is random, and its epicentral distance is determined mathematically by relationships of recurrence and magnitude for this study dam.
  4. Maximum Design Earthquake (MDE): The MDE is designed for a return period of 10,000 for a large dam, where the ground motion parameters are estimated based on PSHA. The average mean values of the ground motion are also taken for the design.

Within all the frameworks of these assessments for the Akosombo seismic design considering the OBE, DBE, MCE and MDE, the peak ground acceleration earthquake coefficient was estimated as 0.03g, 0.08g, 0.21g and 0.39g respectively for 145, 475, 2475 and 10,000 years return periods. The input earthquake motions were compiled from a real accelerograms (Table 3) from online PEER Ground Motion Database - PEER Center (berkeley.edu), Strong motion virtual data Center (VDC) [www.strongmotioncenter.org](http://www.strongmotioncenter.org), and European Strong-Motion (ESM) Database, <http://isesd.hi.is/>. The real accelerograms (Fig. 4) processed by Seism-Signal 2021, were used for the dynamic analysis using Plaxis 2D. The total displacement of the dam crest was chosen as Engineering Demand parameter (EDP). The response spectra estimated from the hard rock site conditions were compared with the 19 response spectra for the different return periods (Fig. 5).

#### Dynamic Stability Analysis and Drawdown Conditions

PLAXIS 2D finite element software used Linear-elastic-perfectly plastic Mohr-Coulomb model for all layers of dam and foundation materials. The slope was modelled in the input module of PLAXIS, based on 15-nodded elements in a plane strain model. Drawdown conditions of stability of the reservoir were examined using a fully coupled flow-deformation analysis with time-dependent pore pressure. A fine mesh element of 2,009 and element nodes of 16,435 were generated in the simulation of slope stability analysis to estimate the least possible factor of safety (FOS). Fig. 6 shows the 2D view for the finite element mesh of Akosombo dam and its foundation.

Drawdown conditions for stability of the reservoir were examined under the following conditions, as listed below:

1. Steady state condition at High reservoir level of the Akosombo dam
2. Rapid drawdown in 5 days duration when the maximum water level is at 84.73 m
3. Slow drawdown in 30 days duration at water level of 84.73 m
4. Low water low at 34.73 m

**Table 3:-** Properties of 19 real earthquakes used.

Earthquake	Record station	M <sub>w</sub>	D <sub>5-95</sub>	a <sub>max</sub>	T	I <sub>a</sub>	EDA
Maximum Design Earthquake (MDE) 10,000 Years							
Friuli (Italy), 1976	Tolmezzo (000)	6.4	4.24	0.35	36.32	0.780	0.329
Imperial Valley (USA), 1979	USGS Station 5115	6.5	8.92	0.29	39.48	1.265	0.333
Kobe (Japan), 1995	Kakogawa (CUE90)	6.9	12.86	0.34	40.90	1.687	0.337
Loma Prieta (USA), 1989	CDMG (47381)	6.9	11.37	0.37	39.90	1.347	0.352
Duzce (Turkey), 1999	IRIGN (498)	7.1	12.26	0.39	35.00	2.196	0.391
Maximum Credible Earthquake (MCE) 2475 Years							
Kocaeli (Gebze Turkey), 1999	Yarimca (KOERI330)	7.4	15.62	0.22	34.96	1.318	0.327
Chi-Chi (Taiwan), 1999	TCU045	7.6	11.94	0.18	52.78	0.353	0.222
Hollister (USA), 1961	USGS Station 1028	5.9	16.53	0.19	39.93	0.257	0.194
Trinidad (USA), 1983	CDMG Station 1498	5.7	7.800	0.19	21.40	0.168	0.175
Design Basis Earthquake (DBE) 475 Years							
Manjil (Iran), 1990	Tonekabun S58E	7.3	18.87	0.10	35.49	0.409	0.100

Spitak (Armenia), 1988	Gukasian 90	6.7	7.480	0.17	20.01	0.299	0.171
Nahanni (Canada), 1985	Site 3, 360	6.9	5.915	0.15	9.545	0.229	0.150
Oroville-01 (USA), 1975	Oroville Station 37	5.9	3.415	0.08	12.19	0.032	0.082
Irpinia (Italy), 1980	Rionero In Vulture 0	6.9	24.55	0.09	37.16	0.349	0.090
Operating Basis Earthquake (OBE) 145 YEARS							
San Fernando (USA), 1971	Via Tejon PV 65	6.6	51.78	0.030	70.185	0.026	0.024
Morgan-Hill (USA), 1984	Apeel 1E-Hayward 0	6.2	34.25	0.027	59.98	0.024	0.040
Northern-Calif (USA), 1960	Ferndale City Hall 224	5.7	24.96	0.047	82.29	0.036	0.074
Helena-Montana (USA), 1997	Helena Fed. Building	6.0	0.555	0.034	21.09	0.0019	0.018
Umbria Marche (Italy), 1997	Aquilpark-Citta 90	5.7	33.19	0.005	99.835	0.0011	0.005

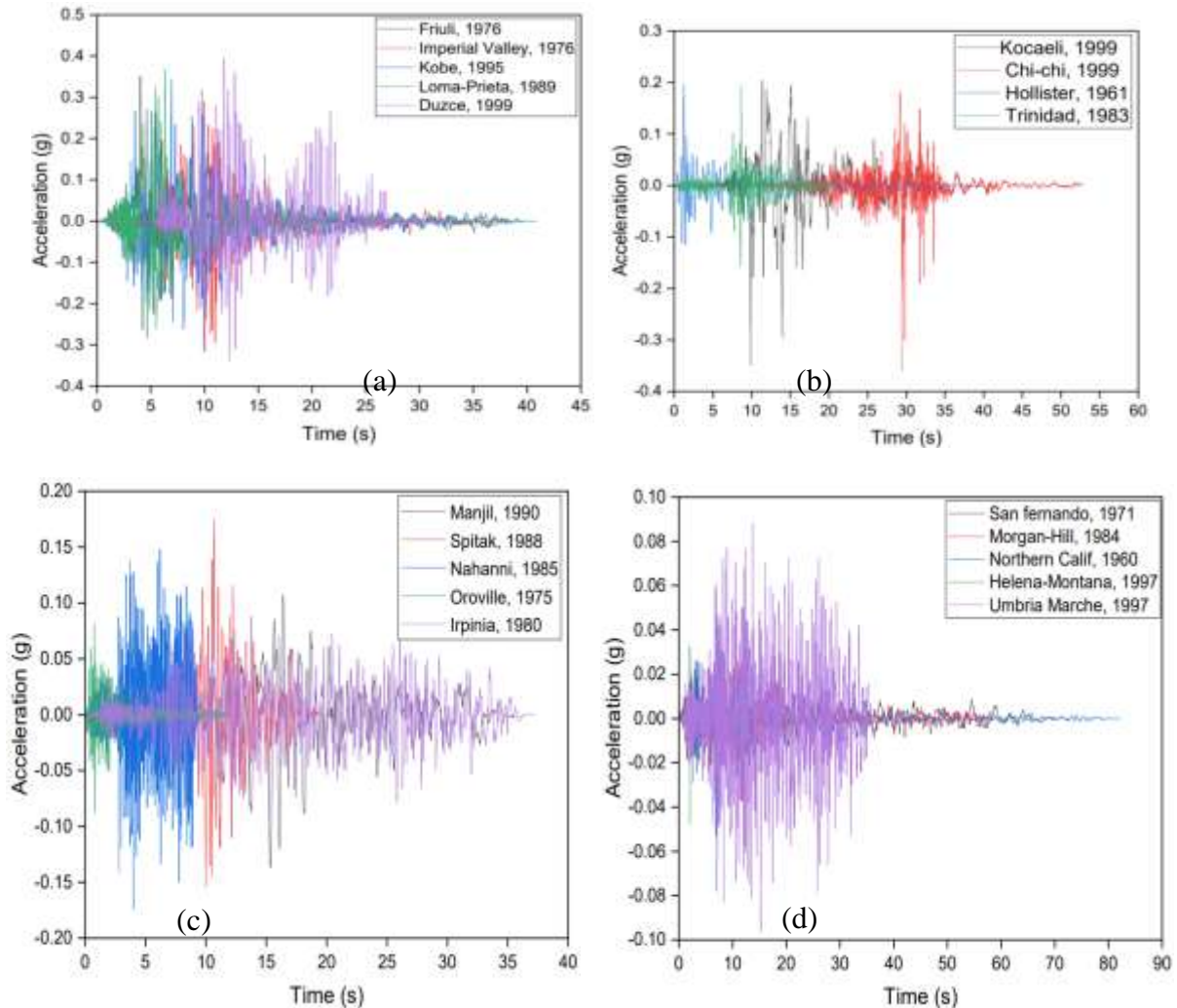


Fig. 4:-Horizontal acceleration-time history input data of nineteen earthquakes representing (a) 145-year (b) 475-year (c) 2475-year, (d) 10,000-year.

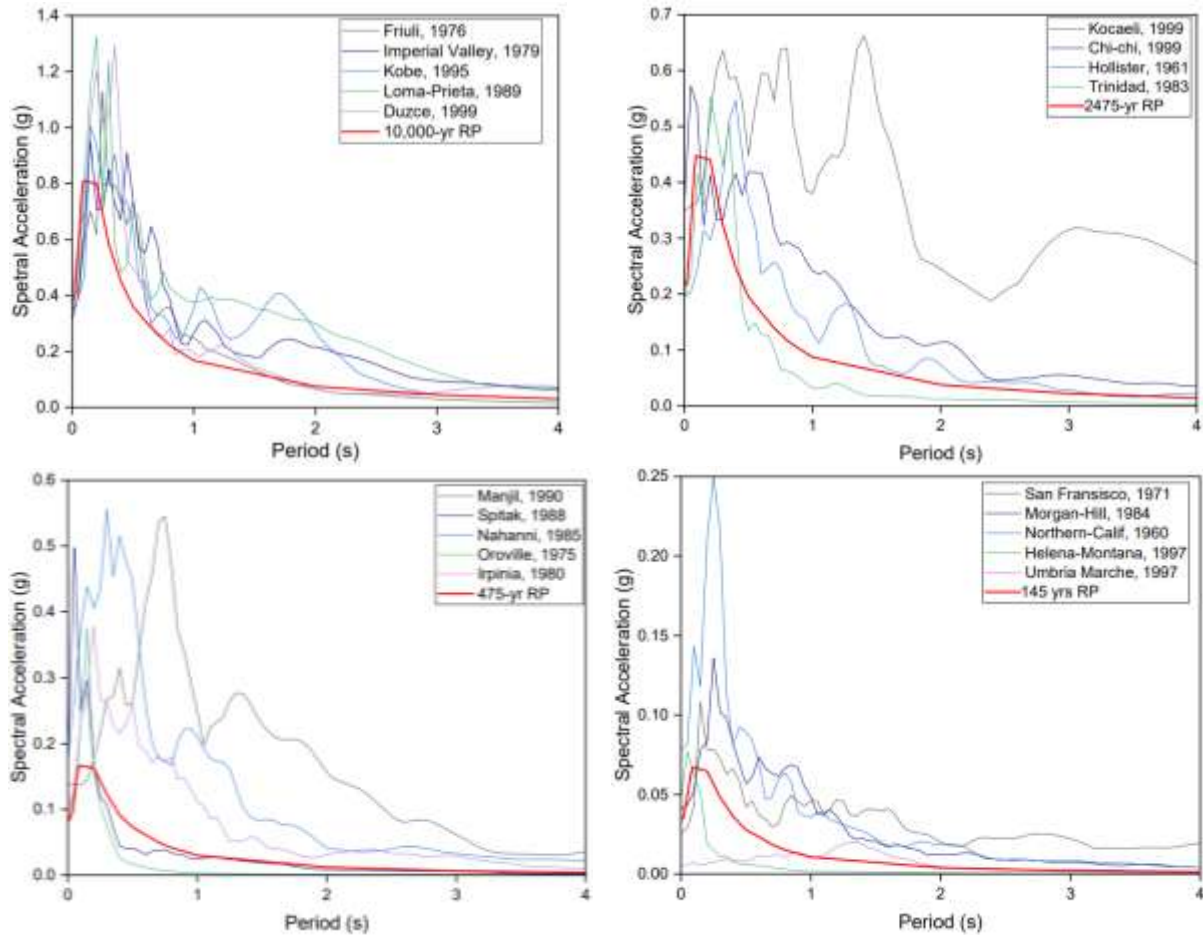


Fig. 5:-Comparison of acceleration response spectra of all the earthquakes and the site response spectrum for different return periods.

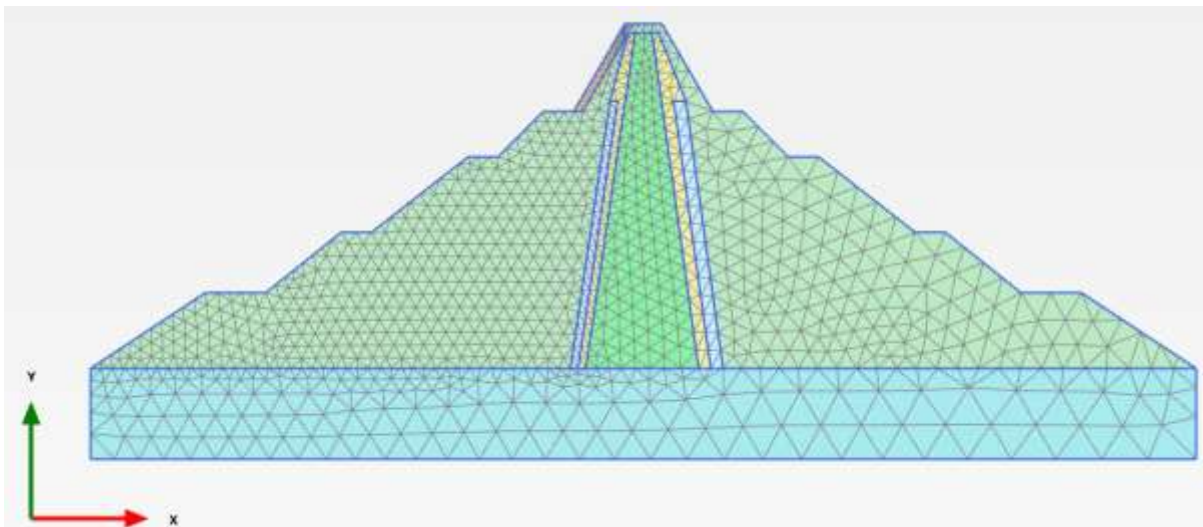


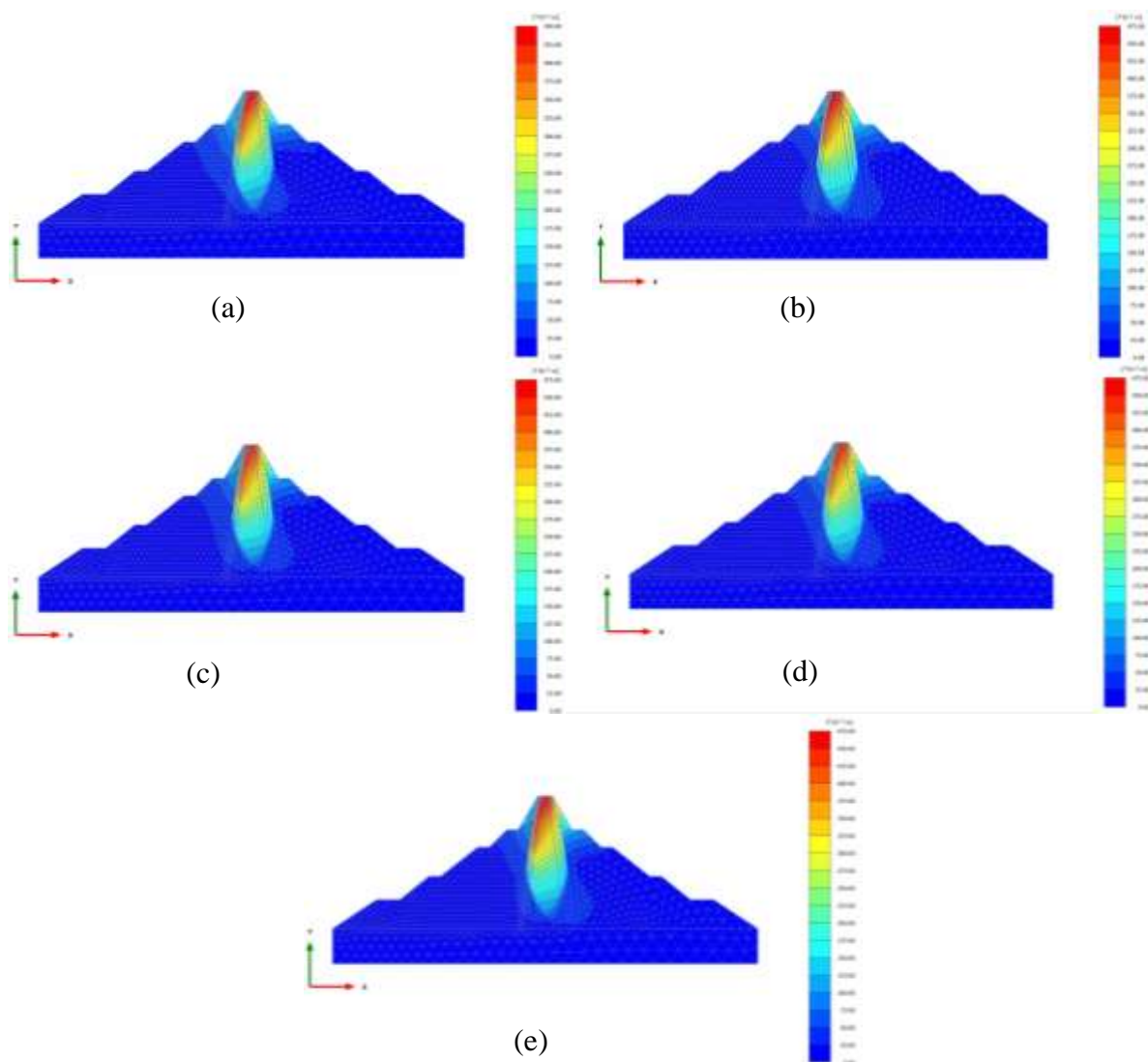
Fig. 6:- Connectivity plot of finite element mesh.

**Dynamic Analysis Results**

Dynamic deformation analysis was considered incremental stresses as a driving force for permanent deformation based on stress redistribution. According to Rampello et al. (2009), permanent displacements have resulted from

plastic strains that accumulated during the earthquake because of progressive plastic loading, which are influenced by the duration of the strong motion. In the dynamic evaluations, the crest behaviour was selected as the parameter to represent earthquake-related deformations because it is the most frequently mentioned quantified measurement of damage presented in dam studies. The amount of crest settlement is related primarily to two factors: the peak ground acceleration at the dam site and the magnitude of the causative earthquake (Selçuk and Terzi, 2015). The computed total displacements along the dam body for MDE, DBE, MCE and OBE values are shown in Fig. (7-10).

As seen in Fig. (7-10), displacements increased with the increased height of the dam. When the dam section undergoes strong base excitation, a slump usually take place rather than failure along a discrete surface. Based on this study, permanent deformations are concentrated in the dam crest. In general, according to the literature, when earthquake shaking occurred in a dam site, the settlements at the crest of an embankment dam should not exceed 1 or 2% of the dam height. The dynamic analysis using nineteen values of PGA showed that the instability of the Akosombo dam took place along the crest where the maximum settlement occurred in the core of the dam. From the dynamic analysis, the analyses show the effect of deformation at the dam body. The estimated total displacements and displacement in horizontal directions of Akosombo dam are listed in Table 4.



**Fig. 7:-** Earthquake induced deformation behaviour of Akosombo dam (a) using Friuli earthquake (b) using Imperial Valley earthquake (c) Kobe earthquake, (d) Loma-Prieta earthquake and (e) Duzce earthquake.

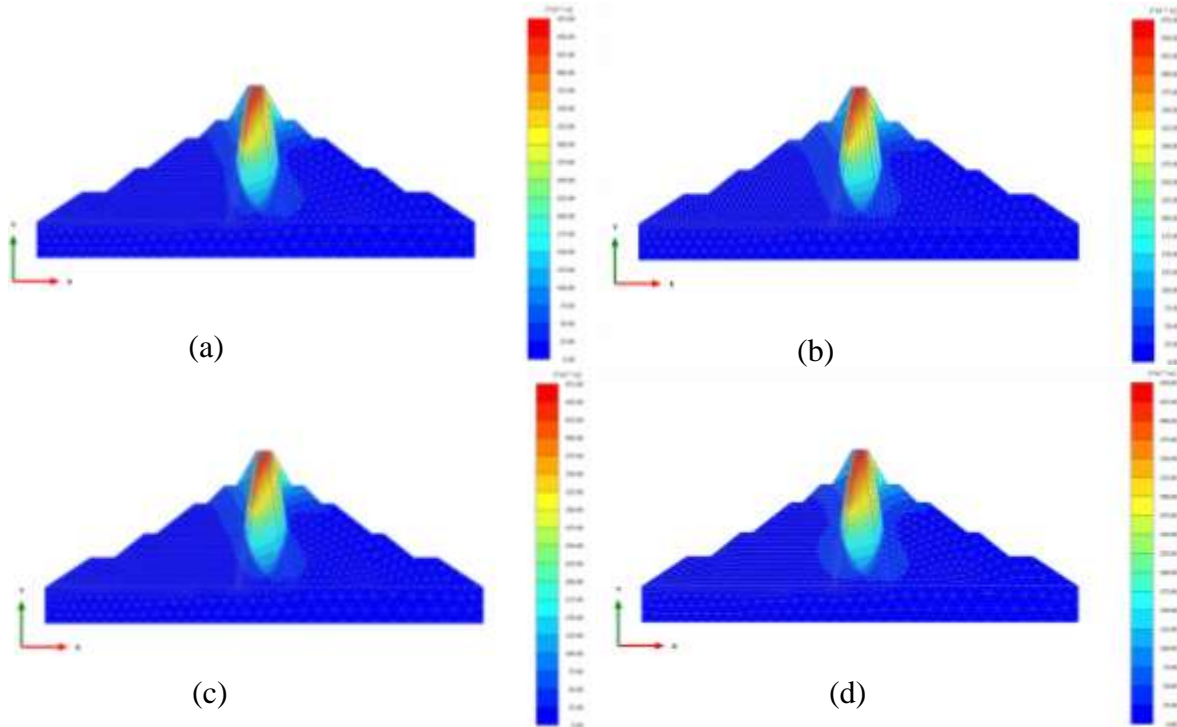


Fig. 8:- Earthquake induced deformation behaviour of Akosombo dam (a) Kocaeli earthquake (b) Chi-Chi earthquake, and (c) Hollister earthquake, and (d) Trinidad earthquake.

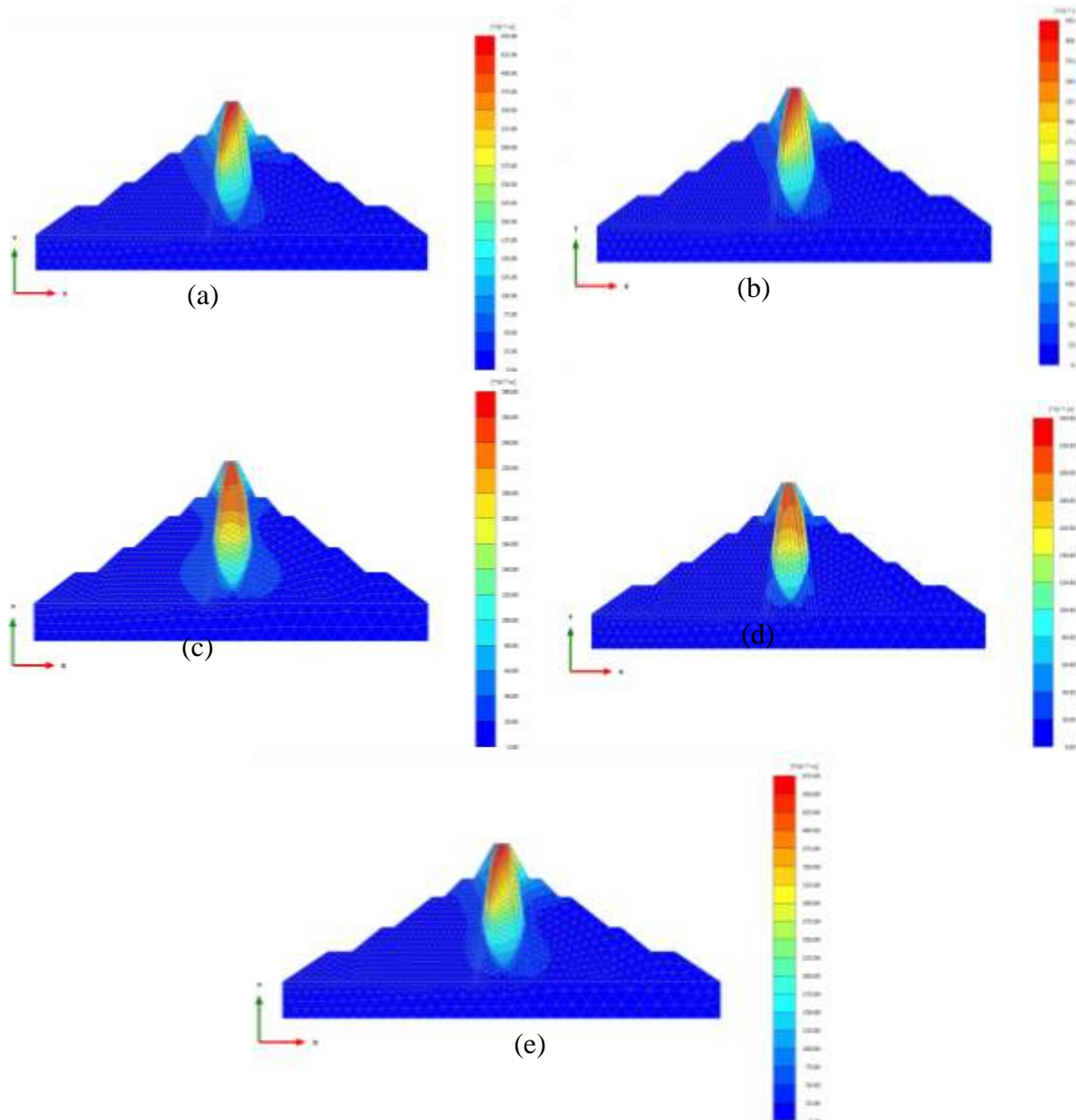
Table 4:- Computed deformations from the numerical analysis of Akosombo embankment dam.

Earthquake	Dam body (U) m	Dam body (U <sub>x</sub> )m
Maximum Design Earthquake (MDE) 10,000 Years		
Friuli (Italy), 1976	0.4429	0.03218
Imperial Valley (USA), 1979	0.4704	0.03180
Kobe (Japan), 1995	0.4526	0.02986
Loma Prieta (USA), 1989	0.4530	0.03218
Duzce (Turkey), 1999	0.4648	0.03373
Maximum Credible Earthquake (MCE) 2475 Years		
Kocaeli (Gebze Turkey), 1999	0.4569	0.03330
Chi-Chi (Taiwan), 1999	0.4597	0.02914
Hollister (USA), 1961	0.4511	0.03212
Trinidad (USA), 1983	0.4313	0.02839
Design Basis Earthquake (DBE) 475 Years		
Manjil (Iran), 1990	0.4434	0.03191
Spitak (Armenia), 1988	0.4072	0.02984
Nahanni (Canada), 1985	0.2607	0.06174
Oroville-01 (USA), 1975	0.2216	0.06147
Irpinia (Italy), 1980	0.4604	0.03029
Operation Basis Earthquake (OBE) 145 YEARS		
San Fernando (USA), 1971	0.4457	0.02758
Morgan-Hill (USA), 1984	0.4482	0.02941
Northern-Calif (USA), 1960	0.4503	0.02762
Helena-Montana (USA), 1997	0.2216	0.06147
Umbria Marche (Italy), 1997	0.4604	0.03029

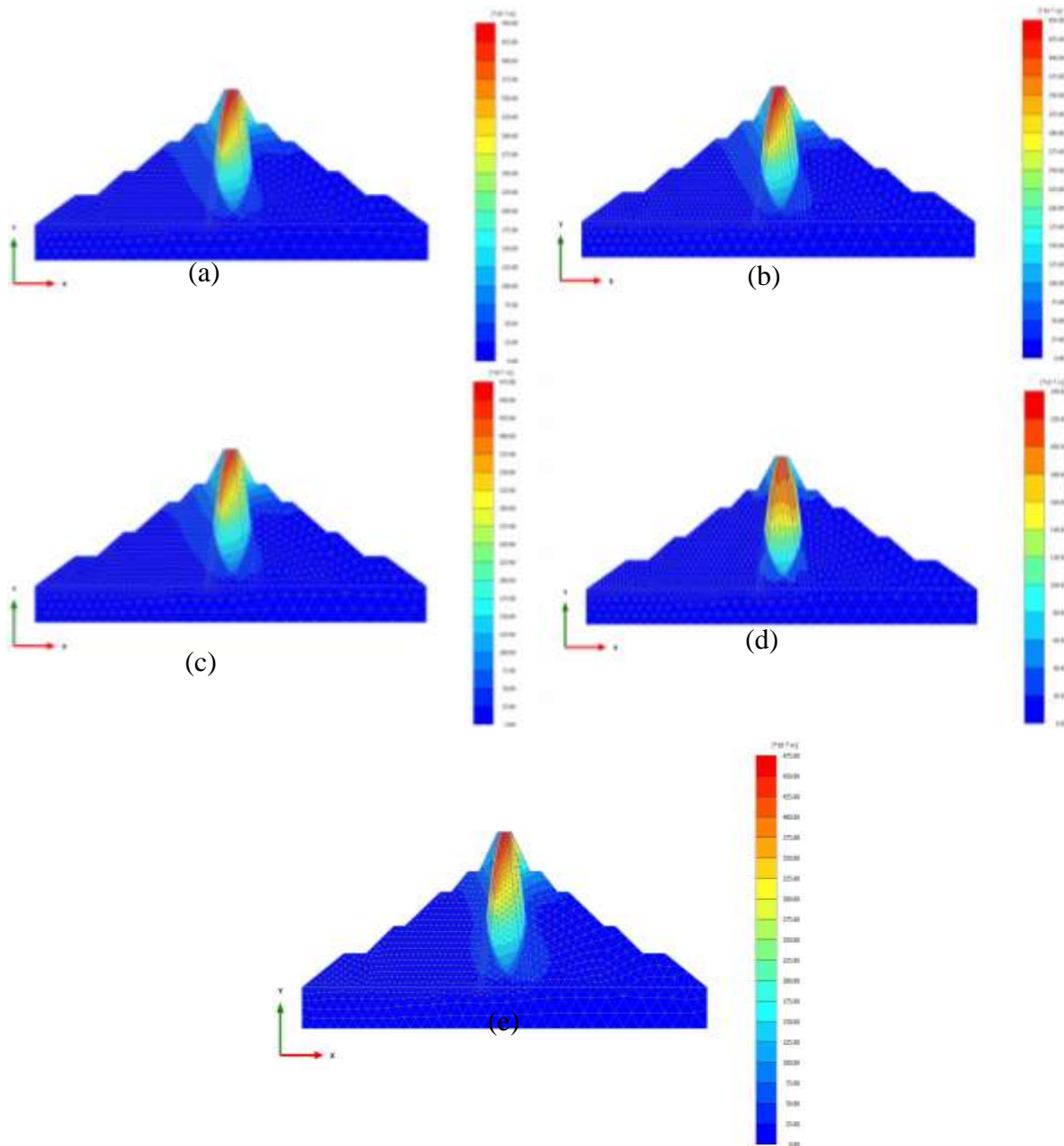
The effects of different 19 earthquakes (Table 3) on the total displacements at the crest of Akosombo dam with different horizontal peak ground accelerations are shown in Fig. 11.

The peak ground acceleration varied with the displacement. The total displacements and peak ground acceleration increase at the dam crest. The horizontal peak ground accelerations and numerical calculated horizontal displacements are shown in Fig. 12. It can be noted that in (Fig.11-12), the maximum peak ground acceleration does not lead to an increase in permanent displacement in the dam body.

The total deformation is estimated larger (0.47 m) when the horizontal peak acceleration is 0.29 g for the Imperial Valley earthquake. Similarly, three earthquakes (Nahanni, Oroville, and Helena-Montana) are responsible for the large horizontal displacements in the numerical estimation of the Akosombo dam with the PGA values of 0.15 g, 0.08 g, and 0.034 g, respectively.



**Figure 9:-**Earthquake induced deformation behaviour of Akosombo dam (a) using Manji earthquake (b) using Spitak earthquake (c) Nahanni earthquake, (d) Oroville earthquake and (e) Irpinia earthquake.



**Figure 10:-**Earthquake induced deformation behaviour of Akosombo dam (a) using San Fernando earthquake (b) using Morgan-Hill earthquake (c) Northern-Calif earthquake, (d) Helena-Montana earthquake and (e) Umbria Marche earthquake.

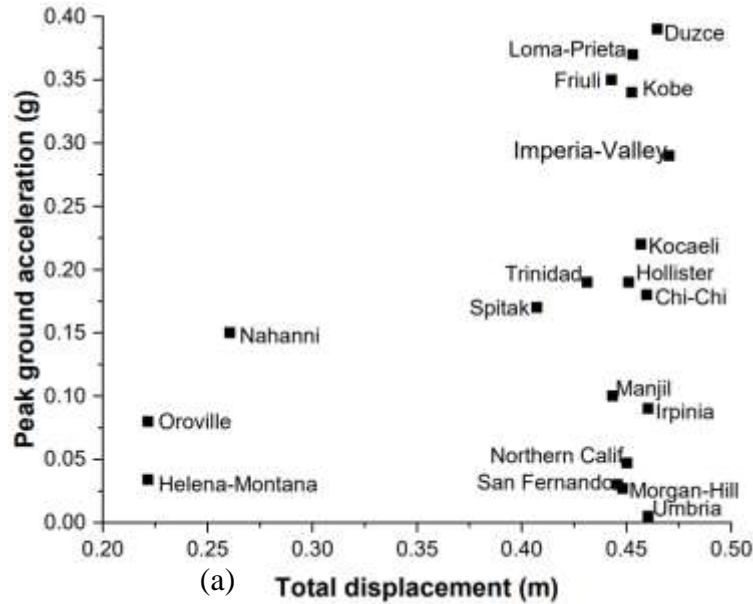


Figure 11:- Peak ground acceleration at the Dam crest Versus Permanent total Displacements.

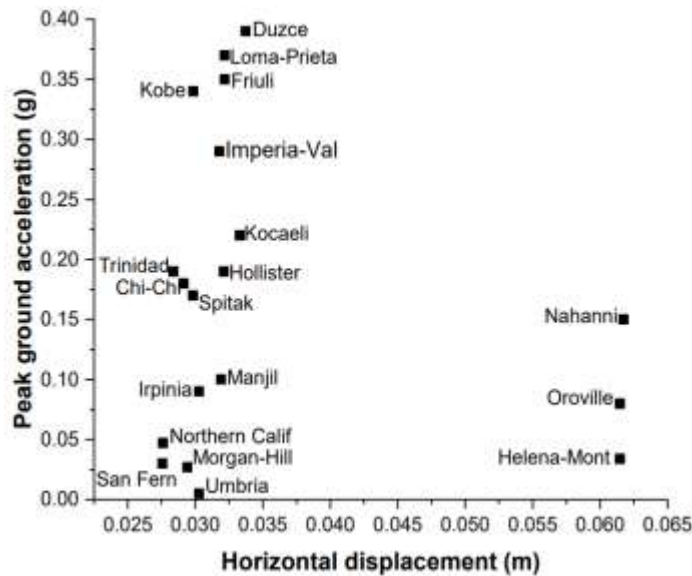


Figure 12:- Peak ground acceleration at the dam crest versus permanent displacements (a) Total displacement (b) Vertical displacement.

**Slope Stability and Results**

The safety factor Akosombo dam was assessed for the dam cross section of the greater height and computed using the ‘φ - c reduction’ procedure. For each phase condition, appropriate slope stability analysis is computed based on shear strength reduction method using FEM Plaxis 2D.

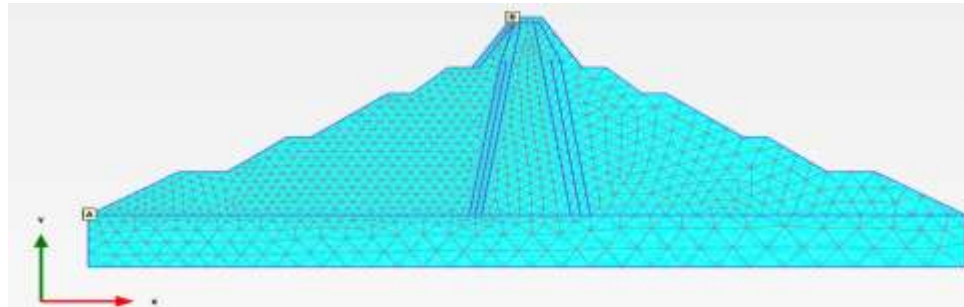
The study computed the factor of safety as the ratio of the available shear strength to the strength at failure by summing up the incremental multiplier (M<sub>sf</sub>) as expressed by:

$$FS = \frac{\text{available strength}}{\text{shear strength at failure}} = \text{value of } \sum M_{sf} \text{ at failure}$$

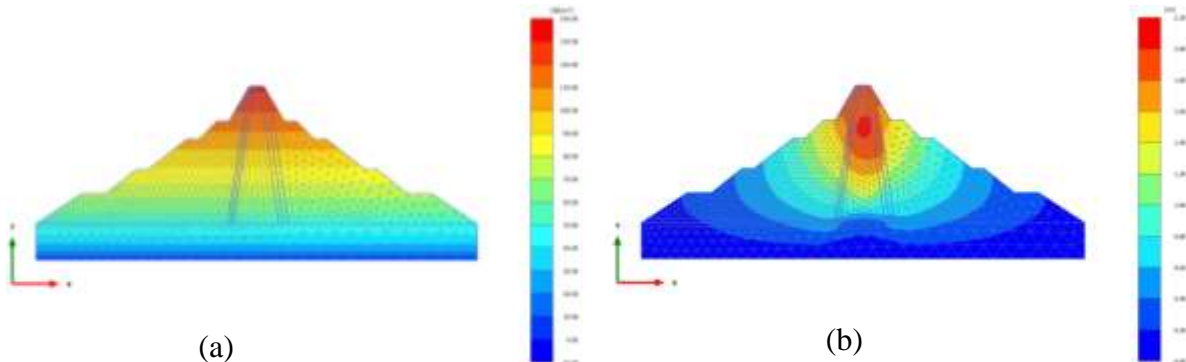
The acceptable minimum value for safety for end of construction and multistage loading is around 1.3 (Afiri and Gabi, 2018). According to USBR (2019), if a static factor of safety against slope instability is greater than 1.5, it is regarded as less likely to damage deformation.

**Steady State Condition at High Reservoir**

In this first stage, stability analysis was performed for the slopes of the dam for two dam points, as shown in Fig. 13, to optimize the volume of the dam body and materials. Gravity loading was used to calculate initial stresses and initial pressure of the dam under normal working condition. The slope analysis results under the steady state of high reservoir give a maximum safety factor of 1.72. For this study, the results of the finite element analyses are illustrated in Fig.14 (a-b). The dam body undergoes a maximum displacement of 2.2m due to the dead load of dam, and the settlement of different zones and foundation of the structure. The calculation of the displacements during this step is necessary because the imposed loads are very high which cause instability and even to a slope failure.



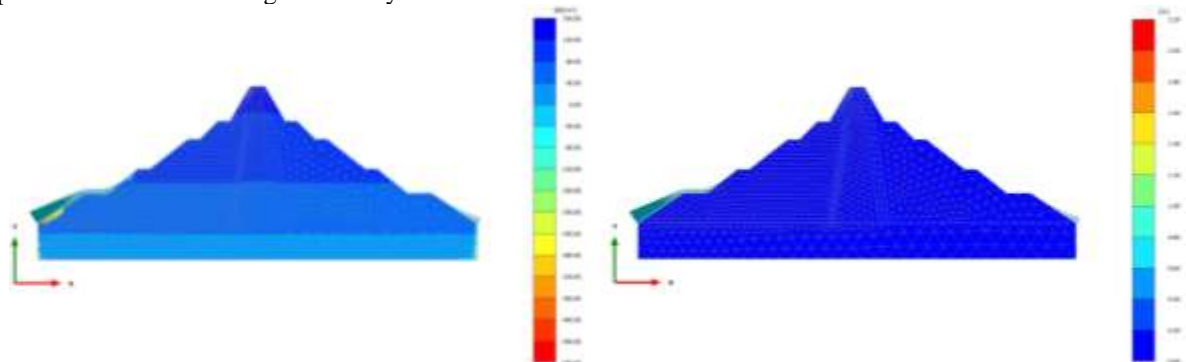
**Fig.13:-** Location of point A and B for factors of safety calculations.



**Figure 14:-** (a) Pore pressure distribution, ( $p_{active}$ ), for high reservoir (b) Total displacement ( $|U|$ ) for high reservoir.

**Rapid Drawdown Under the Maximum Water Level**

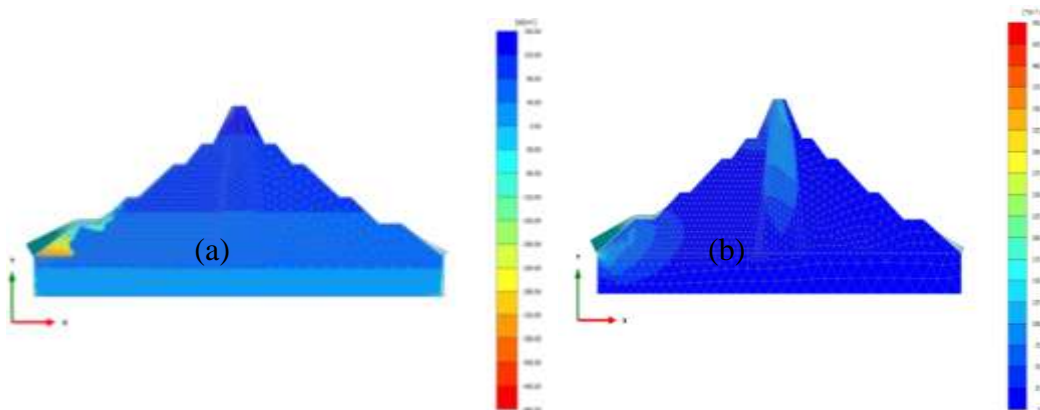
Rapid drawdown condition was used to compute the stability of the dam when the reservoir water level was at the maximum of 84.73 m. The water pressure distribution in the dam is calculated using fully coupled flow-deformation analysis by assigning 5 days value for the Time interval parameter with a reduced head water of 50 m. The computed groundwater for the pore pressure distribution after the rapid drawdown of the reservoir as well the displacement at the dam maximum water level is illustrated in Fig. 15 (a-b). The slope analysis results under the rapid drawdown condition give a safety factor of 1.75.



**Fig. 15:-** (a) Pore pressure distribution, ( $p_{active}$ ), for Rapid drawdown (b) Total displacement ( $|U|$ ) for Rapid drawdown.

### Slow Drawdown Under the Maximum Water Level

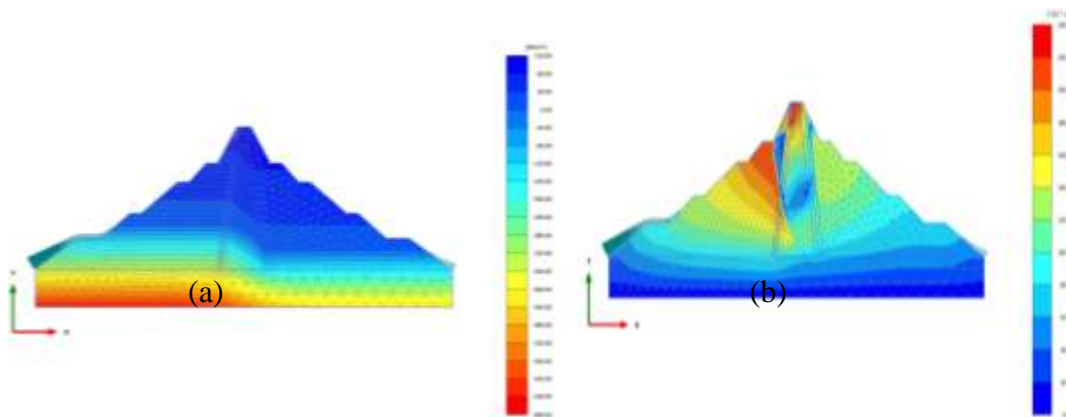
Slow drawdown condition was used to determine the stability of the dam when the reservoir water level was at 84.73 m. The water pressure distribution in the dam is also calculated by fully coupled flow-deformation analysis by assigning 30 days value for the Time interval parameter when the head water reduced to 50 m. When the reservoir filling at the maximum level, the lowest factor of safety of 1.42 is observed. The groundwater computed for the pore pressure distribution under the slow drawdown condition of the reservoir and the deformation estimated are shown in Fig.16.



**Fig. 16:-** (a) Pore pressure distribution, ( $p_{active}$ ), for Slow drawdown (b) Total displacement ( $|U|$ ) for Slow.

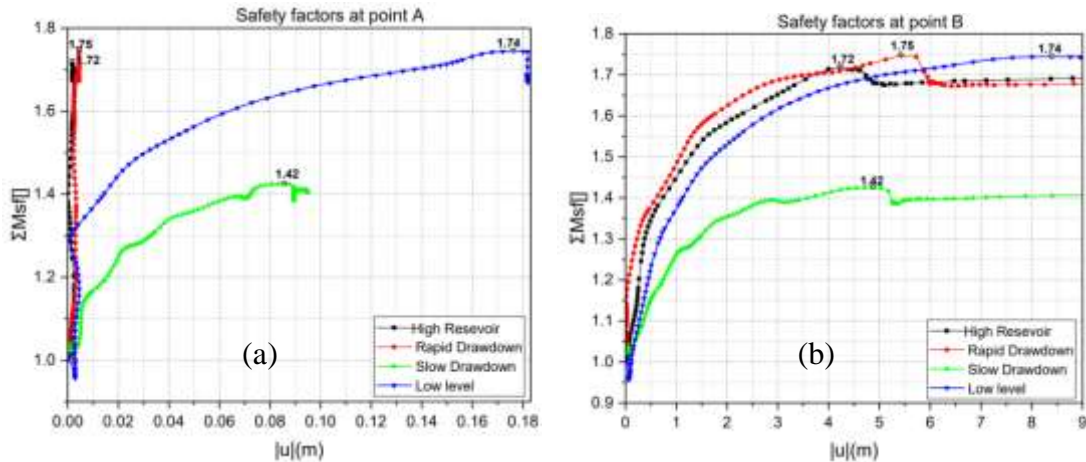
### Low Water Level

A long term situated was considered to estimate the stability of the Akosombo dam when the reservoir water level was at 34.73m. The steady groundwater flow in the dam was calculated using plastic option. The numerical value under the water low level gives a safety factor of 1.74 for all the selected points. The groundwater computed for the pore pressure distribution under the low-level water reservoir and the maximum deformation experienced by the low level are illustrated in Fig.17.



**Figure 17:-** (a) Pore pressure distribution, ( $p_{active}$ ), for Low water level (b) Total displacement ( $|U|$ ) for Low water level.

The behaviour of the Akosombo dam for all the conditions estimated, the displacements that occurred by water pressure in the dam body and foundation are observed. The maximum displacements are 2.2, 0.45, 2.2 and 0.24 for high reservoir under gravity loading, raid drawdown, slow drawdown, and lowest water level conditions, respectively. Factor of safety for all examined analysed conditions are shown in Fig.18. The results show that the safety factor of the dam decreased when the water level was in slow drawdown condition in the reservoir. The Factor of safety values under static condition are within 1.4 – 1.75, which means the slope is safe under static condition.



**Figure 18:-** Computed safety factors for different situations (a) point A at the base (b) point B at the crest.

### Conclusions:-

A preliminary study of the dynamic response of the Akosombo dam was obtained by Plaxis 2D finite element analysis using 19 spectrum earthquake motions. According to the proposed seismic hazard map of southern Ghana, the site of Akosombo dam is situated in a moderate hazard.

The results obtained under the dynamic behaviour of the Akosombo dam provides an adequate basis for the deformations of the dam using different conditions of design earthquakes data. The results showed that deformations increased at the crest of the dam. The present study also aimed to numerically examine the slope stability of the Akosombo earth dam under the drawdown reduction method by Plaxis two-dimensional software. The strength for each load case combination of the dam is expressed through safety factors. Analysing the static stimulation of Akosombo dam using the drawdown principle, displacements and corresponding safety factors are obtained at the high reservoir, reservoir filling, and low water level conditions. The dam body and its foundation parameters used for the analysis are based on engineering properties determined from the literature. The total displacements are significant when studying most of the geotechnical problems. It is particularly recommended that laboratory tests be performed for the dam site conditions. The numerical results show that lower displacements occurred in the dam body and foundation through impounding. A factor of safety presents the primary key for slope stability analysis. The numerical results show that the drawdown reduction method used in the analyse effectively captures the progressive failure induced by reservoir dead load and water level fluctuations. Given the obtained results, when studying the problems related to slope stability in earth/rock-fill dams, it is recommended that inspections be carried out on the Akosombo dam during its service life.

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