

FINITE ELEMENT MODEL UPDATING OF CONCRETE ARCH DAMS AFFECTED BY ALKALI-SILICA REACTION USING AMBIENT VIBRATION MONITORING AND DAM SURVEILLANCE DATA

Thesis Presented for the Degree of

DOCTOR OF PHILOSOPHY

in the Department of Civil Engineering
Faculty of Engineering and the Built Environment
UNIVERSITY OF CAPE TOWN



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May 2024

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DECLARATION

I, Bukhosi Raphael Nyoni, hereby declare that the work presented herein is my own original work (except where acknowledgements indicate otherwise) performed under the supervision of Professor Pilate Moyo, in the Department of Civil Engineering at the University of Cape Town. This dissertation or any part of it has not been, is being or is to be submitted for another degree or examination in this or any other university, college, or institution.

Signed:

Date: May 2024

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ABSTRACT

Dams are important structures whose safety in operation is critical owing to the socioeconomic impacts associated with their collapse. Alkali-Silica Reaction (ASR) is a deleterious chemical reaction that affects the performance and safety of numerous concrete dam structures around the world. To ensure dam safety, a well-structured dam surveillance program that includes ambient vibration monitoring is essential as it enables an in-depth understanding of the behaviour of a dam in operation. Additionally, structural behaviour analysis through numerical models such as finite element (FE) models, is a crucial aspect of dam safety evaluations that allows dam safety engineers to predict the current and future behaviour of dams under various load combinations. The challenge in finite element analysis (FEA) is generating an FE model that is representative of the observed behaviour and condition of the dam to be trusted and used for further study. This study sort to find the best updating parameters and finite element model updating approach to achieve representative and reliable finite element models for the safety assessment of arch dams affected by ASR. This was achieved through sensitivity analysis of the behaviour (both dynamic behaviour and static behaviour) of concrete arch dams to (i) geometrical properties, (ii) material properties and (iii) load effects on the dam-foundation-reservoir system, to identify the best updating parameters. Following which a model updating procedure was proposed and implemented.

It was observed that the behaviour of concrete arch dams is subject to their geometric properties. Their dynamic behaviour was observed to be largely sensitive to the dam-foundation-reservoir system stiffness and water level whereas their thermal behaviour was sensitive to the system thermal properties. Additionally, it was observed that the dynamic response is not as sensitive to ASR loading as initially assumed. Based on these findings, the proposed finite element model updating procedure begins with the updating of the system stiffness using modal parameters, followed by the updating of the thermal properties, and then updating of the parameters critical to ASR modelling. The proposed multi-step model updating procedure was successfully implemented to update a finite element model for a case study double curvature arch dam affected by ASR achieving agreeable results between the measured and predicted behaviour observations.

DEDICATION

To The Almighty God, my wife Pinkie Lynn Nyoni, daughter Lubelihle Nathania Nyoni, father Njabulo Nyoni, mother Gladys Nyoni, and brother Sipho Gabriel Nyoni.

ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to my supervisor Professor Pilate Moyo for the research guidance, technical support, and mentorship. It was a great privilege to work under his supervision.

I would also like to acknowledge the Water Research Commission (WRC) and the Concrete Materials and Structural Integrity Research Unit (CoMSIRU) for providing the funding for the successful carrying out of this research.

Also, I would like to thank and acknowledge the Department of Water and Sanitation for their continued support for this research and for providing the required data and information, especially Mr C. Mahlabela who was the point of contact with the department.

A special thanks goes to all my CoMSIRU colleagues, past and present, whose contribution and support one way or another has been a source of encouragement, especially Dr M. Kabani and Dr P. Bukenya.

My deepest thanks go to my family: my wife Pinkie Lynn Nyoni, daughter Lubelihle Nathania Nyoni, parents Njabulo & Gladys Nyoni, brother Sipho Gabriel Nyoni, extended family and friends for their patience, moral support, and prayers during my studies.

Lastly, I would like to acknowledge Associate Professor F. Martinez and his colleagues, Dr J. Rodriguez, and Dr P. Gonzalez for providing me with their ABAQUS subroutine that they wrote specifically for the application of Saouma's (2014) ASR constitutive model. Without this subroutine, this research was not going to be complete.

Computations were performed using facilities provided by the University of Cape Town's ICTS High Performance Computing team: <http://hpc.uct.ac.za>.

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NOMENCLATURE

List of symbols

C_z	Covariance Matrix of Measured Response
E_r	Weighted sum-of-square residual
W_r	Residual Weighting Matrix
Q	Transformation Matrix (converts nodal pressures into hydrodynamic forces)
S	Sensitivity Matrix
p	Hydrodynamic pressure
r	Residual Column Vector
z	Model Response Column Vector
$\tilde{z}$	Measured Response Column Vector
θ	Column Vector of Updating Parameters
ϵ	Column Vector of Uncertainty/Error
A	Area or Cross-Sectional Area
C_s	Stefan-Boltzmann constant
D_m	Concrete Thermal Diffusivity
E	Material Young's Modulus
H_o	monthly average global solar radiation on horizontal surface
H_b	direct radiation/global irradiance
H_d	diffuse radiation/irradiance
I	Second Moment of Area

I_t	total amount of solar energy reaching the surface
L	Length
K	Bulk Modulus
Q	Rate of heat generation
T	3D temperature field
V	Velocity of pressure waves in water
a	surface absorptivity of solar radiation
b_t	base thickness
cr	Cracking
c	Specific heat
d	Damage Parameter
d_c	Compression related Damage Parameter
d_t	Tension related Damage Parameter
e	surface emissivity
el	Elastic
f_c	Compressive Strength
f_t	Tensile Strength
h	Structural height
h_c	Convection coefficient
i, j, k	indices
k	Heat Conduction Coefficient
l_b	Base length
l_c	Crest length
in	Inelastic
m	number of model outputs
n	number of
p	number of updating parameters

pl	Plastic
q	heat flux
q_c	convection heat transfer rate
q_s	surface heat flux
t	Time
u_o	axial change in length
u	axial displacement
v	transverse displacement
y	distance from the neutral axis
$\bar{x}, \bar{y}, \bar{z}$	Local Cartesian coordinate system
x, y, z	Global Cartesian coordinate system
a	Coefficient of Thermal Expansion
ε	Strain
ε_{el}	Elastic Strain
ε_{pl}	Plastic Strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	Principal strains
ϵ	volumetric Strain
ξ	Expansion
σ	Stress
$\sigma_1, \sigma_2, \sigma_3$	Principal stresses
Δ	Laplace operator for temperature
κ	History-Dependent Hardening Parameter
η	Threshold value
ρ	Material Density
$\frac{\delta G}{\delta \sigma}$	Derivative of the Plastic Potential Function
τ_l	Latency time
τ_c	Characteristic time

ν	Poisson's Ratio
λ	Eigenvalue
ω	Resonant/Natural frequency
$ \ $	Symbol of Absolute Value or Magnitude (Modulus Symbol)
$\ \ \ $	norm
$\cdot$	Symbol meaning derivative taken with respect to time (i.e., rate of)
$\ddot{}$	Second derivative taken with respect to time (i.e., rate of)
$-$	In the local coordinate system
$\{ \ }$	Column Vector
$[\]$	Matrix
$[\]^A$	Initial (a priori) Matrix
$[\]^U$	Initial (a priori) Matrix
$[\]^T$	Matrix Transpose
$[\]^{-1}$	Matrix Inverse
$[\]_e$	Elemental Matrix
$[I]$	Identity Matrix
$[N]$	Shape Function Matrix
$[B]$	Strain Displacement Matrix
$[C]$	Viscous damping model
$[E]$	Material Property Matrix
$[M]$	Mass Matrix
$[K]$	Stiffness Matrix
$[K]^\sigma$	Stress-Stiffness Matrix
$[K]^S$	Secant Stiffness Matrix
$[S]$	Sensitivity Matrix
$[S]^+$	Sensitivity Pseudo-Inverse Matrix
$[S_n]$	Normalised Relative Sensitivity Matrix

$[S_r]$	Relative Sensitivity Matrix
$[T]$	Transformation Matrix
$[k]$	System Thermal Conductivity Matrix
$[\phi]$	Mass Normalised Eigenvector Matrix
$[\partial]$	Matrix of Differential Operators
$[\zeta]$	system heat capacity matrix
$\{\Psi\}$	Eigenvector
$\{x\}$	Displacements at degrees of freedom
$\{u\}$	General Displacements
$\{\Delta\}$	Nodal Displacements
$\{F\}$	Loads/Force applied at the degrees of freedom

List of Abbreviations

<i>AE</i>	Acoustic Emission
<i>ASR</i>	Alkali-Silica Reaction
<i>AVT/M</i>	Ambient Vibration Monitoring
<i>COD</i>	Crack Opening Displacement
<i>DL</i>	Damage Level degree-of-freedom (DOFs)
<i>DOF</i>	Degree-Of-Freedom
<i>DRI</i>	Damage Rating Index
<i>DWA</i>	Department of Water Affairs
<i>DWS</i>	Department of Water and Sanitation
<i>EDX</i>	Energy-Dispersive X-ray
<i>EMPA</i>	Electron Probe Micro Analysis
<i>FE</i>	Finite Element
<i>FEA</i>	Finite Element Analysis
<i>FEM</i>	Finite Element Method

<i>FEMU</i>	Finite Element Model Updating
<i>FRF</i>	Frequency Response Function
<i>FRP(s)</i>	Fibre Reinforced Polymer(s)
<i>FSL</i>	Full Supply Level
<i>FVT</i>	Forced Vibration Testing
<i>GNSS/GPS</i>	Global Navigation Satellite System/ Global Positioning System
<i>GRGWS</i>	Gamtoos River Government Water Scheme
<i>IE</i>	Impact Echo
<i>ITZ</i>	Interfacial Transition Zone
<i>K⁺</i>	Potassium Cation
<i>LFL</i>	Lowest Foundation Level
<i>Li₂CO₃</i>	Lithium Carbonate
<i>LiCl</i>	Lithium Chloride
<i>LiF</i>	Lithium Floride
<i>LiNO₃</i>	Lithium Nitrate
<i>Li₂SiO₃</i>	Lithium Metasilicate
<i>Li₂SO₄</i>	Lithium Sulfate
<i>Na⁺</i>	Sodium Cation
<i>NaCl</i>	Sodium Chloride
<i>NDT/E</i>	Non-Destructive Testing/ Evaluation
<i>Na₂O_{eq}</i>	Equivalent Sodium Content
<i>NaOH</i>	Sodium Hydroxide
<i>NOC</i>	Non-Overspill Crest
<i>PDF</i>	Probability density functions
<i>RH</i>	Relative Humidity
<i>RL</i>	Relative Level
<i>SCM</i>	Supplementary Cementing Material

<i>SDT</i>	Stiffness Damage Test
<i>SEM</i>	Scanning Electron Microscope
<i>SHM</i>	Structural Health Monitoring
<i>T</i>	Temperature
<i>UAS</i>	Unmanned Aerial Systems
<i>UPE</i>	Ultrasonic Pulse Echo
<i>UPV</i>	Ultrasonic Pulse Velocity
<i>UQ</i>	Uncertainty Quantification
<i>USDOT-FHA</i>	United States Department of Transportation, Federal Highway Administration
<i>XFEM</i>	Extended Finite Element Method
<i>XRD</i>	X-Ray Diffraction

GLOSSARY

Absorptivity – refers to the fraction of incident radiant energy absorbed by the concrete surface (Janna, 1986).

Abutment – is that part of the valley wall against which the dam is constructed and the part of a dam that contacts the riverbank (Salamon & Nuss, 2012).

Arch dam – is defined as a solid concrete dam curved in the plan view so as to transmit the major part of the water load to the abutments (Salamon & Nuss, 2012).

Arching action – causes horizontal stresses in the dam and transmit load into the abutments (Salamon & Nuss, 2012).

Arches – of an arch dam are horizontal sections through the dam from abutment to abutment (Salamon & Nuss, 2012).

Axis of dam – is a vertical cylindrical reference plane along the upstream side of the dam crest (Salamon & Nuss, 2012).

Axis radius – is the horizontal distance along the reference plane from the axis of the dam at the crest to the line of centres and defines the axis of the dam (Salamon & Nuss, 2012).

Base – is the bottom surface of the dam resting on the foundation (Salamon & Nuss, 2012).

Base width or thickness – is the radial thickness of a dam measured horizontally between upstream and downstream faces and normal to the axis of the dam but excluding projections for outlets or other appurtenant structures (Salamon & Nuss, 2012).

Calibration – involves adjusting model parameters to maximise its agreement with field test results.

Cantilevers – of a concrete dam are vertical sections through the dam perpendicular to the axis of the dam (Salamon & Nuss, 2012).

Cantilever Action – causes vertical stresses in the dam and transmit forces down into the foundation (Salamon & Nuss, 2012).

Central Angle – angle at extrados centre formed by lines extended to left and right arch abutments (Salamon & Nuss, 2012).

Closure temperature – is the base state temperature state after grouting of construction joints.

Crest – is the top surface of the main body of the dam (excluding parapet walls) or the high point of a spillway (Salamon & Nuss, 2012).

Crest width – is the thickness of a dam at the crest (Salamon & Nuss, 2012).

Crown or crown cantilever – is a vertical upstream to downstream section through an arch dam at the maximum height, generally along the reference plane (Salamon & Nuss, 2012).

Double-curvature arch dam – is an arch dam that is curved in plan (horizontally) and in elevation (vertically) (Salamon & Nuss, 2012).

Diagnosis – involves the identification of the nature or cause of structural deterioration.

Emissivity – is the relative ability of exposed surfaces to emit radiant energy (Nzuza, 2013).

Extrados – define upstream face of the horizontal arches (Salamon & Nuss, 2012).

Failure (of a dam) – is defined as the major unintended and uncontrolled release of retained water, or an event whereby a dam is rendered unfit to safely retain water because of total loss of structural integrity (UK Environmental Agency, 2011).

FE structure – refers to the assemblage of elements (Cook et al., 2002).

Foundation – total mass of sound rock supporting the dam (Salamon & Nuss, 2012).

Heel of dam – of the dam is the intersection of the upstream face of a concrete dam with the foundation (Salamon & Nuss, 2012).

Intrados – define the horizontal arches of the downstream face (Salamon & Nuss, 2012).

Line of centres – define the geometry of the centres of radii of the upstream and downstream faces horizontal arcs of an arch dam along the reference plane (Salamon & Nuss, 2012).

Medium-thick arch dam – is an arch dam with a base thickness to structural height ratio between 0.2 and 0.3.

Prognosis – involves predicting the potential for future expansion and/or resulting further deterioration.

Reference plane – is an imaginary upstream to downstream vertical section through the crown cantilever of a concrete dam and line of centres (Salamon & Nuss, 2012).

Single-curvature arch dam – is curved in plan only and not in section view (Salamon & Nuss, 2012).

Structural height – is the distance between the lowest foundation level (LFL) and the top of dam.

Thick-arch dam – is an arch dam with a base thickness to structural height ratio of 0.3 or greater.

Thickness of dam – horizontal distance between upstream and downstream projection of dam on a line normal to axis of the dam (Salamon & Nuss, 2012).

Thin-arch dam – is an arch dam with a base thickness to structural height ratio of 0.2 or less.

Toe of the dam – is the point of intersection between the downstream face at the base of the dam and the ground (Salamon & Nuss, 2012).

Validation – includes comparing the field test results with the model results to quantify the accuracy of the model to represent the real.

Amorphous – refers to highly disordered crystalline structure.

Hygroscopic – tends to absorb water from surrounding.

Reflectivity – is a ratio of the reflected solar radiation to the total amount of solar radiation that falls on a particular surface (ACPA, 2002).

Structural Deterioration – includes the process of progressive loss or reduction in resistance of a structure to structural loads.

Transmissivity – refers to the fraction of incident radiant energy that penetrates through a layer that is close to the surface on the exposed part of a concrete dam.

Mathematical Model – is defined as a geometric model whose behaviour has been described or approximated by selected differential equations and boundary conditions (Cook et al., 2002).

Mesh – is an arrangement of elements (Cook et al., 2002).

Numerical method – is an approximate computer method for solving a mathematical problem which often has no analytical solution (Essential MATLAB for Engineers and Scientists (5th Ed), 2013).

Chapter 1

INTRODUCTION

1.1 Background to Research

Dams are important structures used for the control and storage of water for agricultural, domestic, or industrial needs, power generation, or flood protection and control, among other uses (ICOLD, 1987). Their catastrophic failure (i.e. dam failure), which is defined as the unintended and unrestrained release of impounded water due to the loss of structural integrity of a dam (UK Environmental Agency, 2011), would inevitably result in socio-economic impacts such as loss of life, economic losses and water scarcity among other impacts. Consequently, the safety and performance of dams is a concern for those involved in their design, construction, and operation (ICOLD, 1987). Additionally, dam safety has increasingly become a subject of public interest in the last decades due to dam failures, such as the Malpasset Dam in France and Leguaseca (Fonsagrada) in Spain, and public concern regarding environmental aspects associated with dams. (Pedro, 1999). The fundamental objective of dam safety is to protect humans, property, and the environment from injurious consequences of improper operation or failure of dams in all stages of their service life (ICOLD, 2017).

The failure of a dam is generally a complex process that usually begins with an undetected abnormality in its behaviour that results in unobserved deterioration leading to further damage and/or dam failure ((ICOLD, 1987). An in-depth understanding of the structural behaviour of a dam in operation, often achieved through a well-structured dam surveillance program, is essential in preventing dam failure (Pedro, 1999). Dam surveillance involves visual inspection and monitoring of a dam, providing for continuously updating the knowledge about the state of the dam throughout its service life ((ICOLD, 2009), (ICOLD, 2017)).

Monitoring for dam surveillance purposes traditionally consists of measuring static physical parameters (generally referred to as static monitoring or static response monitoring) such as displacements, temperature, water level, and pore pressures, which are related to structural parameters such as internal forces, stresses, and strains in the dam wall and its foundation ((ICOLD, 1989) (Fenves et al., 1992)). Whilst dynamic monitoring, initially considered important for dam structures located in seismic regions, has in recent years been implemented to successfully provide crucial information for dam safety evaluation ((ICOLD, 1989) (Fenves et al., 1992) (Gomes & Lemos, 2020)). Dynamic monitoring involves measuring vibrations induced on the dam by water waves, wind, and other operational loads. The measured vibrations are analysed to extract the dynamic properties of the dam ((Alegre et al., 2019) (Wenzel & Pichler, 2005)). The premise for dynamic monitoring is that the measured dynamic properties of dams (i.e., natural frequencies, mode shapes and damping ratios) can be related to their as-built and/or the most current dynamic structural behaviour (i.e., structural parameters, namely, modal mass, stiffness, and damping) ((Brincker & Ventura, 2015) (Haidarpour & Tee, 2020)). Dynamic monitoring of dams in operation where the force causing the vibrations is not measured is referred to as ambient vibration monitoring (AVM) (Wenzel & Pichler, 2005).

Exploration of the application of ambient vibration monitoring of dams started around the 1980s. The challenges reported from these early attempts were associated with limitations of instrumentation and data processing algorithms ((Calciati et al., 1979) (Ellis et al., 1983), (Clough et al., 1984) (Brownjohn et al., 1986)). Since then, there have been considerable advancements in instrumentation and data processing procedures (Wenzel & Pichler, 2005). However, there are still two drawbacks in modal identification using AVM namely, (i) the resulting mode shapes are not mass normalised, and (ii) not all modes are well excited (Brincker & Ventura, 2015). Concerning the first drawback, some critical applications such as structural response simulation and load estimation require mass scaled mode shapes (Parloo et al., 2003). Since input forces are not measured in AVM, the modal parameter identification process does not provide mass normalised mode shapes, only unscaled mode shapes (Parloo et al., 2005). To overcome this limitation, experimental tests in which the dynamic behaviour of a structure is measured while changing its mass or stiffness, are normally carried out ((Lopez-Aenlle et al., 2010) (Khatibi et al., 2009). Alternatively, finite element method (FEM) based mass matrices can also be determined experimentally (Aenlle & Brincker, 2014). However, for large structures such as dams, it is not easy to track the change in mass or

stiffness using AVM (Aenlle & Brincker, 2013). Thus, obtaining the scaling factors for the mode shapes still poses a challenge. Concerning the second drawback, dependence upon uncontrollable excitation sources in AVM means that the range and location of the excitation forces cannot be dictated. Consequently, not all the vibration modes may be well excited resulting in difficulty identifying these less excited modes. To overcome this limitation, an understanding of the expected structural response and/or expected levels of vibration may be useful to focus the vibration measurements within the expected vibration range of the dam structure ((Rainieri & Fabbrocino, 2014) (Brincker & Ventura, 2015)).

A crucial aspect of dam safety evaluation is structural behaviour analysis through numerical models such as finite element (FE) models ((Pedro, 1999) (ICOLD, 2001). FE models allow dam safety engineers to evaluate the current structural response as well as predict and evaluate the future behaviour of dams under various load combinations, including thermal loads, hydrostatic loads, seismic loads, and internal swelling due to chemical reactions ((ICOLD, 2001) (ICOLD, 2013)). The accuracy and reliability of these numerical models depend on how well dam surveillance data (both static and dynamic measurements) is integrated with the numerical modelling process (ICOLD, 2013). It goes without saying that the more information about the current behaviour of a dam one incorporates in the numerical modelling process, the more representative and reliable the numerical model becomes (ICOLD, 2013). To this end, information from static and/or dynamic measurements, is used to calibrate and validate finite element models to match measurements and observations as closely as possible ((Simoen et al., 2014) (Prakash et al., 2022)). The systematic tuning and validation of model assumptions and parameters until there is an agreement between the finite element model and monitoring data is referred to as Finite Element Model Updating (FEMU) (Marwala, 2010).

Finite element model updating is now an established field. The main challenge in the finite element model updating of concrete arch dams using ambient vibration measurements arises from the highlighted drawbacks of ambient vibration monitoring, i.e. the unknown input forces and limited measurement points owing to the large size of these structures. Up until recently ((Calcina et al., 2014) (Hattingh et al., 2019) (Gomes & Lemos, 2020) (Gaftoi et al., 2021)) there has been limited literature on the finite element model updating of dams using ambient vibration measurements, whilst several technical papers have been reported in other fields such as in bridge structures ((Brownjohn et al., 2001) (Xia & Brownjohn, 2004) (Sabamehr et al., 2018) (Sun & Xu, 2020)). Given that ambient vibration monitoring (AVM) has increasingly become an important component in the surveillance and safety evaluation of dam

structures (Gomes & Lemos, 2020), there is need of providing technically and scientifically sound procedures and guidance on the incorporation of AVM in the evaluation of these structures, specifically, in the updating of FE models.

With the understanding of dam failure as a complex process described above, the safety of a dam is realised when the dam is free of any conditions or developments that could lead to its deterioration or destruction (ICOLD, 1987). The safety of numerous concrete dams around the world including some arch dam structures, has been compromised by internal swelling of the concrete due to chemical reactions (commonly referred to as alkali-silica reactions (ASR) or alkali-aggregate reactions (AAR)) between silica-based aggregates and alkalis in the concrete (Curtis, 2000). These reactions will be referred to as ASR henceforth in this thesis. ASR often results in cracking of dam walls and malfunctioning of the dam's operation equipment ((ICOLD, 1991) (Mason & Molyneux, 1998)). Many assessment tools developed to assess the impact of ASR on the performance and safety of concrete structures including different ASR modelling approaches, have been developed based mainly on laboratory-based tests and/or observations which in many ways do not represent field conditions and observations ((Roberts et al, 2012) (Giorla, 2013) (Saouma, 2021) (Nguyen et al., 2019)). Moreover, the key parameters used to define the reaction kinetics for most of the existing ASR constitutive models require laboratory tests. In the absence of laboratory test data, assumptions to characterise the reaction kinetics must be made (Saouma, 2014). Thus, there is a need for a review of existing modelling approaches of ASR to account for field conditions or observations, and the development of procedures to characterise the reaction kinetics using field observations in ensuring the reliable evaluation of the impact of ASR on the safety and operation of affected structures such as concrete arch dams ((Giorla, 2013) (Nguyen et al., 2019)).

Another aspect of the assessment of the impact of ASR on the structural performance of dams that remains a challenge is the determination of the change in wall stiffness of the dam due to internal stresses induced by ASR. While finite element models provide an excellent basis for the evaluation of internal stresses induced by ASR, the link between finite element modelling results and the current state of the structure is often missing. Ambient vibration measurements, coupled with deformation measurements provide an opportunity to evaluate stress-induced stiffness (stress stiffness also known as geometric stiffness) in concrete arch dams (Moyo & Oosthuisen, 2013). Generally, the dynamic properties of concrete arch dams are sensitive to seasonal changes in the water level while temperature effects are negligible.

On the other hand, the seasonal deformation of concrete arch dams is more sensitive to ambient temperature (Bukenya, 2020).

More recently, there has been an increased awareness on potential climate change-related risks to dam safety. Climate change, a consequent of global warming, is projected to cause changes in climatic conditions including and/or evidenced by ambient temperature increases and increased frequency of extreme events such as flooding and heatwaves among other consequences (ICOLD, 2016). Given that, ASR is a temperature-dependent reaction (Saouma, 2014), and since ASR continues to affect dams beyond previously anticipated periods (Giorla, 2013), an improved evaluation of the impact of ASR on concrete arch dams is essential in the context of global warming. Increasing temperatures lead to accelerated ASR reaction rates and swelling of concrete resulting in a permanent change in the state of internal stress in concrete arch dams, affecting dam safety. Additionally, global warming and climate change may result in a shift in climatic and loading events on concrete arch dams to levels not thought and/or imagined during the design of these structures. Thus, it is essential to understand and investigate the potential impact of climate change on the behaviour and safety of concrete arch dams ((ICOLD, 2016)).

In summary, the key issues in the assessment of the structural behaviour of concrete arch dams using AVM that still require further study include the lack of mass scaling of the mode shapes when using this technique. The implications include the difficulty in determining the exact structure's modal mass and/or stiffness directly from the measurement results. Secondly, given that there has been an increase in the application of ambient vibration monitoring (AVM) of concrete arch dams, there is a general need to provide guidance as to the best practice and/or approaches of incorporating AVM data in the management and assessment of dam structures. Thirdly, ambient vibration measurements, coupled with deformation measurements provide an unexplored opportunity to accurately evaluate stress-induced stiffness and hence, overall safety of concrete arch dams using well-updated finite element models. Lastly, the assessment of the impact of ASR in the context of climate change including the determination of the change of dam wall stiffness due to the internal stresses induced by ASR and field-based estimations of the reaction kinetics remain as further challenges.

1.2 Problem Statement

As alluded to in the foregoing discussion, the challenge in finite element analysis (FEA) is generating an FE model that is representative of the observed behaviour and condition of the dam to be trusted and used for further study. The structural behaviour of concrete arch dams in operation depends on many variables including material properties (both the dam wall and foundation properties), varying load conditions (such as the reservoir level, seasonal temperature changes and for dams affected by ASR, swelling with time) and the interaction of the dam system components (dam-foundation-reservoir interaction). Structural Health Monitoring (SHM) and Vibration testing offer insight into the behaviour of concrete arch dams. By monitoring the time varying loads and the response of the dam, a deeper understanding of the sensitivity of structural parameters to time varying loads can be achieved which in turn can be used to improve FE models. Consequently, the accuracy and reliability of FE models depend on how well dam surveillance data (both static and dynamic measurements) is integrated with the numerical modelling process through finite element model updating. As highlighted by the second and third key issues in the summary section of the foregoing discussion, there is therefore a need of practical and holistic model calibration approaches for concrete arch dams that accounts for the complex interactions and influences of the aforementioned variables resulting in accurate and reliable numerical models.

1.3 Key Research Questions

To this end, this research seeks to address the following questions:

- i. What is the state of the art regarding the structural assessment of ASR affected concrete arch dams?
- ii. How is model updating carried out to account for changes in the internal stress state of a structure?
- iii. What is the current state of practice regarding the incorporation of AVM data in the management and safety evaluation of concrete arch dams?
- iv. Which parameters are the best to adjust when updating FE models of concrete arch dams?
- v. What is the best model updating approach of concrete arch dams including those that are subject to ASR, using ambient vibration measurements?

1.4 Research Aim and Objectives

The overarching aim of this research is to contribute to reliable dam safety evaluation of concrete arch dams affected by ASR focussing on the development of reliable and accurate finite element models. This aim will be achieved by addressing the following objectives:

- i. To investigate the influence of geometrical properties of arch dams on their dynamic behaviour under ambient conditions.
- ii. To investigate the influence of stress-induced stiffness on the dynamic behaviour of concrete arch dams.
- iii. To identify the most relevant parameters to update to achieve reliable and accurate models for the assessment of the current state of behaviour of concrete arch dams.
- iv. To develop an integrative FE model updating approach for concrete arch dams.

1.5 Scope and Limitations of the Research

This study focuses on the development of a finite element model updating procedure for concrete arch dams affected by ASR (using both static and dynamic monitoring data) with particular focus on the incorporation of AVM in the model updating procedure. All work conducted in this study is aimed towards this end. As a result, the study considers only the operational behaviour of concrete arch dams in regions with no significant seismic activity. While seeking to remain general to concrete arch dams, this study does not seek to address every load case dam structures in different parts of the world experience. Thus, the load cases considered in this study include gravity loads, water loading, temperature loading and ASR. Additionally, this study does not seek to investigate the factors critical to and the proper implementation of (i) temperature loading (i.e., temperature analysis) and of (ii) ASR loading (ASR constitutive model implementation). Instead, the proper implementation of these loading conditions in FEM is assumed (with the references of the proper implementation illustrations of these loads provided) as the focus is to illustrate the updating procedure. All numerical modelling efforts were done using the general-purpose finite element analysis (FEA) programme, ABAQUS. The proposed updating procedure was implemented to update an FE model of an existing dam (Kouga dam) affected by ASR with all the monitoring data provided by the Department of Water and Sanitation in South Africa and available through field investigations carried out on the dam.

1.6 Significance of the Research and Contribution to Knowledge

The novelty in this research is in decoupling the complex structural behaviour of concrete arch dams (which are a function of geometric & material properties, varying load conditions and the interaction of the dam system components) to enable the defining of FEMU strategy, identification of the relevant parameters to and the information needed to update the model without compromising accuracy and reliability in representing the real dam system, its interactions and behaviour. Current practice in FE modelling of dams does not integrate static and dynamic measurements and/or behaviour analysis and assessment of concrete dam structures. By considering both the dynamic and static behaviour of a dam structure, a more accurate, reliable and realistic representation of the real dam system and its behaviour can be achieved. The detailed contribution of this research includes:

- i. The characterisation of the dynamic behaviour of various arch dam configuration(s) to understand the impact of different parameters that influence the geometry of these structures such as site type, type of arch and classification of arch, on the dynamic behaviour of these structures. The dynamic behaviour characterisation results may be useful in cases where the objective is to understand the dynamic behaviour of different arch dam options and/or get an estimate of the expected dynamic behaviour (with respect to ambient vibration monitoring) for cases in which design and site-specific details are not too important such as in preliminary design stages of new structures.
- ii. A detailed investigation of parameter sensitivities in the context of FE model updating, considering the variation in material properties of and interaction of the dam-foundation-reservoir system components with each other and with their surroundings (through the applied environmental related loads such as ambient temperature and water level) and due to ASR loading. The results of the parameter sensitivity study formed the basis for the parameter selection of the proposed FE model updating approach in this study and have the potential to aid in understanding and evaluating the behaviour of concrete arch dams both for existing and new structures.
- iii. The development of an FE model updating procedure using both static and dynamic monitoring data of concrete arch dams affected by ASR. The proposed finite element model updating approach is among the first attempts to calibrate

concrete arch dam models using both dynamic and static measurements in the development of updated FE models that can accurately predict the current and future behaviour of these dam structures. The proposed model updating approach will be useful to dam safety engineers for the assessment of the performance and safety of these structures.

1.7 Research Approach Strategy

This research study was undertaken by first conducting a critical review of the literature pertinent to this study with three intentions: (i) to develop a comprehensive analysis of literature and state of the art regarding ASR in ensuring dam safety, (ii) to develop a theoretical foundation and state of the art regarding finite element model updating of loaded structures (i.e., accounting for internal stress state changes in structures), and (iii) to describe the current state of practice as regards model development and the incorporation of surveillance data in numerical model assessment of concrete arch dams. Additionally, the last component of the literature review served to provide observational data used to validate some of the theoretical observations from this study. This first step was essential in the development of the numerical analysis methodology ultimately implemented in this study.

The second step in undertaking this research was a sensitivity analysis to understand the sensitivity of the structural behaviour of concrete arch dams to different parameters with the aim of identifying the most effective updating parameters when considering ASR effects on these structures using ambient vibration monitoring data. The last step, building/based on the previous steps was the development and application of a proposed FEMU procedure for concrete arch dams affected by ASR using both static and dynamic monitoring data.

1.8 Overview of the Thesis

This thesis work is presented in eight chapters, which explore the different aspects of the research undertaken.

Chapter 1 provides the research background, problem statement, questions, aim(s), objectives, scope, limitations, significance of research, contribution to knowledge and research approach strategy.

Chapter 2 provides a detailed review of ASR, its structural implications, prevention, diagnosis, prognosis, mitigation, and modelling approaches. This section concludes with a review on structural integrity and performance assessment of concrete arch dams.

Chapter 3 provides the theoretical basis for and a review of technologies for finite element model updating. Additionally, a review on the modelling of, model updating of, and the structural behaviour assessment of loaded structures is here discussed.

Chapter 4 presents the methods used to carry out this study including a detailed discussion on the development of finite element models used in this study, the loading considerations, a detailed description of the modelling approaches for the thermal analysis, ASR modelling, dynamic behaviour analysis and the current state of practice as to the incorporation of surveillance data in the modelling process.

Chapter 5 investigates the dynamic behaviour characterisation of various configurations of concrete arch dams, from which the dynamic properties, i.e. natural frequency and mode shapes of the various configurations were obtained and compared to field observations.

Chapter 6 investigates the sensitivity of the structural response of concrete arch dams to load effects (namely: thermal loads, water loads and ASR) and variation in material properties (namely: concrete density, coefficient of thermal expansion, the dam wall-foundation stiffness Poisson's ratios).

Chapter 7 presents the proposed Finite Element Model Updating procedure for concrete arch dam structures affected by ASR and its application to a case study dam, Kouga Dam.

Chapter 8 presents the main research conclusions and suggests areas for future research.

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Chapter 2

A REVIEW OF THE ASSESSMENT AND MANAGEMENT OF CONCRETE INFRASTRUCTURE AFFECTED BY ALKALI-SILICA REACTION

2.1 Introduction

Alkali-Silica Reaction (ASR) affects the performance, structural integrity, and service life of numerous concrete structures. This chapter discusses the assessment and management of concrete structures affected by ASR with a particular focus on concrete dam structures to meet the objectives of this study. The chapter begins with a brief review of the mechanism of ASR and its structural implications. This is followed by a discussion of the diagnosis, appraisal, prognosis, and mitigation of ASR. Next, a review of the state-of-the-art structural modelling approaches of ASR, including the concrete material model in the context of ASR modelling follows. The chapter ends with a review of concrete dam-specific considerations regarding the assessment and management of these structures.

2.2 Overview of ASR

Since its discovery in the 1940s, ASR has been the subject of many studies by engineers and scientists. Many researchers have conducted a detailed review of the chemistry and mechanism of the reaction, including Stanton (1940), Hansen (1944), Gillott (1975), Diamond et al. (1981), Dent-Glasser & Kataoka (1981), Swamy (1992), Fournier & Berube (2000), and Rajabipour et al (2015). For this study, a brief summary of the mechanism and chemistry of ASR is provided. ASR is a chemical reaction between silica-based aggregates and alkali constituents in concrete whose product, an alkali-silica gel, forms within the boundary of concrete aggregates. The alkali-silica gel swells as it absorbs moisture inducing internal pressures within the concrete microstructure, resulting in cracking (Figure 2.1). The formation

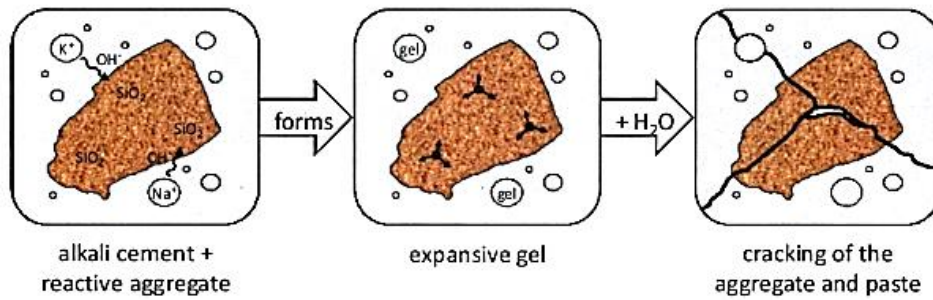


Figure 2.1: ASR Mechanism (adopted from Roberts et al, 2012)

of cracks at aggregate level due to the expansion of the reaction products varies for different aggregate types (Sanchez et al., 2015).

Three types of reaction mechanisms have been reported in literature, namely: (i) peripheral reaction rim formation (Figure 2.2a) observed in quartz-bearing aggregates such as granites and quartzitic diorites, which have a non-porous structure resulting in the reaction products forming around the aggregate. The generated pressure by the swelling alkali-silica gel induces cracking of the aggregates and surrounding cement paste (Ichikawa & Miura, 2007), (ii) diffusion reaction(s) in gel pockets within the aggregates (Figure 2.2b) observed in porous aggregates such as quartzitic sandstones. In this mechanism, thinner reaction rims around the aggregate develop as the alkali can migrate, resulting in the reaction products developing within the aggregates. Consequently, the micro-cracks first develop within the aggregate and spread radially to the surrounding cement paste ((Rivard et al., 2002) (Çopuroglu, 2010)) and (iii) formation of veins within aggregate particles (Figure 2.2c) observed in aggregates such as silica limestone aggregates. In this mechanism, the expansive gel develops as veins that follow the original alignment of the rock bedding within the aggregates. The development of the micro-cracks follows the presence of impure silica becoming thinner towards the cement paste (Esposito, 2015).

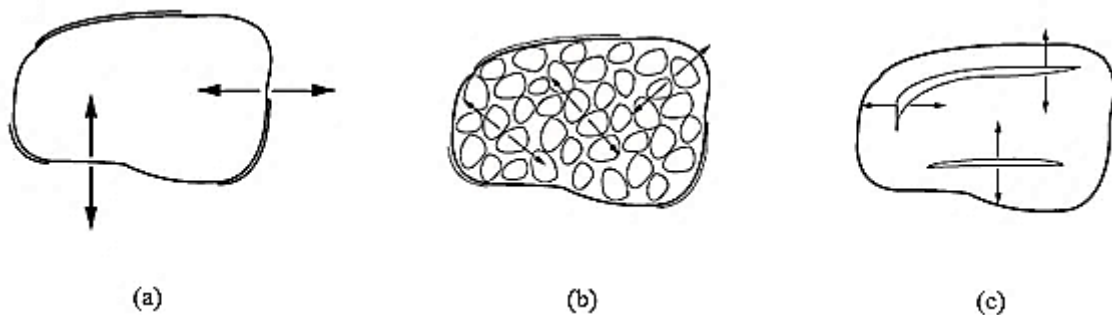


Figure 2.2: Induced ASR damage mechanisms for different types of Aggregates (adopted from Sanchez et al, 2015)

The concrete mix design, casting method, and environmental conditions influence the expansion of concrete induced by the swelling reaction product (alkali-silica gel) ((Swamy, 1992) (Larive, 1998)). A sinusoidal evolution (in time) of the expansion (Figure 2.3a) (i.e., sigmoid expansion curve) proposed by Larive (1998) is widely accepted as characterising the expansion. The concrete mix design from which concentrations of water, alkali and silica are controlled influences the maximum expansion possible, the nature of aggregates and the ability of the pore fluids to move within the concrete (Swamy, 1992). The aggregate size affects the rate and maximum expansion of the concrete. Generally, the expansion rate is faster and the expansion greater for concrete with smaller aggregates compared to concrete with larger reactive aggregates of the same type ((Zhang et al., 1999) (Andic-Cakir et al., 2009) (Multon et al., 2010)). Temperature and humidity (environmental factors) also influence ASR expansion. The reaction rate increases as the temperature increases (Figure 2.3a), whilst, below a relative humidity of 50%, the reaction does not initiate (Figure 2.3b). Lastly, the casting method influences the direction of expansion and crack formation in the concrete ((Swamy, 1992) (Larive, 1998)).

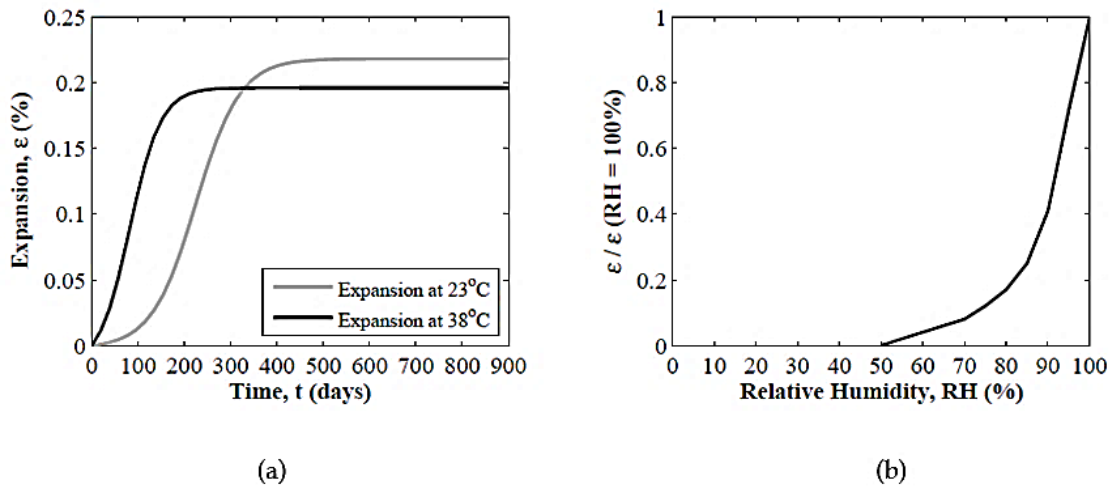


Figure 2.3: Influence of environmental conditions on concrete expansion (a) Effect of temperature (Larive, 1998); (b) Effect of relative humidity (adopted from Swamy, 1992)

2.3 Structural Implications of ASR

The expansion, and degradation of the mechanical properties (due to cracking within the aggregates and/or cement paste) of concrete constitute ASR-induced concrete degradation (Kongshaug et al., 2020). Numerous studies on the degradation of mechanical properties of concrete due to ASR are reported in the literature with varied observations (Swamy & Al-

Asali, 1988) (ISE, 1992) (Ahmed et al., 2003) (Giaccio et al., 2008) (Esposito, 2015) (Nguyen et al., 2019) (Mohammadi et al., 2020). In general, the elastic modulus of concrete (evaluated using both static and dynamic elastic modulus tests to define the residual stiffness in the material) is the most significantly affected and most sensitive to ASR related damage. Similarly, the tensile strength of concrete is easily and significantly affected by ASR ((Nguyen et al., 2019) (Mohammadi et al., 2020)). On the other hand, varied observations due to ASR on the compressive strength have been reported in literature, from strength gain, no effect, to reduction in compressive strength under different conditions ((Mohammadi et al., 2020) (Esposito, 2015)). The compressive strength behaviour due to ASR is the most inconsistent. However, the influence of ASR on the failure mechanism of concrete in compression has been observed (Giaccio et al., 2008). Additionally, a non-linear relationship of the compressive and tensile behaviour with respect to elastic modulus is reported in literature for ASR affected concrete. Thus, the known unaffected concrete stiffness-strength relationships cannot be used to predict the stiffness-strength relationships of ASR-affected concrete (Esposito, 2015). Additionally, varying extents of degradation for the same level of ASR expansion have been reported in literature. Consequently, it is still difficult to directly relate the degradation of mechanical properties to ASR expansion in general due to other factors that affect ASR related degradation such as the concrete mix design, crack distribution and orientation ((Nguyen et al., 2019) (Mohammadi et al., 2020)).

At a structural level, the extent to which ASR compromises the performance of structures and/or structural elements is uncertain as contradictory observations including no change, or an increase in capacity of the affected structural elements have been reported in literature ((Clark & Ng, 1989) (Giannini, 2012)). The pressurizing action due to the expansive alkali-silica gel appears to be a helpful pre-stress within the material, especially if the loading condition and/or reinforcements act to restrain the swelling ((Clark & Ng, 1989) (Giannini, 2012)). Usually, material deterioration is investigated for the expansion of unconfined concrete (Esposito, 2015). Confinement in a structure due to reinforcement, loading conditions, configuration of the structure, and the interaction of different components of a structural system, restrain ASR expansion, reducing the extent of deleterious cracking and consequent degradation of the mechanical properties of concrete. Significant degradation of mechanical properties of cores taken from confined ASR-affected structures has been observed when compared to the in-situ structure due to the loss of confinement once the cores are extracted. Consequently, in some cases the extracted cores cease to represent the context of the structure

((Roberts et al, 2012) (USDT-FHA, 2012)). Furthermore, it has been observed that (i) there is no difference in behaviour between ASR affected and ASR unaffected concrete sections in the linear-elastic range (Roberts et al, 2012), and (ii) the stiffness of restrained concrete sections for structures or structural elements restrained in all three directions increases (Roberts et al, 2012). Thus, confinement of a structure, loading mechanisms and other restraints in structures are key factors when assessing the impact of ASR on a structure. Given the interaction between deleterious ASR expansion and structural restraint, ASR related structural evaluations dependent on structural testing rather than on material testing of cores extracted from the structure is the most favourable for the assessment of structural impacts due to ASR (Roberts et al, 2012).

2.4 Preventative Measures of ASR

There are three conditions essential for deleterious ASR to occur namely, (i) adequate amounts of reactive silica, (ii) adequate concentration of alkali, and (iii) sufficient moisture (Stark et al., 1993). Eliminating any of these conditions will prevent the development of deleterious ASR. The source of the reactive silica is primarily the aggregate ((Offei et al. 2023) (USDT-FHA, 2012)). A careful selection of the type of aggregate (where possible) can ensure that damaging ASR is prevented from occurring (Offei et al. 2023).

Based on Stanton's work (1940), it is widely accepted that ASR is unlikely to develop when the cement alkali content is below 0.6% $\text{Na}_2\text{O}_{\text{eq}}$ (Thomas et al., 2006). Portland cement is considered the main source of alkalis. However, alkalis may be provided by other materials and/or sources including (i) some types of aggregates, (ii) supplementary cementing materials (e.g., slag and fly ash), (iii) external sources (such as seawater and de-icing salts) and (iv) chemical admixtures. Consequently, the preferred approach is to limit the total alkali content of the concrete when specifying alkali levels ((ICOLD, 1991) (USDT-FHA, 2012)). The appropriate use of supplementary cementing materials (SCMs), including pozzolans (e.g., calcined clay, shale, fly ash, or shales) and ground-granulated blast furnace slag, is another efficient means of controlling the total alkali content in the concrete (Lothenbach et al., 2011). The quantity of replacement varies depending on (i) the mineralogical and chemical composition (nature) of SCM, (ii) the type of aggregate, (iii) the availability of alkali within the concrete, and (iv) the exposure conditions of the concrete. In general, the proportion of SCM required to control deleterious ASR increases with the amount of available alkali and/or the more reactive the aggregate used in the concrete mix ((Tapas et al., 2021) (Thomas, 2011)).

Guidelines on the amount of SCM, depending on the type of SCM can be obtained in texts such as ICOLD (1991), Thomas et al. (2006), Thomas (2011) and USDT-FHA (2012).

McCoy and Cadldwell (1951) were among the first researchers to illustrate that various lithium salts (e.g., LiCl, Li₂CO₃, LiF, Li₂SiO₃, LiNO₃, and Li₂SO₄) could control deleterious expansion due to ASR in mortar and concrete, provided they are used at sufficient levels of replacement. There have been subsequent studies that agree with this earlier discovery (Feng et al., 2005). The effectiveness of lithium (Li) to control expansion is dependent on several factors including the form of lithium, the type of reactive aggregate used, and the availability of other alkalis (Na⁺ and K⁺) in the concrete (Feng et al., 2005). The most efficient form of Li for suppressing ASR is Lithium nitrate (LiNO₃) (Stokes et al., 1997).

2.5 Diagnosis of ASR

In the context of this discussion, diagnosis entails identifying the cause and nature of structural deterioration ((Yamada et al, 2014) (Ahmed, 2021)). There are numerous possible causes of deterioration and premature loss of serviceability of concrete infrastructure ((ICOLD, 1991) (USDT-FHA, 2010)). The correct identification of the cause of deterioration in concrete structures is critical to their assessment and management. Thus, every mechanism contributing to the observed deterioration must be considered, as a misdiagnosis may result in unsuitable/inefficient remedial actions (USDT-FHA, 2010). Complete diagnosis of ASR related damage to concrete structures involves multi-disciplinary assessments carried out systematically due to the similarity of ASR related external damage characteristics with those of other deterioration mechanisms such as shrinkage and/or frost attack (Godart et al., 2013).

Routine inspections are normally the starting point in ensuring the safety and reliability for any existing structure (Ahmed., 2021). These inspections often start with the review of available documentation relating to the structure, in which details about the structure such as date of completion, conditions to which the structure has been exposed, details on the mix design, observations from previous inspections, etc. can be ascertained (Godart et al., 2013). The features consistent with ASR damage to look out for during field inspection include extensive cracking; variable expansions observable through relative movements; displacements and deformations; surface discoloration along cracks; surface popouts caused by reaction of coarse aggregate adjacent to the surface. The cause of damage, its extent and distribution on the structure is often not fully established via field inspections. However, these

field inspections provide the basis on which comprehensive investigations involving the testing of core samples are established ((Godart et al., 2013) (Jensen, 2020 9th ICAAR)).

Diagnostic laboratory examinations include visual evaluations of core specimen, petrographic examination, Electron Probe Micro Analysis (EMPA), Scanning Electron Microscope (SEM), and other additional tests. Visual examination in which the pattern of cracking, location of cracking, gel in pores, reaction rims around aggregates observed using photography, core scanning and stereomicroscopy of core samples, provides an overview of the distribution of ASR which is important for deciding on the areas for thin section preparation (Godart et al., 2013). Thin section petrography which involves stereomicroscopy and investigation of thin sections of concrete, is used to characterise the microstructure and ASR related damage. In some cases, SEM used on fractured concrete provides enough information to confirm the presence of ASR. Where reactive rock types are complex, it may be necessary to investigate polished thin sections without a cover glass using SEM and EPMA. Other additional tests include point-counting, fluorescence microscopy of the polished thin-sections and X-Ray Diffraction (XRD) analysis ((Godart et al., 2013) (Saouma, 2021)).

2.6 Assessment of ASR related Damage and Expansion

After confirming ASR as the cause of deterioration, its extent and severity needs to be assessed. The investigation of the severity of ASR often runs in parallel with the diagnostic studies (Godart et al., 2013). Firstly, the current condition of the structure is assessed through visual inspection in which typical signs of ASR distress are reported including cracking (Ahmed, 2021). Measuring cracks provides the best approach to quantifying the extent of ASR damage. Additionally, measuring the physical and mechanical properties of concrete then gives insight as to the extent of damage within the concrete due to ASR (Godart et al., 2013). Lastly, expansion tests often used to provide essential information for prognosis, can also be carried out to investigate the residual ASR related expansion in a diagnostic sense (Saouma, 2021).

2.6.1 Measurement of cracks both in the field and in the laboratory

Several crack mapping techniques exist that quantify ASR related damage at both macroscopic scale and microscopic scale. The crack index is one crack mapping technique expressed as crack-width per measured length (i.e., mm/m) by measuring the crack width along pre-drawn lines (Figure 2.4) (Saouma, 2021). Another method involves the summation of crack widths along five parallel lines, then dividing by the length of lines drawn (Ahmed, 2021). At a

microscopic scale, the crack index method, adapted for thin concrete sections used for diagnosis of ASR, can be used to determine the extent of reaction. The crack index method can also be used together with a method that determines the number of reacted aggregates per thin section, to assess and compare the severity of ASR between different sections of a structure (Saouma, 2021). Damage Rating Index (DRI) is another semi-quantitative petrographic tool used at a microscopic scale to quantify the level of damage in thin concrete sections (Ahmed, 2021). Additionally, SEM can be combined with energy-dispersive X-ray spectroscopy (EDX) to assess and grade the extent of damage (Saouma, 2021).

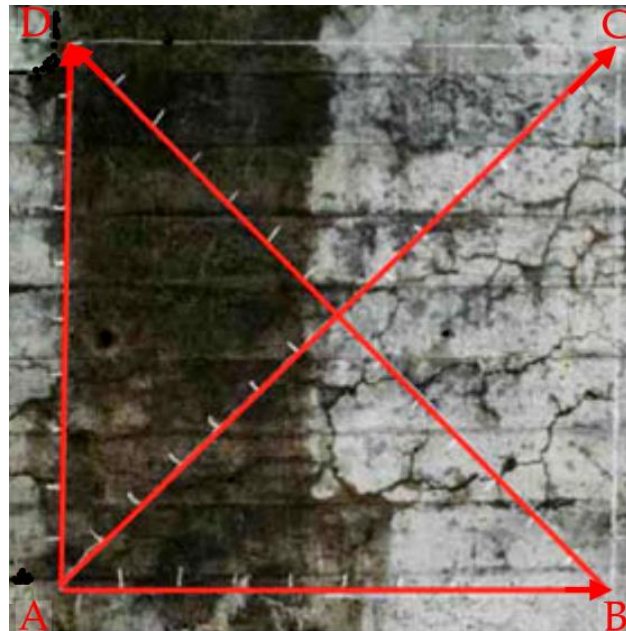


Figure 2.4: Illustration of the determination of the crack-index along the lines A-B, A-C, A-D, and B-C on the 1m x 1m square grid (adopted from Saouma, 2021)

For further details on these test methods, the reader is referred to Saouma (2021). Moreover, the above-mentioned tests are in no way the only available tests developed, other test procedures exist.

2.6.2 Measurement of Physical & Mechanical properties of concrete

As already indicated in the introduction to this section (section 2.6), the measurement of physical and mechanical properties of concrete can provide useful insight concerning the extent of ASR related damage within the structure (Godart et al., 2013). Material properties of concrete (i.e., compressive strength, tensile strength, and modulus of elasticity) are normally

used as a measure of concrete performance. Thus, any change and/or differences in properties between different sections of a structure or element with the same concrete mix design may indicate differences in severity of ASR (Godart et al., 2013) (Ahmed, 2021)). Stiffness Damage Test (SDT) is another test procedure that measures the extent of inner cracking in a concrete specimen with length-diameter ratio of 2.0 using five loading and unloading compressive cycles (Saouma, 2021). Non-Destructive Testing/Evaluation (NDT/E) approaches can be used to assess stiffness in-situ by measuring mechanical properties of the concrete. These tests include impact-echo (IE), ultrasonic pulse echo (UPE), ultrasonic pulse velocity (UPV), acoustic emission (AE), nonlinear ultrasounds, and surface waves techniques. Nonlinear ultrasounds allow for the determination of the extent of ASR-induced damage due to their sensitivity to changes in material properties of the concrete. However, this test method is not suitable for the assessment of concrete cores extracted from confined structures ((Godart et al., 2013) (Saouma, 2021)).

2.6.3 Expansion tests on concrete cores

These tests involve measuring the expansion of concrete cores stored in controlled environments over a period. These tests are usually performed at about 38 °C or at 20 °C (Godart et al., 2013). Different controlled storage conditions each with their unique challenges, have been implemented by different researchers including high humidity, controlled water supply, NaOH solution and NaCl solutions ((Godart et al., 2013) (Saouma, 2021)). The problem with high humidity storage conditions has to do with sustaining steady humidity and alkali leaching in long-term tests, which has led to inconsistent observations. To counter the problems associated with the high humidity test setups, an alternative approach that measures the expansion in relation to the quantity of absorbed water and reduces leaching of alkali by sealing the core in a tightly fitting, and tilted container, was developed ((Godart et al., 2013) (Saouma, 2021)). For the tests performed in NaOH, increased expansion and expansion rates have been observed in comparison to high humidity tests because placing the test specimen in alkaline solution overlooks the exhaustion of alkalis, which may occur in existing structures ((Godart et al., 2013) (Saouma, 2021)).

2.7 Prognosis of ASR

In the context of this discussion, prognosis involves estimating the potential for future expansion (residual expansion) and/or resulting further deterioration. Estimating the residual expansion requires the determination of two parameters namely, (i) the reaction kinetics (i.e., expansion rate) and (ii) residual expansion potential. These parameters are difficult to determine due to the dependency of the reaction on the concrete mix design and the influence of the laboratory testing exposure conditions on the estimate of these parameters ((Ahmed, 2021) (Saouma, 2021)). Numerous approaches that cover accelerated expansion laboratory tests have been developed to estimate these two parameters, for detailed and complete description of the methods, the reader is referred to Saouma (2021). The estimates of these parameters (latency time, characteristic time, and the asymptotic expansion value) are also important for the numerical analysis of the affected structure. Additional limitations of laboratory tests include the difficulty of achieving the exact conditions structures experience in service and other factors (mentioned in section 2.3) ((Ahmed, 2021) (Saouma, 2021)). Consequently, the direct interpretation of the expansion tests to predict expansion in structural members is often only achieved through modelling approaches which have been developed to describe the reaction mechanism and its effects at different levels (refer to section 2.9 for the modelling approaches of ASR). Thus, the assessment of further deterioration due to residual expansion is achieved through predictive assessment of the structural condition through numerical modelling approaches (Saouma, 2021).

2.8 Mitigation Measures of ASR affected Structures

The term mitigation is here used instead of “repair” (USDT-FHA, 2012). For concrete structures undergoing active deterioration such as with ASR affected structures, the probability of failure gradually increases as the deterioration of the structure progresses (Dolen, 2005). Additionally, there is no economically suitable way to stop the reaction once in progress for most large structures (Saouma, 2014). Thus, mitigation measures cannot and are not intended to restore a structure to its original integrity. Instead, they serve to minimise further expansion and its damaging effects (USDT-FHA, 2012). The mitigation measures that have been implemented on structures affected by ASR include (i) decreasing internal relative humidity, (ii) application of external restraint, and (iii) stress relief (USDT-FHA, 2012). These will be discussed briefly below:

2.8.1 Reducing Internal Relative Humidity

Applying any method that reduces the internal relative humidity (RH) is worthwhile since the availability of moisture is essential for deleterious ASR to occur (section 2.4). It is recognised that ASR-related expansion is substantially decreased below an RH of 80% (Pedneault, 1996). Some methods of reducing RH that have been implemented on structures include: (i) improving drainage, (ii) using external cladding to prevent absorption and/or ingress of further moisture. However, in some cases the moisture already in the concrete might be enough for the reaction to progress (USDT-FHA, 2012). Thus, the application of a coating or sealant to inhibit the ingress of water into the concrete but allows for moisture from within the concrete to exit resulting in substantial reduction of the internal RH is a more effective measure (Berube et al., 2002). Siloxanes and silanes are the most used types of coatings to this end. However, the use of coatings as a mitigation measure will most likely be ineffective for dam structures as its benefit is only realised from the treated surface (Charlwood & Sims, 2017). Medeiros & Helene (2008) highlighted that these coatings are not efficient when the concrete is exposed to water and thus cannot replace waterproofing. Whilst Johnson et al. (2014) also showed that the benefits of coatings might not be realised when elements are permanently exposed to moisture, as drying cycles are required for RH to decrease. Moreover, the implementation of coatings or sealants is ineffective in concrete with large crack widths. For larger cracks, flexible caulking or similar products should be used to seal larger cracks (USDT-FHA, 2012).

2.8.2 Application of External Restraint

It is widely accepted that compressive stress (i.e. greater than about 8 MPa) restricts or completely inhibits expansion in the direction of stress application ((Hayward et al., 1988) (Multon, 2004)). Confining a concrete specimen in one direction results in redistribution of the expansion in the other unrestrained directions. Consequently, expansion may be highly restrained under tri-axial compressive confinement (Saouma, 2014). Thus, physical restraint or confinement can considerably decrease ASR expansion in the constrained direction. This mitigation measure is unique in its approach as it affects the structural response. Consequently, a structural engineer must be involved in specifying and designing the method of application for the affected structure (USDT-FHA, 2012).

The external restraint can be applied in a few ways, including but not limited to: (i) post-tensioning, (ii) applying restraint around the affected element using non-reactive concrete,

applied stress or reinforcement ((Rotter, 1995) (Fournier et al., 2004)). Post-tensioned tendons or cables which are generally effective for thin arch dams and for bridge structures (Singhal & Nuss, 1991), are less effective for larger concrete structures and require periodic distressing (Rotter, 1995). Application of restraints around concrete elements can be achieved by wrapping them using Fibre-Reinforced Polymers (FRPs), reinforcing with straps, steel plates or tensioning using bolts ((Wood & Angus, 1995) (USDT-FHA, 2012)).

2.8.3 Stress Relief

ASR often results in a build-up of internal stress in the concrete. Slot cutting, or expansion joints are used as a means of stress relief and to provide space for further expansion (Charlwood & Solymar, 1995). Although this method of mitigation may result in an extension of the service life of a structure, it does not stop the reaction and therefore must be performed repeatedly as expansion continues ((USDT-FHA, 2012) (Thomas et al., 2013)).

2.9 Numerical Modelling Consideration of ASR

The evaluation of concrete structures subject to alkali-silica reaction (ASR) is complex as ASR-induced damage occurs from a microscale to macroscale (Esposito, 2015) and is affected by other factors already discussed above (section 2.3). Consequently, the assessment of these structures often requires the use of numerical models (as indicated in section 2.7) (Saouma, 2021). Numerous modelling strategies to investigate and predict ASR's effects on concrete structures have been proposed by different researchers. To properly assess structures affected by ASR, both the expansion and deterioration of mechanical properties must be considered (Esposito, 2015). This section provides a review of ASR modelling approaches and of the concrete material constitutive models focussing on the implementation of damage theories regarding ASR modelling as considered in this study:

2.9.1 State of the Art Modelling Approaches of ASR

Due to the multi-scale nature of ASR, many experts from different fields including geologists, material, and structural engineers, have studied this reaction, each with different focusses and aims (Esposito & Hendriks, 2019). Consequently, a wide range of modelling approaches have been developed over the years. Esposito (2015) provides a comprehensive review of these modelling approaches. A summary of the modelling approaches is discussed here for completeness. Table 2.1 provides a summary of the modelling approaches, the theories each

model employs, the input parameters that go into the expansion mechanism assumed by each model and the scope of the extent of application of the models that have been proposed. The modelling approaches in Table 2.1 are subdivided based on the level at which the input variables are specified. Models that impose ASR expansion at concrete level to investigate the structural behaviour constitute the first group of modelling techniques. Whilst models that seek to understand concrete behaviour by focussing on the gel expansion at aggregate level and the resulting effect of the internal pressure generated, form the second group of modelling techniques (Table 2.1). The third group of modelling techniques in Table 2.1 study ASR at reaction product level to combine the physical chemistry with the mechanical behaviour. Another group of modelling techniques that mimic the chemical mechanisms for studies at reaction products level constitute the fourth category in Table 2.1 (Esposito, 2015).

The earliest models investigated the effects of ASR on the structural behaviour by simulating the expansion due to ASR as an expansion strain at the concrete level. The progression of the reaction over time was estimated from phenomenological laws at first, then derived based on reaction kinetics laws according to thermodynamics concepts. The imposed expansion from these models could be coupled with a concrete damage criterion when performing a Finite Element Analysis (FEA) (Esposito, 2015). In the early phenomenological models such as those used by Malla & Wieland (1999), the concrete expansion was induced as a thermal strain and the analytical observations were validated using field displacement observations of the affected structures. Subsequently, phenomenological models that accounted for the influence of environmental conditions, the material stress state and reactivity of concrete on the induced expansion were developed based on extensive laboratory campaigns by researchers such as Swamy (1992), Charlwood (1994), Leger et al. (1996), Larive (1998) and Multon (2004).

Capra & Bournazel (1998) then accounted for the spatial distribution of the reaction development areas within structures by merging a phenomenological approach with a probabilistic cracking theory. In which, they evaluated ASR related damage using linear fracture mechanics theory for a numerical analysis adapted to evaluate free and restrained expansion tests on laboratory concrete samples. Following which, expansion models that express the free-expansion of concrete in terms of thermodynamic kinetics laws, commonly referred to as thermo-chemo-mechanical models, were developed from an idea proposed by Ulm et al. (2000) based on experimental investigations reported by Larive (1998). Ulm et al. (2000) derived a phenomenological sinusoidal law to describe the expansion progression of concrete characterised by three parameters: (i) characteristic time, (ii) latency time, and (iii) the

Table 2.1: Overview of modelling approaches for ASR in concrete (adopted from Esposito, 2015)

			Reaction Products Level	Aggregate Level	Concrete Level			Structural Level						
Input					Ions & water	Gel	Swelling	Damage	Swelling	Damage	Mech. Prop.	Lab specimens	Members	Structures
Scope														
Demonstration														
Experimental validation														
	Model type	Imposed strains dependent on	Category 1											
Charlwood (1994)	Phenomenological	stress state												
Leger at al. (1996)	Phenomenological with reduction of stiffness and tensile strength	stress state, temperature, moisture, reactivity												
Capra & Bournazel (1998)	Phenomenological with probabilistic fracture mechanics	stress state, temperature, moisture, reactivity												
Malla & Wieland (1999)	Phenomenological with linear-elastic fracture mechanics	-												
Ulm et al. (2000)	Thermo-chemo- plastic with kinetics law	temperature												
Li & Coussy (2002)	Thermo-chemo- plastic with kinetics law	temperature, moisture												
Capra & Sellier (2003)	Thermo-hygro- chemo-damage with kinetics law	temperature, moisture, reactivity												
Farage et al. (2004)	Thermo-chemo- cracking with kinetics law	temperature												
Bangert et al. (2004)	Thermo-hygro- chemo-damage based on mixture theory	temperature, moisture												
Saouma & Perotti (2006)	Thermo-hygro- chemo with kinetics law and reduction of stiffness and tensile strength	stress state, temperature, moisture												
Winnicki & Pietruszczak (2008)	Thermo-hygro- chemo-plastic for RC with kinetics law	stress state, temperature, moisture												

			Reaction Products Level	Aggregate Level	Concrete Level		Structural Level		
Input			Ions & water Gel	Swelling Damage	Swelling Damage Mech. Prop.	Lab specimens	Members	Structures	
Scope									
Demonstration									
Experimental validation									
	Model type	Imposed strains dependent on	Category 1						
Comi et al. (2009)	Thermo-chemo-damage with kinetics law	temperature							
Pesavento et al. (2012)	Thermo-hygro-chemo-damage based on mixture theory	temperature, moisture							
Esposito & Hendricks (2012)	Thermo-chemo-cracking rheological-based	temperature							
		Imposed	Category 2						
Bazant et al. (2000)	Linear-fracture mechanics theory	pressure in particle rim							
Dormieux et al. (2004)	Chemo-poro-micro-elastic	pressure in porosity							
Schlangen & Van Breugel (2005)	Lattice model with quasi-brittle aggregate, ITZ and matrix phase	strain in aggregate and/or ITZ							
Copuroglu & Schlangen (2007)	Lattice model with quasi-brittle aggregate, ITZ and matrix phase	strain in aggregate and/or ITZ							
Comby-Peyrot et al. (2009)	Particle model with linear-elastic aggregate phase and elastic damage matrix phase	strain at aggregates rim							
Reinhardt & Mielich (2011)	Linear-elastic fracture mechanics	pressure in aggregates							
Anac et al. (2012)	Lattice model with quasi-brittle aggregate, ITZ and matrix phase	strain in aggregate and/or ITZ							

			Reaction Products Level	Aggregate Level		Concrete Level		Structural Level	
Input			Ions & water	Gel	Swelling	Damage	Swelling	Damage	Mech. Prop.
Scope									
Demonstration									
Experimental validation									
	Model type	Imposed	Category 2						
Wu et al. (2014)	FE multiscale model	strain in porosity at the lower level							
Esposito (2015)	Micro-poro-fracture-mechanical	Pressure in porosity and strain in porosity and/or at aggregate rim and concrete level							
		Variation of gel	Category 3						
Ulm et al. (2002)	Chemo-micro-poro-plastic	mass							
Lemarchand (2001)	Chemo-micro-poro-elastic	mass							
Grimal et al. (2008)	Thermo-hygro-chemo-damage rheological-based	mass							
Dunant & Scrivener (2010)	XFEM with quasi-brittle aggregate and cement paste phases, elastic gel pockets	surface							
Giorla (2013)	XFEM with quasi-brittle visco-elastic cement paste phase, quasi-brittle aggregate, and elastic gel pockets	surface							
Pignatelli et al. (2013)	Chemo-micro-poro-damage	mass							
Charpin & Ehrlicher (2014)	Chemo-micro-poro-fracture	volume							

			Reaction Products Level	Ions & water	Gel	Aggregate Level		Concrete Level			Structural Level		
Input						Swelling	Damage	Swelling	Damage	Mech. Prop.	Lab specimens	Members	Structures
Scope													
Demonstration													
Experimental validation													
	Model type	Diffusion-reaction mechanism	Category 4										
Bazant & Steffens (2000)	Mathematical	dissolution silica, water diffusion within gel rim, water imbibition from porosity											
Suwito et al. (2002)	Chemo-micro-poro-elastic	diffusion alkali ions, diffusion of gel in cement paste											
Poyet et al. (2007)	Chemo-micro-poro-elastic	diffusion-reaction of sodium and calcium ions within gel rim											
Multon et al. (2009)	Chemo-micro-poro-damage	diffusion-reaction of sodium and calcium ions within gel rim											
Puatatsananon & Saouma (2013)	Computational multiscale	diffusion of alkali ions, diffusion of gel in cement paste											
Alnaggar et al. (2013)	Lattice discrete particle	dissolution silica, water diffusion within gel rim, water imbibition from porosity											
Liuaudat et al. (2014)	Mathematical	forward and backward diffusion-reaction of alkali and silica ions in the rim											
Nguyen et al. (2014)	Chemo-elastic	diffusion reaction of sodium ions											

asymptotic expansion value. The time values ((i) and (ii)) are dependent on temperature and the activation energies as defined by Arrhenius' concept. However, the imposed expansion does not consider the influence of the level of internal RH and the stress state of the material. The thermo-chemo-mechanical model developed by Ulm et al. (2000) was expanded and implemented in conjunction with damage models in existence by different researchers. For example, Li & Coussy (2002) expanded the model to include the influence of the moisture content on the imposed strain of the kinetics law and expressed the damage criterion using the plasticity theory for structural assessment. Whilst Capra & Sellier (2003) adopted a probabilistic approach to characterise the primary damage processes in structures, in which, the reaction kinetics was dependent on temperature, relative humidity, and the consumption of alkalis. Saouma & Perotti (2006) proposed an ASR constitutive model inspired by the thermo-chemo-mechanical approach suggested by Ulm et al. (2000) and the work by Larive (1998). In their model, they mainly focussed on the redistribution effect of the expansion caused by combining the internal expansion with the material stress state. They define the applied volumetric strain in terms of temperature, moisture and the induced material deterioration applied as a decrease in tensile strength and stiffness (Esposito, 2015).

Winnicki & Pietruszczak (2008) further defined the thermo-chemo-mechanical relation in the context of a thermo-hygro-chemo-plastic approach in which the concrete strain is dependent on the compressive stress. Whilst Bangert et al. (2004) and Pesavento et al. (2012) formulated a thermo-hygro-chemical coupling approach by linking the reaction kinetics with the physics of the ASR. They considered concrete as a porous material composed of multiple phases consisting of capillary porosity and a solid skeleton. They assessed the expansion of the alkali-silica gels as an applied strain on the concrete skeleton that depends on temperature and the moisture content (Esposito, 2015).

With advancements in microscopic investigation technologies, examination of the ASR processes at aggregate level to understand its effects at the concrete and structural level have been undertaken. Consequently, these investigations informed the development of models that assume an initial pressure when evaluating the ASR mechanism at the level of the aggregate. These proposed models can be classified based on three approaches: (i) mathematical approaches that characterise the damage process at aggregate level (Esposito, 2015). Research work under this classification includes the work by Bazant et al. (2000) and Reinhardt & Mielich (2011) who adopted a linear-fracture mechanics theory to evaluate the cracking mechanism in reactive aggregates, (ii) micro-poro-mechanical methods which blend

the concept of micro-mechanics, poro-mechanics, and the analytical homogenisation approaches to numerically correlate local and global parameters at aggregate, concrete or structural level. In this regard, the work by Dormieux et al. (2004) is among the first research work to treat concrete affected by ASR using the micro-poro-mechanics theory framework. In this approach the concrete is treated as a porous material with pores filled with the swelling reaction products and the concrete structure represented by a solid matrix, and (iii) analytical techniques that compute the damage level at the aggregate using FEM techniques. Examples include work by Schlagen & Van Breugel (2005) who assumed a lattice model to study the damage mechanism at aggregate level. Whilst Comby-Peyrot et al. (2009) used a 3D FE model in which they represented the aggregates as particles and Wu et al. (2014) developed a multiscale material model that accounts for the different mechanisms from reaction products level to concrete level (Esposito, 2015).

Numerous numerical techniques have been developed to evaluate ASR at reaction products level whilst coupling the chemistry of the reaction with the mechanical behaviour of the concrete (Esposito, 2015). Ulm et al. (2002) used the micro-poro-mechanical theory framework to investigate the link between the expansion of the reaction products in the aggregate and the resulting swelling of the concrete. While Lemarchand (2001) applied a chemo-poro-elastic theory to evaluate the type of free expansion behaviour of concrete through two mechanisms: (i) the topochemical mechanism in which the pores are uniformly filled with the reaction product, and (ii) the through-solution mechanism in which the free expansion due to ASR is described using the development of cracks around the aggregates. Charpin (2013) employed a chemo-micro-poro-fracture model to investigate induced anisotropic expansion behaviour of confined concrete specimens. Whilst Dunant & Scrivener (2010) analysed the aggregate level damage due to ASR through a combined use of computational methods and microscopic observations. Pignatelli et al. (2013) expanded the work by Comi et al. (2009) to investigate the correlation between the development of reaction gel product and aggregate level behaviour (Esposito, 2015).

Most recently, research work has been carried out to investigate the chemical mechanisms at the reaction products level particularly the ion dissolution-reaction mechanisms. This has led to the development of mathematical techniques implemented in analytical models to describe the mechanical consequences of ASR in concrete. For example, Bazant & Steffens (2000) developed a mathematical model on this basis to describe the reaction kinetics of ASR affected concrete specimens. Whilst Suwito et al (2002) adopted a micro-poro-mechanical model to

investigate the pessimum aggregate size effect. Poyet et al. (2007) implemented a model that accounts for (i) the migration of sodium and calcium ions within the aggregate and (ii) the equilibrium of their concentrations outside the aggregate to describe the reaction kinetics in the case of spherical aggregates. Nguyen et al. (2014) linked the diffusion of alkali ions to ASR expansion in the concrete. Whilst Liuaudat et al. (2014) used a diffusion-reaction model to study a single interface system mechanism between the aggregate and the cement paste.

2.9.2 Material Behaviour of Concrete

It is observable from the summary of the different proposed ASR modelling approaches (section 2.9.1) that different researchers defined or characterised the damage due to ASR in concrete using different theories. An understanding of the behaviour of concrete is crucial in the selection of the appropriate material constitutive model that will capture any damage of the concrete accurately.

Concrete is an anisotropic material whose compressive strength is significantly greater than its tensile strength, characterised by brittle failure. In cases where stresses are low or where the nonlinear behaviour of a structure is not crucial, concrete may be assumed to behave linear elastically. The stress-strain relation of concrete is generally dependent on the development of microcracks in the concrete matrix (Enzell & Tollsten, 2017).

Uniaxial Compressive Behaviour

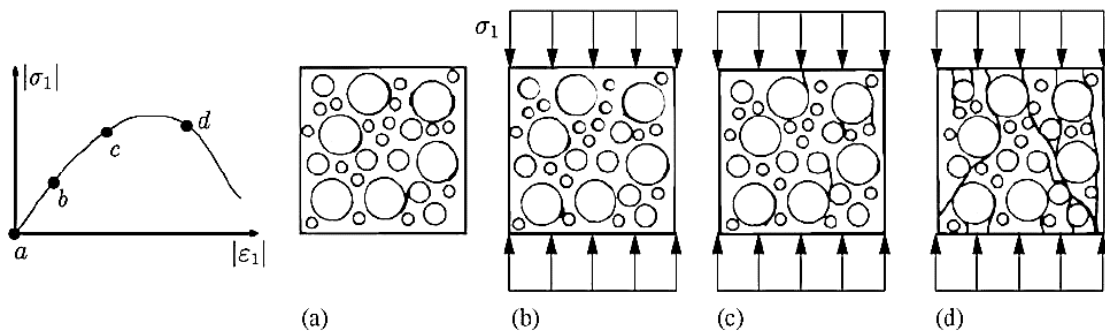


Figure 2.5: Compressive stress-strain relationship and crushing of concrete (adopted from Mang et al., 2003)

Figure 2.5 illustrates the uniaxial compressive behaviour of concrete. The development of micro-cracks in the concrete matrix are as a result of tensile strains perpendicular to the loading direction. Concrete exhibits a linear stress-strain relationship up to about 40% (Figure 2.5b) of the compressive strength of concrete, around which, micro-cracks start to develop (Figure 2.5c) and the stress-strain relationship starts to have a gradual curvature as the micro-

cracks propagate until the compressive strength is reached (Figure 2.5d). Beyond the compressive strength, strain-softening starts and continues as the stress drops till the concrete fails (Enzell & Tollsten, 2017).

Uniaxial Tensile Behaviour

Figure 2.6 illustrates the uniaxial tensile behaviour of concrete. In this instance, concrete exhibits a linear stress-strain relationship up to about 60% of the tensile strength of concrete (Figure 2.6b) after which, the concrete stiffness decreases until the tensile strength is reached (Figure 2.6c). At the tensile strength (Figure 2.6d), the micro-cracks propagate forming continuous cracks causing the concrete to lose its load-bearing capacity (Enzell & Tollsten, 2017).

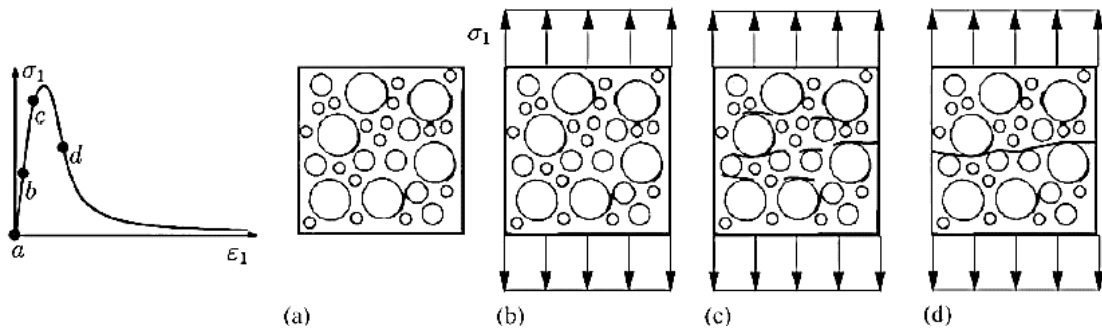


Figure 2.6: Development of the tensile failure of concrete (adopted from Mang et al., 2003)

In design, the tensile load-bearing capacity contribution of cracked concrete is normally ignored. However, this assumption is often insufficient when assessing the actual behaviour of structures. To this end, fracture energy and crack-opening curves are normally used to describe the cracked tensile behaviour of concrete (Malm, 2016). Fracture energy relates to the amount of energy needed for a crack unit area to open till it is stress-free (Karihaloo, 2003). Fracture energy depends on several factors including the water/cement ratio, aggregate size, concrete age, curing conditions, and the size of the structure (fib, 2010). On the other hand, crack-opening curves describe the residual tensile stress as the crack propagates in the concrete. The area under the crack-opening curve corresponds with the fracture energy. There are three crack-opening curves that are most implemented namely: linear, bilinear, and exponential curves (Malm, 2009). Of these three, the exponential crack-opening curve best describes the real unloading behaviour of concrete. For FEA it is recommended that the crack-opening curve be implemented based on the displacements rather than the strains so that the

fracture energy does not vary depending on the mesh size ((Malm, 2016)) (Enzell & Tollsten, 2017)).

Multiaxial Behaviour

Figure 2.7 illustrates the biaxial loading failure envelope for concrete. The development of cracks under multiaxial loading depends on the principal stresses. The compression behaviour of concrete under multiaxial loading differs from its uniaxial loading behaviour. The compressive strength under biaxial compression of the concrete increases by about 16% due to partial confinement of concrete and is considerably higher under triaxial compression. Whilst the tensile behaviour under biaxial and triaxial tension of concrete remains unaffected ((Malm, 2016) (Enzell & Tollsten, 2017)).

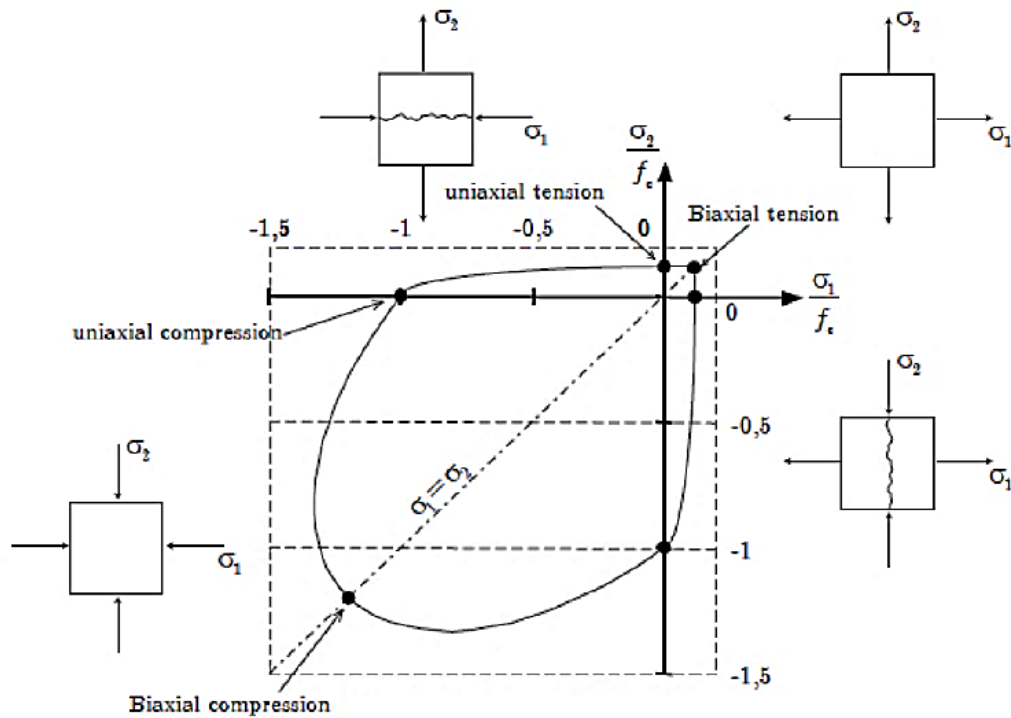


Figure 2.7: Biaxial failure envelope (adopted from Malm, 2006)

2.9.3 Non-linear Material Models for Concrete

Cracks can be modelled using two different approaches, namely: (i) discrete crack models, and (ii) smeared crack models (Figure 2.8) (Malm, 2006). In discrete cracking models, a physical crack is generated by defining a crack plane, assuming linear elastic behaviour for the uncracked sections of the structure. Whilst in smeared crack models, the crack is apportioned across an element with the failure, which happens when the tensile or compressive strength is

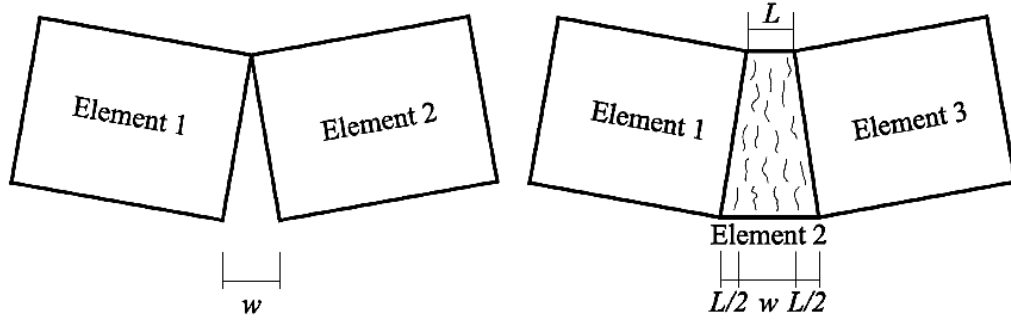


Figure 2.8: A discrete and a smeared crack modelling approach illustration (adopted from Malm, 2006)

reached, represented by a reduction in stiffness. Additionally, smeared crack models do not require the location of the crack to be known prior to the analysis. Thus, cracks can initiate anywhere in the structure (Enzell & Tollsten, 2017). Consequently, this research will focus on smeared crack modelling techniques.

There are three basic theories which are the basis of smeared crack models, namely: (i) fracture mechanics theory, (ii) damage theory, and (iii) plasticity theory (Malm, 2016). A brief description of the last two theories here follows:

Damage Theory

In damage theory, a damage parameter (d) that represents the amount of damage in the material is introduced to reduce the stiffness with increasing damage i.e., for uncracked concrete the damage parameter is zero whilst for completely cracked concrete it is one (Malm, 2016). The size of the damage parameter is defined with a failure criterion. If one damage parameter is used, the material is idealised as an isotropic material. To represent anisotropic behaviour of a material, two different damage parameters may be used. For example, two damage parameters d_t for tension and d_c for compression may be used to represent the anisotropic behaviour of concrete. In this theory, there is no permanent deformation after loading (Enzell & Tollsten, 2017). The uniaxial damage evolution is expressed using equation 2.1 and illustrated in Figure 2.9.

$$[K]^s = (1-d) * [K]^A \quad (2.1)$$

where $[K]^s$ is the secant stiffness matrix and $[K]^A$ is the initial stiffness matrix.

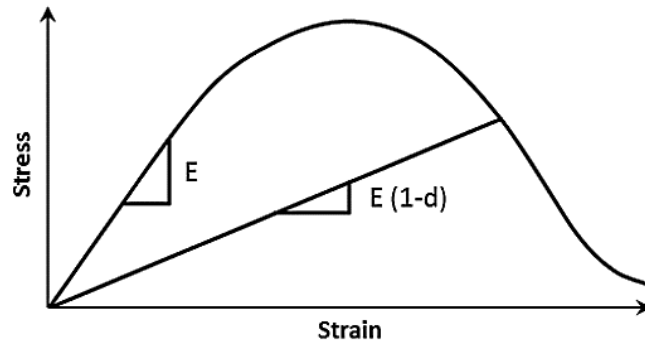


Figure 2.9: Uniaxial damage evolution (adopted from Enzell & Tollsten, 2017)

Plasticity Theory

Plasticity theory is usually implemented for the analysis of ductile materials such as steel. Thus, to analyse a quasi-brittle material such as concrete using this theory, the flow theory of plasticity that assumes small strains and correlates stresses to stress rates instead of total strain, is typically employed instead of the total deformation theory (Karihaloo, 2003). The strain consists of two components namely: (i) elastic strain (ϵ_{el}), and (ii) plastic strain (ϵ_{pl}). The material is assumed to behave plastically beyond its elastic loading limit (Enzell & Tollsten, 2017).

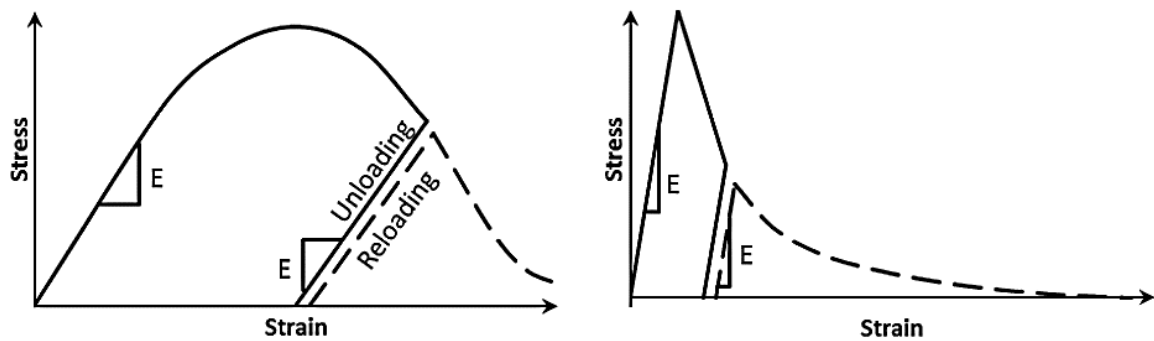


Figure 2.10: Uniaxial plastic stress-strain relationship for concrete (adopted from Enzell & Tollsten, 2017)

Figure 2.10 illustrates the typical uniaxial plastic stress-strain relationship for concrete which shows that the elastic strain is recovered when the material is unloaded, and the material follows the unloading track when the material is reloaded i.e., the material behaviour resumes from the point in the stress-strain relationship it was last unloaded from. In compression, concrete is assumed to behave linear elastically up to 60% of the compressive strength, after which the concrete undergoes strain hardening until the compressive strength is reached, beyond which, the material softens until the failure. In tension, concrete is assumed to behave

linear elastically until it reaches the tensile strength after which it begins to soften till complete failure (Enzell & Tollsten, 2017). Based on the yield, hardening, and softening of the concrete material in its stress-strain relationship, yield surfaces, hardening and the flow rule are defined to accurately represent the assumed material behaviour of concrete when assessing structures through the Finite Element Method.

Yield surface - in a multiaxial system, the yield surface in the stress-space is used to define the elastic limit. The Drucker-Prager and the Mohr-Coulomb yield functions are the most used yield surface models for concrete (Malm, 2009).

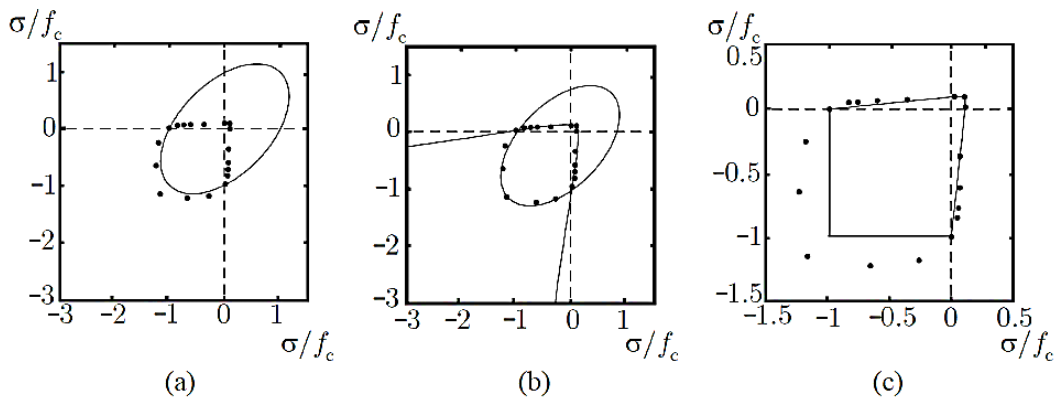


Figure 2.11: (a) Von Mises, (b) Drucker-Prager, and (c) Mohr-Coulomb biaxial failure criterion in biaxial loading (adopted from Malm, 2006)

Hardening - is controlled by a history-dependent hardening parameter (κ) which describes the evolution of the plastic behaviour, which is itself dependent on the plastic strain rate ($\dot{\epsilon}_{pl}$) (equation 2.2) of the material (Enzell & Tollsten, 2017):

$$\dot{\kappa} = f(\dot{\epsilon}_{pl}) \quad (2.2).$$

Flow rule - describes the material plastic deformation by coupling the stress-strain relationship and the yield surface. Equation 2.3 describes the flow rule expressed in terms of the plastic strain rate ($\dot{\epsilon}_{pl}$) (Karihaloo, 2003):

$$\dot{\epsilon}_{pl} = \dot{\kappa} \frac{\delta G}{\delta \sigma} \quad (2.3)$$

where $\frac{\delta G}{\delta \sigma}$ is the derivative of the plastic potential function.

Damage-Coupled Plasticity

Tensile and compressive failure in concrete is characterised by irreversible displacements and the degradation of stiffness. The plasticity theory does not account for the reduction of stiffness whilst the damage theory does not result in permanent displacements. Thus, a damaged-coupled plasticity theory combines these two approaches to develop a material behaviour representative of concrete (Figure 2.12) (Gasch, 2016).

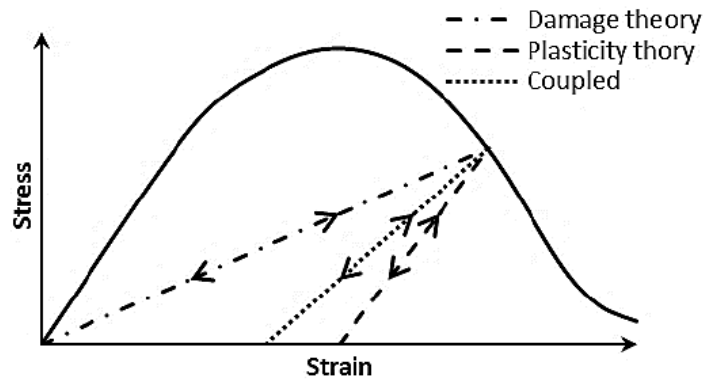


Figure 2.12: Comparison between the plasticity theory, damage theory and the damaged-coupled plasticity (adopted from Gasch, 2016)

2.10 Structural Integrity and Performance Assessment of Arch Dams Affected by ASR

The essential building blocks for the performance assessment of concrete arch dams have already been discussed in the preceding sections of this chapter. In this section, the process by which concrete arch dams affected by ASR maybe assessed is here summarised: Visual observations, instrumentation field records, laboratory concrete testing results combined with structural analysis results all provide crucial information towards studying the past behaviour, assessing the current structural integrity, and predicting the future performance of a concrete dam (Saouma, 2021).

2.10.1 Dam Surveillance

Dam surveillance allows for continual awareness of a dam's condition throughout its service life achieved through visual inspection and instrumentation (section 1.1). Visual inspections involve ocular field examinations carried out regularly by qualified persons who understand dam operations to be able to detect any deviation from normal behaviour, damage such as

cracks and/or deterioration such as that due to ASR. Whilst instrumented monitoring of dams is used to check the behaviour of the dam by measuring certain physical parameters and to monitor their development (Bukanya, 2020). A well-structured dam surveillance program that meets its objectives of achieving precise and timely diagnosis of the behaviour of a dam provides early alerts that allow for mitigation against any undesirable consequences such as dam failure ((ICOLD, 2009) (ICOLD, 2018) (Bukanya, 2020)). In the context of this discussion, the first indications of concrete swelling due to ASR are normally observed and recorded through the dam surveillance programme such as relative movements between blocks at the dam crest, general observable concrete deterioration, spalling along contraction joints and cracking in the dam galleries among other observations (Saouma, 2021). Following which, proper diagnosis and appraisal of the structure may follow and/or integrated into the surveillance programme to monitor the progression of the reaction. Some methods that have been used to monitor concrete dams affected by ASR include unmanned aerial systems (UAS) photogrammetry, monitoring of equipment operations, monitoring dam displacements, recording of temperature measurements, moisture monitoring, reservoir water level monitoring, crack mapping, non-destructive geophysical surveys, and concrete coring (Saouma, 2021).

2.10.2 Laboratory Testing

Laboratory based assessments are essential for the confirmation of ASR (diagnosis), to investigate the severity of the ASR related damage (appraisal of the dam structure) and predict the potential for future expansion (prognosis) in dam structures. Laboratory tests carried out for diagnosis purposes include visual examination of core samples, petrographic examination, Electron Probe Micro Analysis (EMPA), Scanning Electron Microscope (SEM), and other additional tests (see section 2.5). Whilst the measurement of physical and mechanical concrete properties can provide useful information on the extent of damage within the concrete due to ASR (see section 2.6). On the other hand, accelerated expansion tests provide essential information to evaluate the potential for future expansion.

2.10.3 Numerical Structural Analysis

Numerical structural analysis is a critical component in the assessment of concrete dam structures affected by ASR as it allows engineers to evaluate the current state of the structure

and to predict the future state with the input from field and laboratory measurements. Direct interpretation of the expansion tests in the context of predicted expansion in structural members is often only achieved through modelling approaches developed to describe the reaction mechanism and its effects at different levels. Assessment of further deterioration due to residual expansion is achieved through predictive assessment of the structural condition through numerical modelling approaches (section 2.7) ((Ahmed, 2021) (Saouma, 2021)). Additionally, numerical models may also be used to assess different remedial methods and interventions (Charlwood & Sims, 2017).

2.10.4 Mitigation Measures of ASR Affected Dam Structures

There are essentially two approaches used in the management of ASR affected dam structures, namely: (i) attempting to control the rate and duration of expansion of ASR and (ii) attempting to extend the service life of the dam by treating the consequences of ASR expansion. Regarding the former approach, different options have been implemented such as adding coatings or membranes to the upstream face, installation of post-tensioned anchors and the use of chemical means such as lithium salts, without much success in controlling ASR mainly due to the massive size of dam structures (Charlwood & Sims, 2017). Whereas for the later approach where the objective is to maximise the remaining economic service life of the structure, several effective strategies have been successfully implemented. These include the use of anchors (post-tensioned anchors) and grouting, which have been found to be the most effective approaches towards maintaining the stability and integrity of concrete dam structures. Slot cutting has also been effectively used in many cases to control stresses, equipment interferences, and concrete cracking (Charlwood & Sims, 2017). Since, in this approach ASR continues unabated, repeated interventions may be required. However, there may be a limit to sustainable life extension if the expansion behaviour continues indefinitely. Comprehensive continuous monitoring and modelling is therefore crucial in understanding the status and forecasting the future requirements throughout the slot cutting approach. In some cases, dam structures have been decommissioning and/or replacement (Charlwood & Sims, 2017).

2.11 Chapter Summary

This chapter provides a brief review, of ASR, its structural implication, assessment, and management of concrete infrastructure. Additionally, a crucial aspect in the assessment and management of ASR in concrete structures, numerical modelling, is reviewed. The chapter concludes by providing a summarised contextualisation of the reviewed aspects in this chapter in the assessment and management of concrete dam structures.

The key issues to highlight from this review include the fact that the assessment of concrete structures affected by alkali-silica reaction (ASR) is a complex problem due to the multiscale nature of ASR (i.e., ASR-induced damage occurs from the aggregate up to the structural level of concrete). Also, the damaging process of ASR-affected concrete is strongly correlated to its function within a structure and depends on the stress state of the structure. Consequently, structural assessment reliant on structural testing rather than typical material type testing on cores extracted from the structure would be the most favourable for the evaluation of structural impacts due to ASR. Generally, stiffness is the most sensitive to ASR related deterioration. Additionally, it is important to understand the development of ASR, its structural implications and stress state, to apply the best ASR modelling approach. Lastly, ASR often results in a build-up of internal stress in the concrete.

Typically, ASR evaluation techniques (including the constitutive modelling approaches) discussed in this section seem to be largely applied to assess the impact of ASR on the static behaviour of the affected structures. Given that there is a need for the development of field-based structural evaluation approaches of ASR affected concrete, and that stiffness is the most sensitive to ASR damage; it could be worth investigating the effect of ASR on the dynamic behaviour of concrete arch dams via numerical model assessment and/or ambient vibration testing/monitoring.

Additionally, ASR results in a build-up of internal stress in structures resulting in internally loaded structures. Another aspect of the assessment of the impact of ASR on the structural performance of dams that remains a challenge is the determination of the change in wall stiffness of the dam due to internal stresses induced by ASR. In this study, the assessment of the impact of stress induced stiffness due to ASR expansion on the dynamic behaviour of concrete arch dams is investigated to meet the objectives of this study. To this end, the chapter that follows deals with the modelling consideration of loaded structures to account for stress-stiffness in their assessment.

2.12 References

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Chapter 3

FINITE ELEMENT MODEL UPDATING OF AXIALLY LOADED STRUCTURES

3.1 Introduction

In the previous chapter (Chapter 2) it was observed that ASR may lead to the development of internal stresses which can affect the stiffness of a structure. Finite element models provide an excellent basis for the evaluation of internal stresses induced by ASR. However, the link between finite element modelling results and the current state of a structure is often missing for several reasons some of which were indicated in Chapter 2 including the potential development of stress-stiffness due to ASR expansion. The change in stiffness due to internal forces is referred to as stress stiffening or initial stress stiffness or geometric stiffness ((Cook et al., 2002) (Clough & Penzien, 2003)). For concrete dams affected by ASR it is important to understand the influence of the ASR generated forces on the stiffness of the dam in order to accurately calibrate numerical models. Consequently, this chapter reviews the model updating approaches of loaded structures (i.e., structures subject to axial loading) with particular focus on stress-stiffness updating to inform the evaluation of the impact of stress-stiffness on the dynamic behaviour of concrete arch dams to meet the objectives of this study.

The following sections explore the state of the art regarding the use of model updating approaches in practical situations focussing on loaded structures. The concept of geometric stiffness is illustrated by considering a simple beam element. However, it is not the intention of this section to provide an all-inclusive review of the field, for this, the reader can consult texts such as those by Imregun & Visser (1991), Mottershead & Friswell (1993), Friswell & Mottershead (1995) and more recently by Ereiz et al. (2022). Additionally, there is limited technical information on the dynamic behaviour assessment of dams affected by ASR.

Therefore, the chapter ends by highlighting some experiences of the use of vibration-based assessment of and the application of geometric stiffness in the assessment of loaded structures.

3.2 Finite Element Model Updating (FEMU)

The aim of finite element model updating (FEMU) is to calibrate an FE model predicated on the operational behaviour of the structure observed through static and/or dynamic testing/monitoring, to improve the accuracy and reliability of the FE model. Consequently, the FEMU problem is described as the difference between predicted and the actual structural behaviour (Friswell & Mottershead, 1995). FEMU methods can be classified as manual or automated methods. Generally, manual methods are used when there are a few parameters to update, whereas automated methods are recommended in cases where more parameters are considered. Often, a combination of manual and automated methods is usually used to achieve updating of the model whereby: manual updating is employed to bring the initial analytical model as close to the observed behaviour as possible. Then automated updating is performed to further fine tune the estimates of the unknown parameters to obtain more reliable estimates. This approach often results in better results and improved computational time ((Feng et al., 2016) (Altunisik et al., 2018) (Bianconi et al., 2020)).

Generally, FEMU methods are more commonly classified as non-iterative (direct) and iterative (indirect) methods, which are briefly discussed next:

3.2.1 Direct Methods

Direct methods constitute the oldest methods of model updating that update global system matrices (stiffness matrix and mass matrix) in a single step, reproducing accurately the experimentally observed behaviour (based on the experimental data used in updating). However, the ability of the model to accurately predict the structure's response for other loadings or test configurations decreases and/or cannot be guaranteed as the model's physical parameters are not directly adjusted during the updating process when using these methods. Consequently, the use of direct methods in model updating of structures has decreased despite their computational efficiency ((Ewins, 2000) (Baruch, 1978) (Heo & Jeon, 2016)). Examples of direct methods of FEMU include the Lagrange multiplier method and matrix update methods among others (Asma, 2019) (Jull et al., 2019).

3.2.2 Iterative Methods

Iterative FEMU techniques are the most used updating methods that seek to enhance the correlation of the observed/measured response and the predicted FE model response by adjusting the physical structural parameters of each element or substructure iteratively (to find the global optimum) to produce models that are physically realistic ((Lee, 2006) (Ereiz et al., 2022)). To this end, the FEMU problem is defined as an optimisation or statistical problem (depending on the updating method used). Consequently, iterative FEMU methods can be further classified as deterministic and stochastic methods based on the updating method used.

Iterative deterministic methods

In practical engineering applications, the model-updating problem is converted into an optimisation problem by defining an objective function in terms of residuals between numerically derived and experimentally derived data groups (which includes both dynamic and static data). The weighted sum-of-square residual, E_r , is the most common choice of objective function (equation (3.1)):

$$E_r = \mathbf{r}^T \mathbf{W}_r \mathbf{r} \quad (3.1)$$

$$\mathbf{r}(\theta) = \tilde{\mathbf{z}} - \mathbf{z}(\theta) \quad (3.2)$$

where $\mathbf{r}(\theta)$, is the residual column vector of m model outputs in terms of p updating parameters, θ ; $\tilde{\mathbf{z}}$ is a column vector with m components of the measured data and $\mathbf{z}(\theta)$ the corresponding column vector of m model outputs. $\mathbf{W}_r$ is the residual weighting matrix given as:

$$\mathbf{W}_r = \mathbf{C}_z^{-1} \quad (3.3)$$

where $\mathbf{C}_z$, is the covariance matrix of $\tilde{\mathbf{z}}$.

If $\mathbf{W}_r$ is diagonal and decomposable into a scalar multiple of $\mathbf{I}$ (identity matrix), then the sum-of-square residual, E_r , can be written as:

$$E_r = \sum_{i=1}^m \mathbf{r}_i^T \mathbf{w}_i \mathbf{r}_i = \sum_{i=1}^m w_i \mathbf{r}_i(\theta)^2 = \sum_{i=1}^m w_i \|\tilde{z}_i - z_i(\theta)\|_2^2 \quad (3.4)$$

(Bartilson et al., 2020).

In most cases, dynamic properties (i.e., natural frequencies and mode shapes) are used as the first measure of the current state of behaviour of a structure as any change in the structure and

its properties will result in a change in structural properties (i.e., stiffness and/or mass) and hence a change in its dynamic properties (Ereiz et al., 2022). The widely used iterative deterministic methods which use dynamic properties include those that use eigenvalue residuals, eigenvector residuals, frequency response function (FRF) residuals, modal flexibility residuals and their combinations among others ((Jaishi & Ren, 2005) (Ereiz et al., 2022)). Static data such as displacements and strain data have also been successfully used for FEMU ((Sanayei et al., 1997) (Liao et al., 2012) (Tchemodanova et al., 2021)). Additionally, both static and dynamic data sets have also been used in combination to perform model updating ((Sanayei et al., 2006) (Sun & Xu., 2020)). Two approaches are generally used in performing model updating when using disparate data sets namely: (i) the single objective approach and (ii) the multi objective approach.

For the single objective approach, the weighted sum-of-square residual, E_r , is defined as a function of the weighted residuals for the different data sets (equation (3.5)). The optimal weighting factor values are determined through trial and error or by statistical criteria based on how easily and accurately each experimental data set is obtainable and/or based on the sensitivity of the experimental data set to changes of the updating parameters under consideration. It is assumed that the optimum values of the weighting factors lie between 0 and 1 ((Pacheco et al., 2017) (Kim et al., 2019) (He et al., 2020)):

$$E_r = \mathbf{r}^T \mathbf{W}_r \mathbf{r} = \begin{bmatrix} \mathbf{r}_a^T & \mathbf{r}_b^T \end{bmatrix} \begin{bmatrix} \mathbf{W}_r^a & \\ & \mathbf{W}_r^b \end{bmatrix} \begin{bmatrix} \mathbf{r}_a \\ \mathbf{r}_b \end{bmatrix} = \mathbf{r}_a^T \mathbf{W}_r^a \mathbf{r}_a + \mathbf{r}_b^T \mathbf{W}_r^b \mathbf{r}_b \quad (3.5)$$

where sub or super scripts a & b are used to differentiate the disparate data sources (Bartilson et al., 2020).

Whilst, for the multi objective approach, different objective functions per data set are used in conjunction, in the model updating ((Osyczka, 1978) (Sun et al., 2020)). Thus, for the same data set for which equation (3.5) was defined, two independent objective functions will be defined as in equation (3.6) and (3.7). In this approach, the aim is to optimally find good trade-offs between any clashing objective functions. This is achieved by defining a reasonable criterion. The most used criterion seeks a solution for which a small improvement in one objective function leads to large worsening in at least one other objective function (Branke et al., 2004). Based on the research papers that have reported on the comparison of the single objective and multi objective function approaches in model updating, the latter approach shows better performance ((Jin et al., 2014) (Ereiz et al., 2022)):

$$\mathbf{E}_r^a = \mathbf{r}_a^T \mathbf{W}_r^a \mathbf{r}_a \quad (3.6)$$

$$\mathbf{E}_r^b = \mathbf{r}_b^T \mathbf{W}_r^b \mathbf{r}_b \quad (3.7)$$

(Bartilson et al., 2020).

Iterative stochastic methods

To update the FE model using the iterative stochastic approach, the updating problem is viewed as a statistical problem that includes an assessment of the effect of uncertainty using uncertainty quantification (UQ) methods on both the calibrated model parameters and on predicted parameters such as response predictions ((Ereiz et al., 2022) (Simoen et al., 2014)). When updating FE models, there are two possible sources of uncertainty, (i) pertaining to the FE model and, (ii) pertaining to the empirical data (Ereiz et al., 2022). The uncertainties associated with the FE model have been discussed in section 3.2. Sources of uncertainty/error associated with experimentally obtained data include random measurement noise, bias or systematic error resulting from defects and/or limitations of the measuring equipment or setup, signal processing and the problem of post processing the measurement data, in general ((Ereiz et al., 2022) (Simoen et al., 2014)).

The modelling error (ϵ_{model}) can be defined as the difference between the model predictions and the true/real structural system (equation (3.8)):

$$\epsilon_{\text{model}} = \mathbf{z} - \mathbf{z}(\theta) \quad (3.8)$$

where ϵ_{model} , is the column vector of model uncertainty, $\mathbf{z}$ is a column vector with m components of the true system output and $\mathbf{z}(\theta)$ the corresponding column vector of m predicted model outputs.

Whilst the measurement error (associated with the experimentally obtained data) can be defined as the difference between the observed structural behaviour and the true/real structural response (equation (3.9)):

$$\epsilon_{\text{meas}} = \tilde{\mathbf{z}} - \mathbf{z} \quad (3.9)$$

where ϵ_{meas} , is the column vector of measurement uncertainty, $\mathbf{z}$ is a column vector with m components of the true system response and $\tilde{\mathbf{z}}$ the column vector with m components of the measured data (i.e., measured structural response).

Eliminating the (unknown) true system output from equation (3.8) and equation (3.9) results in equation (3.10), which illustrates that the model prediction corrected by the modelling error is equal to the experimentally obtained data corrected by the measurement error:

$$\tilde{\mathbf{z}} - \boldsymbol{\epsilon}_{\text{meas}} = \mathbf{z}(\boldsymbol{\theta}) + \boldsymbol{\epsilon}_{\text{model}} \quad (3.10).$$

Equation (3.2) can therefore be rewritten as follows:

$$\mathbf{r}(\boldsymbol{\theta}) = \tilde{\mathbf{z}} - \mathbf{z}(\boldsymbol{\theta}) = \boldsymbol{\epsilon}_{\text{model}} + \boldsymbol{\epsilon}_{\text{meas}} = \boldsymbol{\epsilon} \quad (3.11)$$

where $\boldsymbol{\epsilon}$, is the column vector of the total observed prediction error (Simoen et al., 2014).

UQ methods naturally reflect both model and measurement uncertainties in their results and can generally be classified into two main categories, probabilistic and non-probabilistic approaches ((Bartilson et al., 2020) (Simoen et al., 2014)).

UQ in science and engineering has gained substantial interest over the recent decades resulting in the development of a wide array of analysis methods and classification systems. In general, probabilistic UQ methods are based on the classical approach to modelling uncertainty using probability theory in which uncertainty of model updating parameters and model outputs is modelled using probability density functions (PDF) ((Ereiz et al., 2022) (Bartilson et al., 2020) (Simoen et al., 2014)). Whereas non-probabilistic UQ method are mainly based on interval analysis in which a range of values (intervals) for parameters and model outputs are estimated based on the upper and lower bounds of measurement data (Bartilson et al., 2020). For a comprehensive review of the field, the reader is directed to review papers by Simoen et al. (2014) and Ereiz et al. (2022) among others.

As highlighted earlier in this section, iterative FEMU are essentially predicated on solving an optimization problem. Usually, computational optimization algorithms are used to solve the problem. Thus, FEMU can be achieved using artificial intelligence techniques. For more information and application of intelligent algorithms, the reader is referred to texts by Giagopoulos et al. (2019), Marwala et al. (2017) etc.

3.2.3 Selection of the updating parameters

The choice of suitable parameters of the FE model to update during model updating is the most important activity (Friswell & Mottershead, 1995). The chosen model parameters should embody the unknown structural properties and should be uncertain in the model to produce

updated models that are physically realistic ((Marwala, 2010) (Brownjohn & Xia, 2000)). The number of parameters (from the many parameters that could be considered such as material properties, geometric properties, boundary conditions, etc.) is also important. Trying to determine too many parameters often results in an ill-conditioned or arbitrary problem due to insufficient measured data to solve it (Brownjohn & Xia, 2000). Additionally, an insufficient amount of measured data results in constrained parameter estimation leading to a system of equations that is underdetermined in deterministic methods or result unsolved for parameters in stochastic iterative methods (Ereiz et al., 2022).

It is essential to choose updating parameters that are most useful to define a well-conditioned updating problem to improve the model of a structure (Brownjohn & Xia, 2000). Model parameterisation that incorporates sensitivity analysis of the model suppresses the problem of inadequacy whilst providing sensitive parameters (Ereiz et al., 2022). Different methods of model parameterisation that include sensitivity analysis have been developed such as the parameter clustering method and subset selection method ((Jang & Smyth, 2017) (Yuan et al., 2019)). Despite these different methods, the choosing of updating parameters is best achieved through good engineering judgement, having a good appreciation of structural concepts and objectives of the test. To meet both (i) the accuracy and reliability requirement of FE models and (ii) the optimal implementation of the model updating procedure, the parameterisation process should satisfy the following criteria:

- i. The number of selected updating parameters should be limited to overcome ill-conditioned problems,
- ii. Model parameterisation should be used to correct the uncertainty in models, and
- iii. The FE model outputs must be sensitive to the selected updating parameters.

(Ereiz et al., 2022).

3.2.4 Sensitivity based model updating

Sensitivity analysis provides a way to assess that the chosen updating parameters are sensitive to the measured/updating data. The core notion of the sensitivity-based method is predicated on finding the linear approximation of a generally non-linear relationship between the measurable outputs and the model parameters to be updated, developed in terms of a first-order Taylor series as follows:

$$\epsilon = \tilde{z} - z(\theta) \approx r(\theta_j) - S_{ij}(\theta - \theta_j) \quad (3.12).$$

The residual $r(\theta_j)$ is defined at the j^{th} iteration as in equation (3.13):

$$r(\theta_j) = \tilde{z} - z(\theta) \quad (3.13)$$

so that the linearisation is carried out at $\theta = \theta_j$. ϵ , $\tilde{z}$, $z(\theta)$ and θ are as previously defined. For all selected responses and parameters, the sensitivity of response i to change in updating parameter j is given by equation (3.14):

$$S_{ij} = \left[\frac{\partial z_i}{\partial \theta_j} \right] \quad (3.14)$$

where $i=1, 2, \dots, m$ for m responses and $j=1, 2, \dots, p$ for p parameters.

The S_{ij} entry of the sensitivity matrix computed at the present value of the entire vector parameters $\theta = \theta_j$ in which the error, ϵ , is presumed to be small for parameters near θ_j . Thus, at each iteration, equation (3.12) is solved for:

$$\Delta \theta_j = \theta - \theta_j \quad (3.15)$$

and the model is updated to give:

$$\theta_{j+1} = \theta_j + \Delta \theta_j \quad (3.16).$$

The procedure continues until consecutive θ_j and θ_{j+1} are successfully converged (Mottershead et al., 2010).

In general, the sensitivity method yields an expression of the form:

$$\{\Delta \text{obs}\} = [S]\{\Delta p\} \quad (3.17)$$

where $\{\Delta p\}$ is a set of changes to a set of updating variables upon which the structural matrices depend, $\{\Delta \text{obs}\}$ is a set of the error in response between the FE model derived and experimental obtained responses (Greenings, 1999).

Equation (3.14) computes the absolute sensitivities. To compare sensitivities of different parameters the relative sensitivity matrix $[S_r]$ is defined as:

$$[S_r]_{ij} = \left[\frac{\partial z_i}{\partial \theta_j} \right] [\theta_j] \quad (3.18)$$

Additionally, the relative sensitivities can also be normalised with reference to the response value resulting in the development of the normalised relative sensitivity matrix, $[S_n]$, defined as:

$$[S_n]_{ij} = [z_i]^{-1} \left[\frac{\partial z_i}{\partial \theta_j} \right] [\theta_j] \quad (3.19).$$

Equation (3.17) may be determined, over-determined or under-determined for the case whereby the number of responses m is equal to, greater than or smaller than the number of parameters respectively. Solving equation (3.17) using the pseudo-inverse technique yields (Brownjohn et al., 2001):

$$\{\Delta p\} = [S]^+ \{\Delta obs\} \quad (3.20)$$

where, for consistency, $\{\Delta p\} = \{\Delta \theta\}$, $\{\Delta obs\} = \{\Delta z\}$ and $[S]^+$ is the pseudo-inverse matrix of the sensitivity matrix $[S]$ given by:

$$[S]^+ = \begin{cases} [S]^{-1} & \text{for } m = p \\ ([S]^T [S]^{-1}) [S]^T & \text{for } m > p \\ ([S]^T [S] [S]^T)^{-1} [S]^T & \text{for } m < p \end{cases} \quad (3.21)$$

where $[S]^+$ can be calculated by means of the Bayesian estimation or singular-value decomposition methods (Brownjohn et al., 2001).

The singular-value decomposition technique will be discussed in the following section, for more information on the Bayesian estimation, the reader is referred to Simoen et al. (2014) and Ereiz et al. (2022). When the differences between the experimental response and predicted model response is great, the first order Taylor series truncation is underdetermined often resulting in the divergence of the iterative process. Consequently, the starting FE model before updating must be close to the experimentally observed behaviour as alluded to earlier in this section (Brownjohn et al., 2001).

3.2.5 The use of Singular Value Decomposition in Model Updating

The Singular Value Decomposition (SVD) is a powerful tool in numerical analysis and model updating used to optimise over and under determined multiple parameter problems ((Greenings, 1999) (Lee, 2006)). The method described by Golub & Van Loan (2013), may be used to solve an equation of the form:

$$\begin{matrix} [A] & \{\mathcal{U}\} & = & \{b\} \\ M \times N & N \times 1 & & M \times 1 \end{matrix} \quad (3.22)$$

where $\{b\}$ is a set of measured variables, $\{\mathcal{U}\}$ set of updating parameters and $[A]$ is a set of sensitivities of the measured variables, $\{b\}$, to change in the set of updating parameters, $\{\mathcal{U}\}$ ((Greenings, 1999) (Lee, 2006) (Golub & Van Loan, 2013)).

The SVD of an $M \times N$ real matrix $[A]$ involves recasting the matrix as the product of two orthogonal matrices and a set of singular values:

$$\begin{matrix} [A] & = & [\{u\}_1 \{u\}_2 \dots \{u\}_M] & [\Sigma] & [\{v\}_1 \{v\}_2 \dots \{v\}_N]^T & = & [U] & [\Sigma] & [V]^T \\ (M \times N) & & (M \times M) & (M \times N) & (N \times N) & & (M \times M) & (M \times N) & (N \times N) \end{matrix} \quad (3.23)$$

where $[U]$ and $[V]$ are orthogonal matrices, so that:

$$\begin{aligned} [U]^T &= [U]^{-1} \\ [V]^T &= [V]^{-1} \end{aligned} \quad (3.24)$$

in which the columns of $[U]$ and $[V]$ are the left and right singular vectors respectively and $[\Sigma]$ is a rectangular matrix of singular values given as:

$$[\Sigma] = \underbrace{\begin{bmatrix} \sigma_1 & & & 0 \\ & \sigma_2 & & \\ & & \ddots & \\ 0 & \ddots & \ddots & \sigma_N \\ \ddots & \ddots & \ddots & \ddots \\ 0 & 0 & 0 & 0 \end{bmatrix}}_N \left. \begin{matrix} \end{matrix} \right\} \begin{matrix} N \\ M - N \end{matrix} \quad (3.25)$$

where σ_i are the singular values assumed to be ordered from the largest to the smallest along the leading diagonal:

$$\sigma_1 \geq \sigma_2 \geq \dots \sigma_N > 0 \quad (3.26)$$

(Lee, 2006).

Thus, the SVD provides an idea of the rank of an updating problem as r where:

$$\sigma_1 \geq \sigma_2 \geq \dots \sigma_r \geq \sigma_{r+1} \geq \dots \geq 0 \quad (3.27)$$

and

$$\sigma_{r+1} < \eta \quad (3.28)$$

where η is a threshold value below which sets of equations are not considered to contribute beneficially to the solution. An indication of the practical rank of a problem can be given by a comparison of the ratio of the k^{th} singular value to the first singular value (Golub & Van Loan, 2013).

Equation (3.23) can be rearranged as follows based on equation (3.24):

$$[U]^T[A][V] = [\Sigma] \quad (3.29).$$

The optimal solution to solve equation (3.22) is to minimise residual, value R , defined as:

$$R = \|\{b\} - [A]\{\mathcal{U}\}\|_{\mu} \quad (3.30)$$

where the parameter μ shows that different residuals can be specified based on the type of norm. The most used norm is the Frobenius norm, commonly known as the two-norm ((Greenings, 1999) (Lee, 2006) (Golub & Van Loan, 2013)).

For any problem of this type, there exists exactly one $\{\mathcal{U}\}$ that minimises the sum of the squares of the residuals,

$$\min(R_{LS}^2) = \|\{b\} - [A]\{\mathcal{U}\}\|_2^2 \quad (3.31).$$

For

$$A = \sum_{i=1}^{\text{rank}(A)} \sigma_i u_i v_i^T, \quad (3.32)$$

where the least squares estimate of $\{\mathcal{U}\}$ is:

$$\{\mathcal{U}\}_{LS} = \sum_{i=1}^{\text{rank}(A)} \left(\frac{u_i^T b}{\sigma_i} \right) v_i, \quad (3.33)$$

resulting in:

$$R_{LS}^2 = \sum_{i=\text{rank}(A)+1}^N (u_i^T b)^2 \quad (3.34).$$

The vector $[A]\{\mathcal{U}\}_{LS}$ is the orthogonal projection of $\{b\}$ onto the range of $[A]$. The optimal solution for equation (3.31) results in equation (3.35) obtained from equation (3.22):

$$\begin{matrix} \{\mathcal{U}\}_{LS} \\ N \times 1 \end{matrix} = \begin{matrix} [A]^+ \{b\} \\ N \times M \quad N \times 1 \end{matrix} \quad (3.35)$$

where $[A]^+_{N \times M} = ([A]^T_{M \times N} [A]_{N \times M})^{-1} [A]^T_{M \times N}$ is the pseudo-inverse of $[A]$, if the rank of $[A] = M$. For the case where $M = N$,

$$[A]^+ = [A]^{-1} \quad (3.36)$$

Using SVD to solve the problem, equation (3.29) can be rearranged as:

$$[A]^+_{N \times M} = ([V]^T_{N \times N})^+ [\Sigma]^+_{N \times M} [U]^+_{M \times M} \quad (3.37)$$

and

$$[A]^+_{N \times M} = [V]_{N \times N} [\Sigma]^+_{N \times M} [U]^T_{M \times M} \quad (3.38),$$

since $[U]$ and $[V]$ are orthogonal and full rank matrices. Also,

$$[\Sigma]^+ = \left[\underbrace{\begin{bmatrix} \frac{1}{\sigma_1} & & & 0 \\ & \frac{1}{\sigma_2} & & \\ & & \ddots & \\ 0 & & & \frac{1}{\sigma_N} \end{bmatrix}}_N \underbrace{\begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}}_{M-N} \right] \quad (3.39).$$

Thus equation (3.35) can be rearranged using equation (3.38) to give:

$$\{\mathbf{U}\}_{LS}_{N \times 1} = [V]_{N \times N} [\Sigma]^+_{N \times M} [U]^T_{M \times M} \{\mathbf{b}\}_{N \times 1} \quad (3.40)$$

((Greenings, 1999) (Lee, 2006) (Golub & Van Loan, 2013)).

3.3 Numerical Modelling Considerations of the Dynamic Behaviour of Axially Loaded Structures

The dynamic behaviour of a structure is primarily a function of its mass and stiffness. While the stiffness mainly depends on the elastic properties of the constituent materials and the geometry of the structure, it can be changed by internal forces. An excellent example in which internal forces alter the stiffness and the dynamic behaviour of a structure is stringed instruments where increasing or decreasing the tension in the string changes its pitch (Greening,

1999). There are two principal factors that alter the dynamic response of loaded structures, namely: (i) stress (geometric) stiffening and (ii) the structure's deflected shape while under loading. Both these principal factors are generally considered under geometric nonlinearities. Additionally, these two factors are not the only way in which static loading may affect the dynamic response of a structure, other mechanisms exist such as the damping of joints which are not considered in this discussion. The structure's deflected shape while under loading is also not investigated in this study as it is assumed that it does not play a critical role in arch dam structures and is thus not discussed further in this section.

3.3.1 Stress Stiffening

Stress stiffening arises due to the influence of axial/membrane forces on lateral deflection. In other words, it represents the tendency towards buckling. Stress stiffening (also known as initial stress stiffness or geometric stiffness), depends on the condition of loading and configuration of the structure. ((Cook et al., 2002) (Clough & Penzien, 2003)). In this discussion, lateral deflections are presumed acceptably small such that the axial/membrane forces are considered constant during dynamic loading (Clough & Penzien, 2003). The work on stress stiffness predates the extensive usage of matrix methods. Additionally, using vibration data to assess a structure's loading and boundary conditions continues to be of interest to engineers ((Stephens, 1936) (Lurie, 1951)). The concept of stress stiffening is demonstrated in this section using a simple beam-type element based on a technique to evaluate the stress stiffening effects on FE beam elements as detailed by Przemieniecki (1968).

The strain of a beam in bending in which only normal stresses contribute to the energy stored is given by:

$$\epsilon_{xx} = \frac{\partial u_0}{\partial x} - \frac{\partial^2 v}{\partial x^2} y + \frac{1}{2} \left(\frac{\partial v}{\partial x} \right)^2 \quad (3.41)$$

where u_0 is the change in axial length, $v(x)$ is the transverse displacement and y is the dimension measured from the neutral axis.

The first term in equation (3.41) relates to the axial strain term, whilst the second term corresponds to the stored energy due to curvature in the beam, and the third term describes the increase in the length of the beam element owing to its rigid body rotation.

The strain energy is determined using:

$$U_i = \frac{E}{2} \int_V \epsilon_{xx}^2 dV \quad (3.42)$$

Substituting equation (3.41) into equation (3.42) leads to:

$$U_i = \frac{E}{2} \int_V \left[\frac{\partial u_0}{\partial x} - \frac{\partial^2 v}{\partial x^2} y + \frac{1}{2} \left(\frac{\partial v}{\partial x} \right)^2 \right]^2 dV \quad (3.43).$$

For a constant cross-section area, A , expanding equation (3.43) and excluding higher order terms gives:

$$U_i = \frac{EA}{2} \int_0^L \left(\frac{\partial u_0}{\partial x} \right)^2 dx + \frac{EI}{2} \int_0^L \left(\frac{\partial^2 v}{\partial x^2} \right)^2 dx - \frac{EA}{2} \int_0^L \frac{\partial u_0}{\partial x} \left(\frac{\partial v}{\partial x} \right)^2 dx \quad (3.44).$$

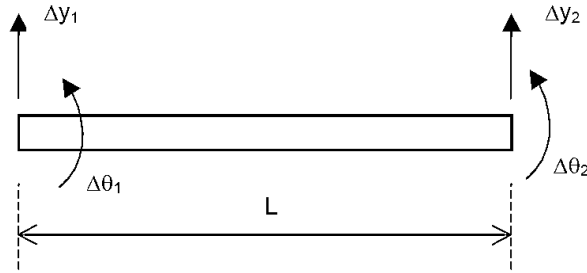


Figure 3.1: Beam under shear and rotational loading (adopted from Greening, 1999)

Assuming the general formulation for lateral displacement in terms of end displacements for a beam undergoing shearing and rotations (Figure 3.1) is given as:

$$\{u\} = [N_1 \quad N_2 \quad N_3 \quad N_4] \begin{Bmatrix} \Delta y_1 \\ \Delta \theta_1 \\ \Delta y_2 \\ \Delta \theta_2 \end{Bmatrix} \quad (3.45).$$

And using a set of cubic functions to denote the assumed shape functions, N_i (equations (3.46) to (3.49)) as:

$$N_1 = 1 - \frac{3x^2}{L^2} + \frac{2x^3}{L^3} \quad (3.46)$$

$$N_2 = x - \frac{2x^2}{L} + \frac{x^3}{L^2} \quad (3.47)$$

$$N_3 = \frac{3x^2}{L^2} - \frac{2x^3}{L^3} \quad (3.48)$$

$$N_4 = \frac{x^2}{L} + \frac{x^3}{L^2} \quad (3.49),$$

$\{v\}$ can be expressed as:

$$\{v\} = \begin{bmatrix} 1 - \frac{3x^2}{L^2} + \frac{2x^3}{L^3} & x - \frac{2x^2}{L} + \frac{x^3}{L^2} & \frac{3x^2}{L^2} - \frac{2x^3}{L^3} & \frac{x^2}{L} + \frac{x^3}{L^2} \end{bmatrix} \begin{Bmatrix} v_1 \\ \theta_1 \\ v_2 \\ \theta_2 \end{Bmatrix} \quad (3.50),$$

which, when differentiated with respect to x gives:

$$\frac{\partial v}{\partial x} = \begin{bmatrix} \frac{6x}{L^2} + \frac{6x^2}{L^3} & 1 - \frac{4x}{L} + \frac{3x^2}{L^2} & \frac{6x}{L^2} - \frac{6x^2}{L^3} & \frac{2x}{L} + \frac{3x^2}{L^2} \end{bmatrix} \begin{Bmatrix} v_1 \\ \theta_1 \\ v_2 \\ \theta_2 \end{Bmatrix} \quad (3.51).$$

The second derivative of equation (3.50) with respect to x gives:

$$\frac{\partial^2 v}{\partial x^2} = \begin{bmatrix} \frac{6}{L^2} + \frac{12x}{L^3} & \frac{4}{L} + \frac{6x}{L^2} & \frac{6}{L^2} - \frac{12x}{L^3} & \frac{2}{L} + \frac{6x}{L^2} \end{bmatrix} \begin{Bmatrix} v_1 \\ \theta_1 \\ v_2 \\ \theta_2 \end{Bmatrix} \quad (3.52).$$

Substituting equations (3.51) & (3.52) into equation (3.44) and minimising the total potential energy with respect to nodal displacements leads to the well-known and widely used two-dimensional cubic beam element Euler-Bernoulli formulation (equation (3.53):

$$[K]_e^{\text{zero}} = \begin{bmatrix} \frac{AE}{L} & 0 & 0 & -\frac{AE}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{2EI}{L} \\ -\frac{AE}{L} & 0 & 0 & \frac{AE}{L} & 0 & 0 \\ 0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \quad (3.53)$$

plus, the elemental stress stiffness matrix, $[K]_e^g$, (equation (3.54)):

$$[K]_e^\sigma = \frac{F}{L} \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ & \frac{6}{5} & \frac{L}{10} & 0 & -\frac{6}{5} & \frac{L}{10} \\ & & \frac{2L^2}{15} & 0 & -\frac{L}{10} & -\frac{L^2}{30} \\ & & & 0 & 0 & 0 \\ & & & & \frac{6}{5} & -\frac{L}{10} \\ \text{sym} & & & & & \frac{2L^2}{15} \end{bmatrix} \quad (3.54)$$

where F is the axial load.

The formulation in equation (3.54) is not an exclusive expression of the influence of stress stiffening but is normally implemented in commercial FE software (Greening, 1999).

The elemental stiffness under an axial/membrane load is expressed as:

$$[K]_e^{\text{loaded}} = [K]_e^{\text{zero}} + [K]_e^\sigma \quad (3.55)$$

where $[K]_e^{\text{loaded}}$ the loaded beam elemental stiffness (Lee, 2006).

3.3.2 Updating of Stress Stiffening Effects

Assuming the updated stiffness matrix is given in terms of a set of updating parameters $p_1, p_2, \dots, p_{n_p}$ and that it is derived based on the a priori model as follows:

$$[K]^U = [K]^A + [\Delta K] \quad (3.56)$$

where the superscripts U and A stand for the updated stiffness matrix and initial (a priori) stiffness matrices respectively (Greening, 1999).

The updated stiffness framed as a first order Taylor expansion with respect to the original structural matrix in powers of p is expressed as:

$$[K]^U = [K]^A + [\Delta K] = [K]^A + \sum_{j=1}^{n_p} \delta p_j \frac{\partial}{\partial p_j} [K]^A \quad (3.57).$$

Expressing the a priori stiffness matrix as developed from a set of unloaded and loaded constituent elemental matrices gives:

$$[K]^A = \sum_{j=1}^{n_{\text{elements}}} \left([K]_{e_j}^{\text{zero}} + [K]_{e_j}^\sigma \right) \quad (3.58).$$

Equation (3.58) assumes that the axial load in an element (implied in the stress stiffening term) is accessible. Modifying the a priori stiffness matrix as a function of a set of factors $p_1, p_2, \dots, p_{n_{\text{elements}}}$ on the stress stiffening terms leads to:

$$[K]^A = \sum_{j=1}^{n_{\text{elements}}} ([K]_{e_j}^{\text{zero}} + p_j [K]_{e_j}^{\sigma}) \quad (3.59).$$

Differentiating equation (3.59) with respect to p_j results in:

$$\frac{\partial [K]^A}{\partial p_j} = [K]_{e_j}^{\sigma} \quad (3.60)$$

from which higher derivatives are seen to be zero (Greening, 1999).

Comparing equation (3.57) and equation (3.60), the relation between the updated structural matrix in terms of the updating parameter can be expressed as:

$$[\Delta K] = \sum_{j=1}^{n_{\text{elements}}} \delta p_j [K]_{e_j}^{\sigma} \quad (3.61).$$

As can be seen, the stress stiffness matrix has the axial load as a factor. Substituting $[\hat{K}]$, defined as the standardised stress stiffness, into equation (3.59), it results in the parameter p_j standing for the value of the load in equation (3.62):

$$[K] = \sum_{j=1}^{n_{\text{elements}}} ([K]_{e_j}^{\text{zero}} + p_j [\hat{K}]_{e_j}^{\sigma}) \quad (3.62)$$

(Greening, 1999).

A similar formulation can be derived for the mass matrix. In this study, only the variations to the stiffness matrix are investigated.

Considering the dynamic problem defined by equation (3.63):

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = \{F\} \quad (3.63)$$

where $\{F\}$ is a vector of the loads/force applied as a function of time at the degrees of freedom of the structure, $[K]$ is the stiffness matrix, $[M]$ is the mass matrix, $[C]$ is the viscous damping model, and $\{\dot{x}\}$ & $\{\ddot{x}\}$, the first and second derivatives of the displacements $\{x\}$ with respect to time.

Assuming a free and undamped vibration since in this study the concerned is with modal placement and not amplitude dependency, equation (3.63) becomes:

$$[M]\{\ddot{x}\} + [K]\{x\} = 0 \quad (3.64),$$

the solution of which can be written in the form:

$$\{x\} = \{\Psi\}_j e^{i\omega_j t} \quad (3.65)$$

where ω_j is the j^{th} resonant frequency, $\{\Psi\}_j$ the mode shape aligned with the j^{th} system natural frequency, ω_j , and t represents time.

Substituting equation (3.65) into equation (3.64) leads to the well-known eigen problem:

$$[K]\{\Psi\}_j = \lambda_j [M]\{\Psi\}_j \quad (3.66)$$

where

$$\lambda_j = \omega_j^2 \quad (3.67).$$

Scaling the eigenvectors such that:

$$[\Phi]^T [M] [\Phi] = [I] \quad (3.68).$$

results in:

$$[\Phi]^T [K] [\Phi] = \text{diag}(\lambda) \quad (3.69)$$

where $[\Phi]$ is the mass normalised eigenvector matrix.

The sensitivity of the i^{th} eigenvalues of a system to adjustments to the p_j updating parameter can be shown to be:

$$\frac{\partial \lambda_i}{\partial p_j} = \{\Phi\}_i^T \left(\frac{\partial}{\partial p_j} [K] - \lambda_i \frac{\partial}{\partial p_j} [M] \right) \{\Phi\}_i \quad (3.70)$$

where λ_i and $\{\Phi\}_i$ the i^{th} eigenvalue and mass normalised eigenvector respectively.

Substituting equation (3.60) into equation (3.70) leads to:

$$\frac{\partial \lambda_i}{\partial p_j} = \{\Phi\}_i^T [K]_{e_j}^{\sigma} \{\Phi\}_i \quad (3.71).$$

The sensitivity of n_{modes} eigenvalues to n_p parameter quantities is given as:

$$\begin{bmatrix} \frac{\partial \lambda_1}{\partial p_1} & \frac{\partial \lambda_1}{\partial p_2} & \dots & \frac{\partial \lambda_1}{\partial p_{n_p}} \\ \frac{\partial \lambda_2}{\partial p_1} & \frac{\partial \lambda_2}{\partial p_2} & & \vdots \\ \vdots & & \ddots & \frac{\partial \lambda_{n_{\text{modes}}}}{\partial p_{n_p}} \\ \frac{\partial \lambda_{n_{\text{modes}}}}{\partial p_1} & & & \end{bmatrix} \begin{Bmatrix} \delta p_1 \\ \delta p_2 \\ \vdots \\ \delta p_{n_p} \end{Bmatrix} = \begin{Bmatrix} \delta \lambda_1 \\ \delta \lambda_2 \\ \vdots \\ \delta \lambda_{n_{\text{modes}}} \end{Bmatrix} \quad (3.72)$$

or in matrix, vector form:

$$[S]\{\delta p\} = \{\delta \lambda\} \quad (3.73)$$

in which the rows of $[S]$ and $\{\delta \lambda\}$ provide the necessary data to estimate of the unknown changes in the updating parameters $\{\delta p\}$. These changes can be ascertained by pre-multiplying the right-hand side of equation (3.73) by the pseudo-inverse of the sensitivity matrix expressed in a least-square context using the singular value decomposition previously discussed in section 3.3.5:

$$\{\delta p\} = [S]^+ \{\delta \lambda\} \quad (3.74).$$

Using the previously illustrated minimisation of the Frobenius norm of the residual based on the SVD given as:

$$R = \|\{\delta \lambda\} - [S]\{\delta p\}\| \quad (3.75),$$

an iterative updating process is then used to obtain the best estimates of $\{\delta p\}$ for the given $\{\delta \lambda\}$ (equation (3.16)) in which the $(k+1)^{\text{th}}$ set of updating parameters are derived as:

$$\{p\}_{k+1} = \{p\}_k + \{\delta p\}_k \quad (3.76)$$

where the magnitude of the variations in the parameter is used to measure convergence at a specific step in the updating process:

$$\text{mag} = |\{\delta p\}| \quad (3.77).$$

Convergence occurs when the magnitude value becomes less than a defined threshold. For structural loading, the definition of the threshold sets the accuracy with which the loads can be determined (Greening, 1999).

3.3.3 Effect upon Validation/Updating

In the foregoing discussion, it has been illustrated that in the case of loaded structures modifying their dynamic behaviour through model updating limited to elemental stiffness, density, or cross-sectional area modifications only is inadequate and invalid. Acceptable changes to the original FE model to account for the load effect can be achieved through the stress stiffening matrix, and accounting for large-deflection transformation for flexible structures. Stress stiffening generally alters the transverse stiffness of a structure subjected to axial loading, resulting in a shift in the same direction of the flexural resonant frequencies whilst extensional frequencies are not altered.

3.4 Structural Behaviour Assessment of Axially Loaded Concrete Structures

This section provides a brief review of literature illustrating the use of vibration-based approaches and the application of stress stiffness in the evaluation of loaded structures:

3.4.1 Vibration-based Assessment of Loaded Concrete Structures and/or Elements

Siegert et al (2005) conducted research aimed at compiling an experimental database for assessing the degradation of mechanical properties of beam structural elements subject to ASR. The tests carried out included periodical measurements of the natural frequencies of beams, to investigate the viability of vibration-based health monitoring in detecting global variations of physical parameters such as the Young's modulus or the loss of the bending rigidity of ASR damaged beams. The amount of stiffness reduction determined using natural frequency variations of the ASR damaged concrete beams was found to be in the range of between 12 % to 20 %. Further, the researchers observed that the sensitivity of the natural frequencies increased with an increase of the spatial extent of damage and that the bending rigidity was substantially fully recovered at the end of the tests. The recovery of the bending rigidity was explained by a "healing" process as the reaction progresses and/or due to structural effects. An important conclusion of their study was that vibration-based monitoring of ASR affected structures needs a careful analysis due to various factor at play including moisture artefacts (Siegert et al, 2005).

Investigations have also been undertaken to evaluate the effects of cracks on the dynamic behaviour of concrete beams including for pre-stressed RC concrete beams and frames (Ibrahim et al, 2013). Some of these studies focussed on the nonlinear behaviour of the beams with the closing and opening of the cracks and their effect on the bending rigidity of the beam

(Breccolotti et al, 2009). Whilst other researchers such as Huszar (2008) studied the influence of the sign of the moment and its overall effect on the dynamic behaviour of the cracked reinforced and prestressed concrete beams. Moreover, some of these studies assessed the feasibility of assessing and monitoring the prestressing forces in prestressed concrete structures using vibration-based methods amongst other methods ((Abdel-Jabber & Glisic, 2019) (Materazzi et al, 2009)).

Of particular interest is the application of vibration-based assessment of existing structures. One such application is the case of Cahora Bassa Dam, a thin double curvature arch dam that has been in operation for about 50 years and whose swelling due to ASR was detected in the 1980s (Alegre et al., 2023). A dynamic monitoring system was installed in 2010 to monitor the dynamic behaviour of the dam (Carvalho et al., 2023). Observations from the ambient vibration measurements reveal no influence of ASR on the dynamic behaviour of the dam over a twelve-year period (Alegre et al., 2023). To the best of the author's knowledge, no other similar applications on real structures have been reported to date.

3.4.2 Geometric stiffness application illustrations in the assessment of Axially Loaded Concrete Structures

Papadopoulos et al. (2009) examined the confinement of a reinforced concrete (RC) column from the context of stability of concrete. They reported sudden drops in axial compressive stress for beyond a specified axial compression strain of a RC column. They simulated the RC column using truss bars that obey a nonlinear stress-strain law of concrete or steel to investigate the column instability. The equilibrium and stiffness conditions were written in terms of the deformed truss within each loading step incremental procedure to account for geometric nonlinearities. A failure mode of the column that constitutes in the buckling of internal vertical concrete struts was observed, from which a technique to calculate the required the required mechanical ratio of transverse reinforcement to prevent the global instability of the column by hand was developed. Lateral geometric stiffness of the vertical concrete struts was one of the three parts that constituted the total stiffness. The other two parts were lateral elastic stiffness of the vertical concrete strut and the stiffness of the transverse steel bar of the model (Papadopoulos et al., 2009). In a later publication, Papadopoulos & Plasatis (2015) further investigated the conditions under which the buckling of internal struts occurs for the global instability model. The required mechanical ratio of transverse reinforcement to prevent the buckling of the concrete was achieved through a two-step process using: (i) equilibrium

conditions along the direction of transverse reinforcement, which is in tensile yielding, and (ii) an inequality written such that elastic stiffness minus geometric stiffness must be greater than zero. The proposed procedure was applied on a typical RC column and was found to be in close approximation with relevant uniform building code (UBC) recommendations that are based on test data (Papadopoulos & Plasatis, 2015).

Shariff & Menon (2015) presented details of a displacement controlled nonlinear analysis algorithm whose aim was to trace the load-deflection relation of reinforced concrete beams, frames, and grids. A geometric stiffness matrix was used together with a primary stiffness matrix adjusted iteratively to account for geometric nonlinearity. The geometric stiffness was developed based on stability functions and added to the elemental stiffness matrix to account for P- Δ effects. Moreover, the algorithm accounted for material nonlinearity using prescribed moment-curve formulations, that incorporates stiffness degradation due to the concrete cracking, steel yielding tension stiffening and strain softening. The authors illustrated the validation of the methodology they proposed against experimental data.

Wahrhaftig et al (2013) assessed the computational results of a proposed mathematical formulation that accounts for the influence of axial stress on the natural frequency of cantilevered structures. The mathematical formulation results were compared with dynamic laboratory tests observations. The objective of the study was to assess the effect of axial force on the natural frequency of isolated bars to aid in the evaluation of the fundamental frequency of real structures. The proposed analytical expression was based on the principle of virtual work and on Rayleigh's method (Wahrhaftig et al., 2013). Wahrhaftig & Brasil (2016) further illustrated the implementation of the proposed analytical approach to determine the first natural frequency of a 46-m-high concrete mobile phone mast system, accounting for the geometric stiffness, concentrated forces, and self-weight of the structure. The vibration frequency of the fundamental mode was calculated analytically and compared with an FEM based simulation. The difference in the solutions for the first natural mode was reported as 3% with the inclusion of the geometric stiffness component and about 6.5% without the inclusion of the geometric stiffness component (Wahrhaftig & Brasil, 2016). Wahrhaftig et al (2017) expanded on the developed expression to calculate the fundamental frequency to include effects of creep and applied this method in evaluating the resonant frequencies in the vibration of a prestressed reinforced concrete beam (Wahrhaftig et al., 2017).

Michiels & Adriaenssens (2017) presented results from a parametric study to explore the influence of the shell geometry on the dynamic properties and earthquake responses of reinforced concrete (RC) roof shells with a square plan. The aim of the study was to identify the most important design variables that result in a shell structure for a specified plan area whose deformation, stress and buckling performance is optimal. It was observed that small to mid-sized thin concrete roof shells (the characteristics of which they defined, refer to (Michiels & Adriaenssens, 2017)) can be intrinsically earthquake-resistant due to their greater geometric stiffness and low mass that results in higher natural frequencies that exceeds the normal seismic driving frequencies. They also observed that for shallow shells, the earthquake's vertical vibration components induced greater stresses in the shell compared to the horizontal component, which could be detrimental for these structures. An increase in the rise and curvature of the roof shell could result in an increase in frequency reducing the detrimental impact of the vertical vibration component of an earthquake. Among other conclusions, they concluded that in the case of seismic events that do not have a strong vertical component, the shape and span of the reinforced concrete roof shell which they investigated, can play a critical role in ensuring earthquake-resistant roof shell structures (Michiels & Adriaenssens, 2017).

3.5 Chapter Summary

In this chapter, a review of FEMU approaches and the theoretical basis for updating procedures that account for geometric stiffness was described and its implementation in FEM illustrated using a 2D beam element. Additionally, some experiences on the incorporation of geometric stiffness in the assessment of loaded structures and the use of dynamic approaches in the assessment of these structures were highlighted.

One key observation to highlight from the preceding discussion is that in most cases, the dynamic properties are used as a first measure of the current behaviour of a structure in the model updating process, since modal parameters are sensitive to any changes in the structural system's stiffness and mass. The vibration-based assessment experiences of ASR affected structures reported in this section are based on this premise as well. This is an important observation for this study, as it might not be adequate to merely calibrate models based on the static behaviour of concrete arch dams, as the static measurements do not, in most cases capture the systems characteristics and/or changes in these characteristics. This is an important observation which is key in the consideration of the proposed model updating approach.

Another key observation that has been illustrated in the preceding discussion is that model updating of flexible loaded structures limited to modifying the elemental stiffness or cross-section area is insufficient and invalid. This point is further corroborated by the reported influence of geometric stiffness on the earthquake resistance of RC roof shell structures. However, there is limited research reported in technical literature, on the finite element model updating of stiffness changes induced by internal stresses in a dam wall, specifically, aggregate silica reaction (ASR) induced stresses that may impact a dam's operation. Most studies have focussed on the elastic degradation aspect of ASR on concrete structures. To accurately assess the current state of behaviour and safety of concrete arch dams affected by ASR, the development of finite element model updating procedures that account for stress-induced stiffness changes in these structures maybe essential. Consequently, to meet the objectives of this study, the potential influence of stress-stiffness on the dynamic behaviour of concrete arch dams was investigated and is reported on in Chapter 6.

The last key observation to highlight is that model updating approaches are generally constrained by the amount of data available to be used in the updating process or the number of unknown parameters to update. One way to solve the resulting ill-conditioned problem for the case where too many parameters are unknown, is to measure, where possible, all parameters that can possibly be measured essentially reducing the number of unknown parameters. Additionally, selecting the model updating parameters based on an understanding of the model's sensitivity to the updating parameters in dealing with the problem of inadequacy, is another useful approach to defining a well-conditioned updating problem to improve the model of a structure. Consequently, a sensitivity study towards the selection of model updating parameters for the proposed model updating approach in this study (presented in Chapter 7) was carried out and the observations are presented in Chapter 6.

3.6 References

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Chapter 4

FINITE ELEMENT MODELLING OF CONCRETE ARCH DAMS IN OPERATION

4.1 Introduction

This research study focuses on the development of a finite element model updating procedure for concrete arch dams affected by ASR using dam surveillance data including dynamic monitoring, specifically, ambient vibration monitoring (AVM), data in the model updating procedure. Thus, the proper development of concrete arch dam FE models and their analysis is critical to this study and is the subject of this Chapter. FE models provide an idealisation of the real physical dam systems including the interaction with their surrounding through physical or chemical processes which account for the external actions on dam structures. The idealisation of the physical system is normally carried out under a specific level of abstraction through mathematical models based on laws of physics ((VanDerHorn & Mehadevan, 2021) (Chiachio et al, 2022)). A mathematical model is here understood as a simplified geometric model whose behaviour has been described (i.e., material properties, loads and boundary conditions have been defined) and an analysis theory applied for its solution (Cook et al., 2002). These models can be used to assess the structural performance and its evolution over time using states and parameters that can be adaptively updated based on the measured performance of dams and of the environmental variables ((VanDerHorn & Mehadevan, 2021) (Chiachio et al, 2022)). The objective of this chapter is to describe the general model development, analysis and model updating process for concrete arch dams in operation whilst highlighting the relevant methodological aspects for this study. The methodology aspects not covered in this Chapter which are relevant for different aspects investigated towards achieving the objectives of this study are given in the relevant chapters.

The different considerations in the development of mathematical models for the analysis of arch dams in this chapter will be covered as follows: firstly, the classification of concrete arch dams which influences the geometric characteristics of concrete arch dams and form the basis upon which the various arch dam configurations considered throughout this study are selected and developed, is considered first. Following which, the method by which the dam system geometries are developed, material properties, boundary conditions and interactions specified in ABAQUS, is detailed. Next, the different loading conditions investigated in this study and their implementation for analysis are considered. Lastly, a discussion on the calibration of the developed models using measured data follows. The chapter ends with an overall overview of the different aspects investigated in this study and how the implementation of the aspects covered in this section contribute towards achieving the objectives of this study.

4.2 Classification of Concrete Arch Dams

In this study, an arch dam is defined as a solid concrete dam that is curved in plan (i.e. in the horizontal plane) (Salamon & Nuss, 2012). Arch dams are designed to transmit loads (static and dynamic loads) as efficiently as possible into the abutments and foundation achieved by optimising the thickness, shape, and curvature of the arch dam design for each dam and site location. Consequently, arch dams may vary according to the curvature, dam thickness and the number of lines of centres necessary to describe the face or portion of dam (Salamon & Nuss, 2012). Additionally, the type of arch dam structure used in a design is highly dependent on the overall shape of the site (USACE, 1994).

4.2.1 Curvature of arch dam

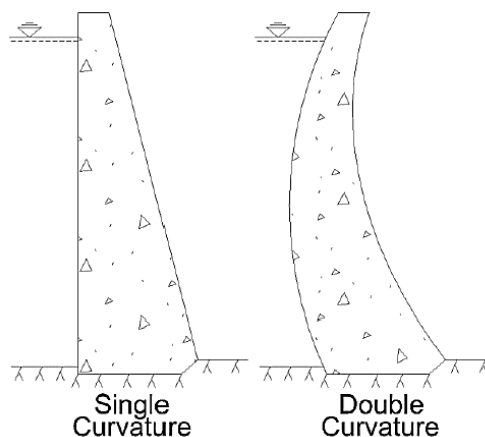


Figure 4.1: Crown cantilever single and double curvature arch dam illustration (adopted from Enzell & Tollsten, 2017)

The crown cantilever shape defines the curvature classification of arch dams as either single curvature (curved in plan only) or double curvature (curved in both plan and in elevation) (Figure 4.1) (Salamon & Nuss, 2012).

4.2.2 Dam thickness

Arch dams are generally classified as thin, medium-thick, and thick based on the dam base thickness (b_t) to structural height (h) ratio. Additionally, their thickness may vary from the crest to the base. In this study, a thin arch is assumed to have a b_t : h ratio of 0.2 or less, a medium-thick arch dam as having a b_t : h ratio between 0.2 and 0.3, and a thick arch dam defined by a b_t : h ratio greater than 0.3 (USBR, 1977).

4.2.3 Lines of centres

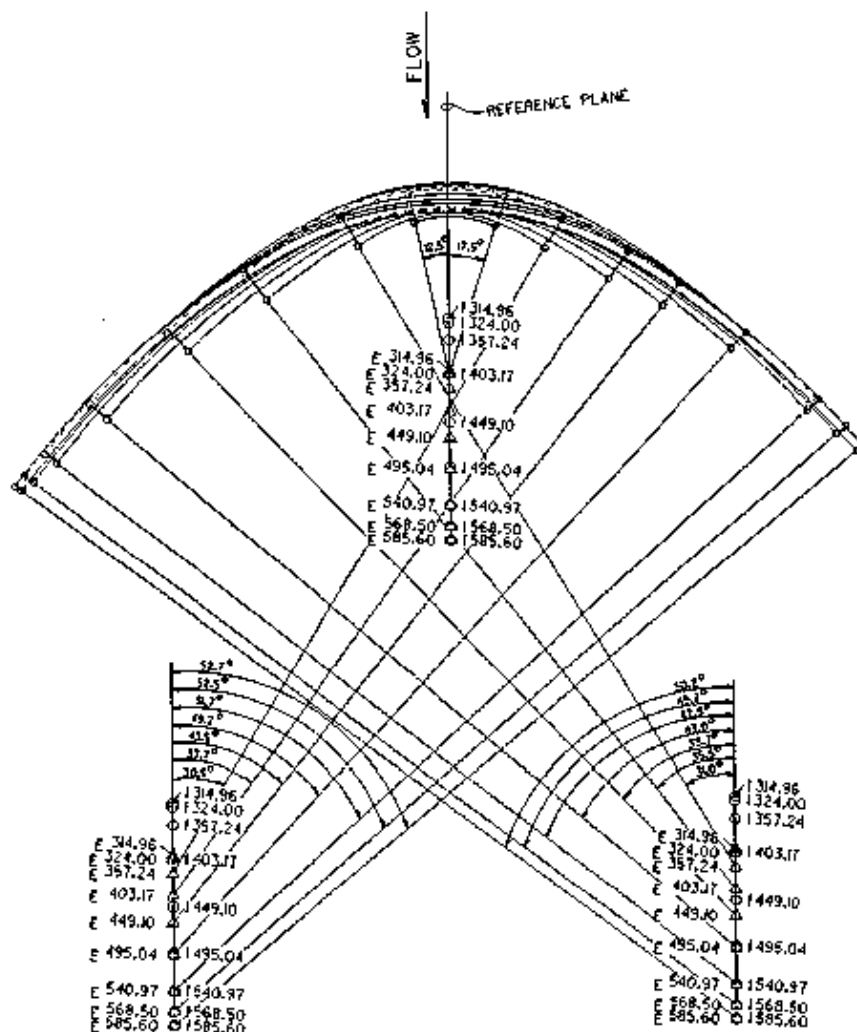


Figure 4.2: Plan view illustration of the lines of centre for a three-centred variable-thickness arch dam (adopted from USACE, 1994)

The lines of centres prescribe the horizontal profile of the arches used to construct the arch geometry at each height of the dam. The number of lines of centres necessary to describe an arch dam varies from one for a circular dam with uniform thickness arches to six for a three-centred dam with variable-thickness arches ((Salamon & Nuss, 2012) (USACE, 1994) (USBR, 1977)). Figure 4.2 shows a plan view of a three-centred variable-thickness arch dam.

4.2.4 Dam site profiles

The type of arch dam structure used in a design is dependent on the overall shape of the site. The profiles of the dam site can generally be classified as narrow-V, narrow-U, wide-V, or wide-U shaped based on the crest length-to-height ratio ($l_c: h$) and streambed width (idealised in this study using the dam structure's base length (l_b)). Dam sites with a $l_c: h$ ratio of 2 or less are classified as narrow sites whereas sites with a $l_c: h$ ratio of 5 or more (but less than 10) are classified as wide sites. The canyon walls of a U-shaped site are assumed near vertical and the streambed width about one-half the canyon width at the crest (USACE, 1994).

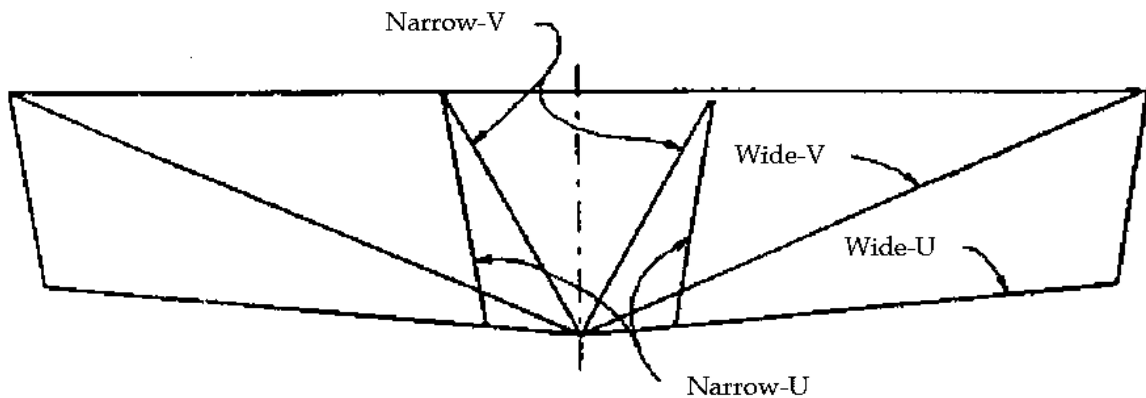


Figure 4.3: Schematic profile illustration for the general dam site classification(s) (adopted from USACE, 1994)

4.2.5 Arch dam configurations considered in this study

In this study single-centre with variable-thickness single curvature (Figure 4.4a) and double curvature (Figure 4.4b) arch dam configurations are investigated. Figure 4.5 shows the different configurations of arch dams considered in this study. The structural height (h) for all dam configurations was kept constant at about 84 m. The crest length-to-height ratio ($l_c: h$) and base length-to-crest length ratio ($l_b: l_c$) for each narrow, wide, U-shaped, or V-shaped dam configuration were kept constant (Figure 4.5). Additionally, a $b_t: h$ ratio of 0.12 was used for all arch dam configurations classified as thin, 0.27 for medium-thick and 0.36 for thick classified arch dam configurations.

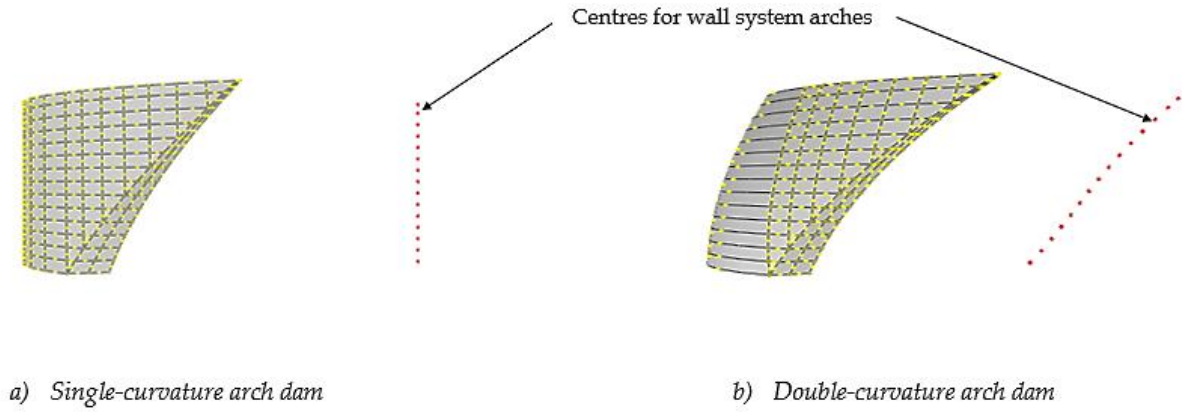


Figure 4.4: Illustration of the position of centres for arches of the wall systems considered in this study

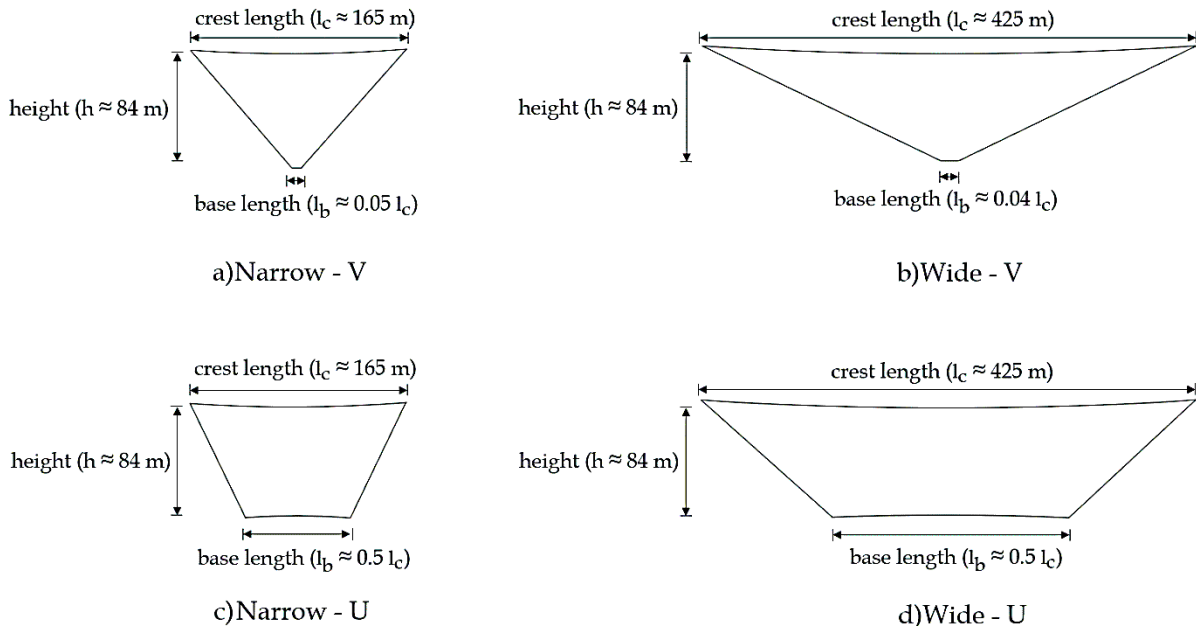


Figure 4.5: Arch dam configurations considered in this study

4.3 Finite Element Model Development of Concrete Arch Dams in Operation

Finite element (FE) models are usually used to assess the current response and to estimate the future structural response of concrete arch dams as they provide a solid foundation for evaluating the safety and performance of concrete arch dam structures (Pedro, 1999). All the analytical modelling work in this study was carried in the general-purpose finite element programme, ABAQUS.

4.3.1 Typical Finite Element Model Development of an Arch Dam Structure

The typical FEM development of an arch dam begins with the formulation of the mathematical model that FEA can be applied to. This is achieved by defining the geometry, material properties, loads, boundary conditions and assumptions to simplify the analysis without losing accuracy of the required results (Cook et al., 2002). For an arch dam structure, the geometry of the dam system includes the dam wall, foundation, and fluid components.

4.3.2 Main Characteristics used to describe the Geometry of Concrete Arch Dams

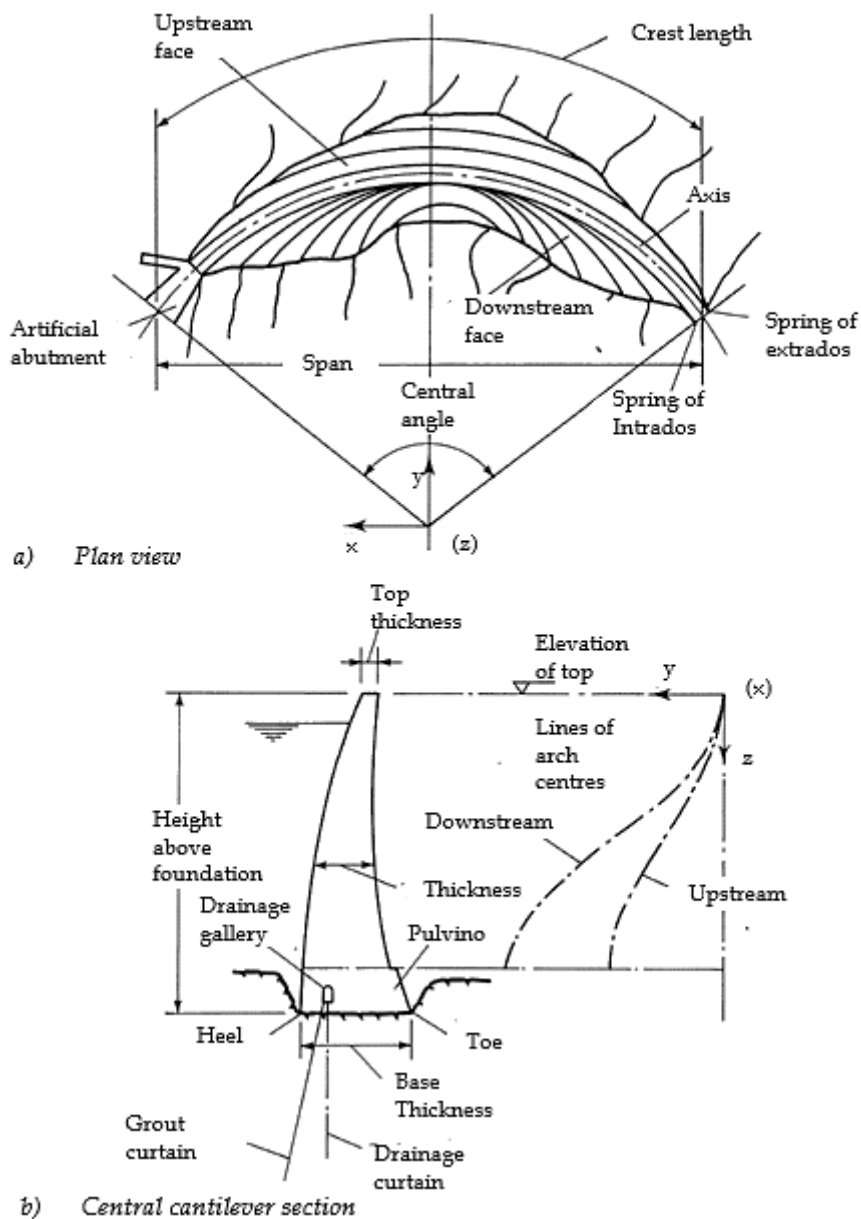


Figure 4.6: Arch dam terminology (adopted from Pedro, 1999)

Figure 4.6 illustrates the terminology used in describing arch dams, which is critical in the development of the geometry of these structures. This information is usually contained in the construction and/or as-built drawings of a dam structure. In this study, the information pertinent to the development of the geometry was obtained from the engineering drawings.

4.3.3 Development of Dam Wall Geometry

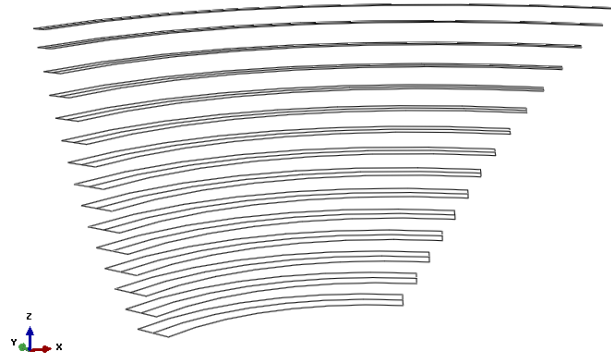


Figure 4.7: Cross-sectional arch wires used to develop the dam wall system developed for a thin walled narrow-U arch dam configuration

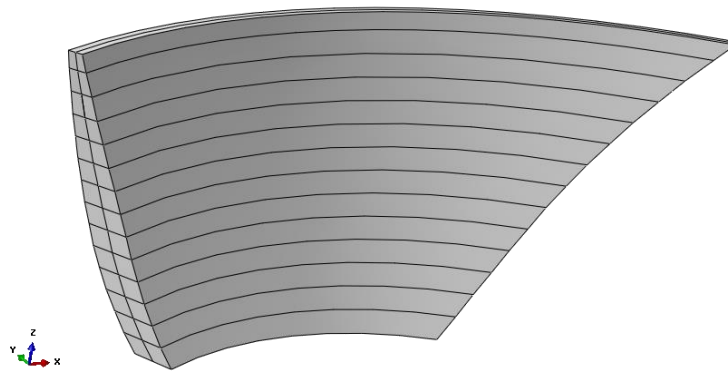


Figure 4.8: 3D deformable monolithic solid dam wall part developed for a thin walled narrow-U arch dam configuration

The dam wall model geometries developed in this study were adapted from drawings provided by the Department of Water and Sanitation. In developing the dam wall system, arch wires representing the horizontal cross section of the dam at different relative level(s) (RL(s)) were drawn on different planes that varied along the z- direction based on the model reference coordinate system (Figure 4.7). After which, a lofting method/function in ABAQUS was

employed to construct the three-dimensional deformable solid region/part of the dam wall (Figure 4.8). The dam wall system was developed assuming a monolithic structure.

4.3.4 Development of Fluid Geometry

The fluid medium was developed similarly to the dam wall system (Figure 4.9), the 3D deformable solid part was generated from cross-sectional wire features developed using the extrados radii and abutment edges via the lofting procedure. The fluid medium was developed to extend about three times the height of the dam as recommended by USACE (1994).

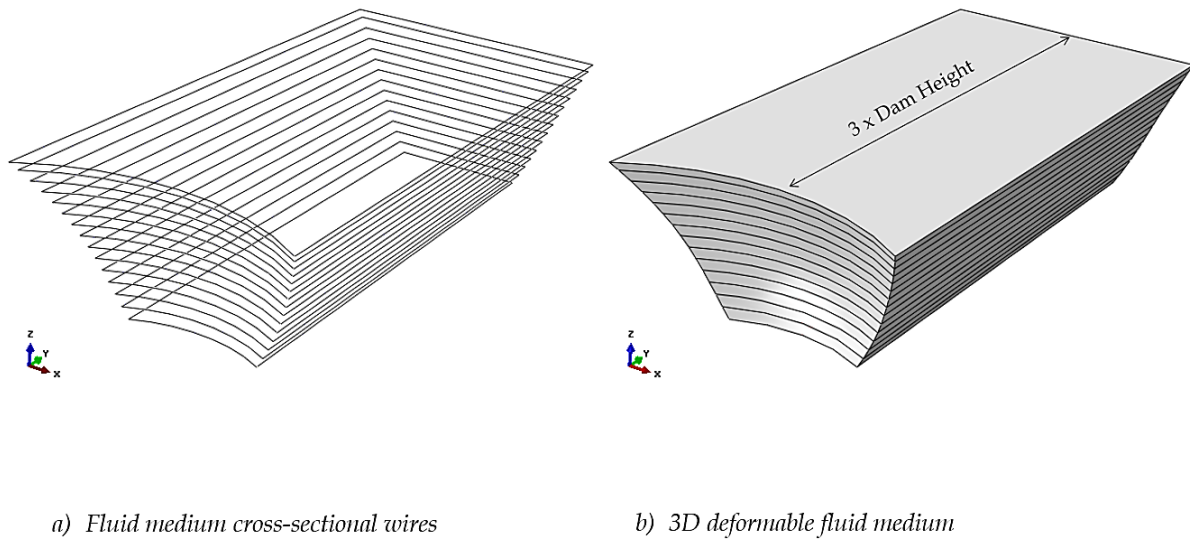


Figure 4.9: 3D deformable fluid medium part development

4.3.5 Development of Dam Foundation Geometry

The foundation system was modelled as a homogenous solid structure assuming a uniform canyon and semi-unbounded regular region, to idealise a foundation system that extends to infinity such that boundary effects on the dam stresses are negligible ((Chopra, 2020)) (USACE, 1994)). The foundation system was developed similarly to the dam wall and fluid medium through the lofting of cross-sectional wires features technique (illustrated in Figure 4.10).

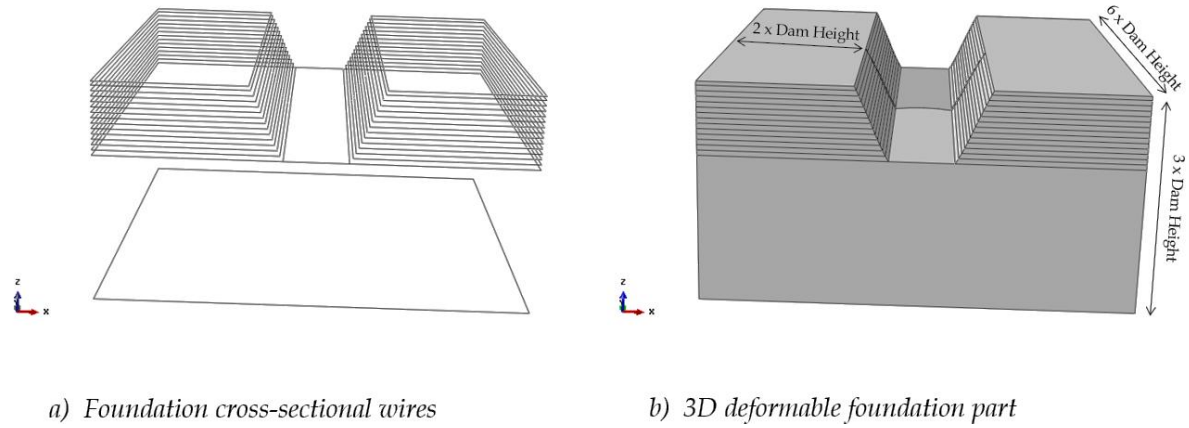


Figure 4.10: 3D deformable foundation part development

4.3.6 Definition of the Dam-Foundation-Fluid Interaction

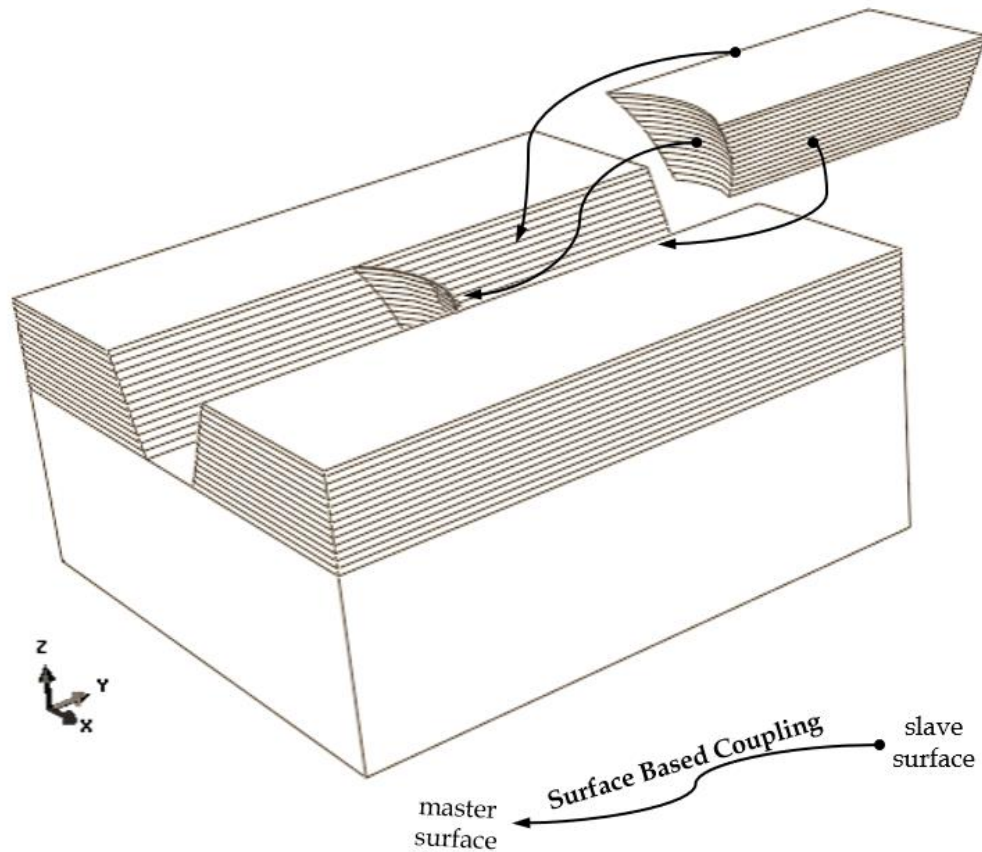


Figure 4.11: Illustration of the surface-based tie constraints applied for the dam-foundation-fluid interaction

The behaviour of a dam in service is subject to the dam-foundation-reservoir/dam-foundation-fluid/dam-foundation-water system and their interaction. The dam wall and the rock foundation parts were merged to generate a single part or developed as one part. Thus, no boundary condition or interaction between the foundation and dam wall had to be defined.

The fluid part was connected to the dam wall and rock foundation parts by means of a surface-based tie constraint in which the wall and foundation interfaces were defined as the master surfaces and the fluid interface, the slave surface (Figure 4.11). The surface-based tie interaction allows the fusion of two surfaces using a master-slave technique for compatibility between the two surfaces.

4.3.7 Meshing of complete FE model

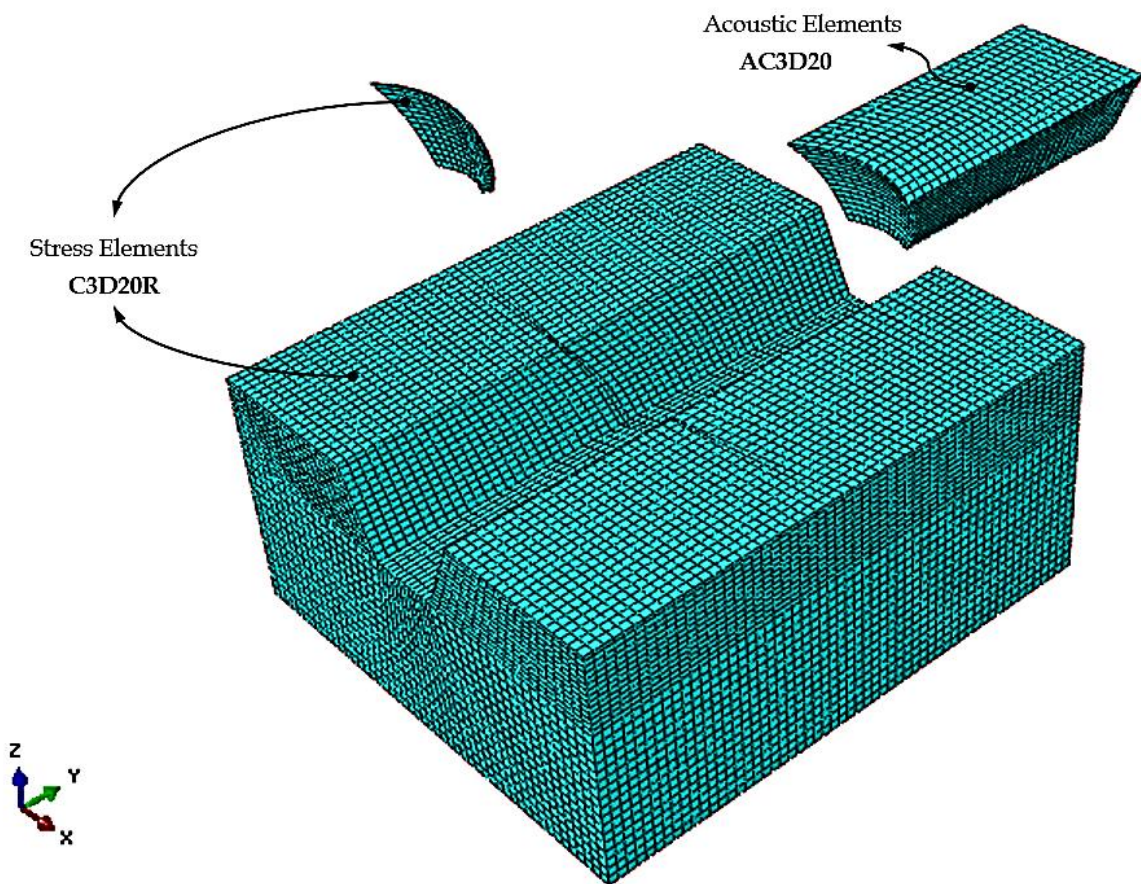


Figure 4.12: Illustration of the type of elements used for each part of the dam-foundation-fluid system

A mesh sensitivity of different element types, shapes, and seeds in ABAQUS, was carried out as the first step, to find the most computationally efficient and reasonably accurate mesh for use in this study. The mesh quality was confirmed through the verification feature in ABAQUS that applies a failure criterion to check these specifications:

- i. Aspect ratio greater than 10,
- ii. Face corner angles greater than 160° ,

- iii. Face corner angles less than 10° .

Any elements failing under these categories may exhibit poor behaviour when analysed, resulting in erroneous analysis results.

For the dam wall, foundation, and reservoir system 20-node isoparametric brick elements that allows for quadratic interpolation were used (Figure 4.12). A reduced integration element (C3D20R) part solution was implemented for the dam wall and rock foundation due to its capability to overcome shear locking and to constrain the transmission of the hourglass modes for these parts. Acoustic elements (AC3D20) for the fluid medium were employed allowing for a realistic representation of the impounded water body. For different configurations of the dam, different seed types were used to optimise the computational time whilst still obtaining sufficiently accurate output results.

4.3.8 Model Material properties

Tables 4.1 to 4.3 provide a summary of the key material parameters used in modelling the dam structure systems in this study. Unless otherwise stated, linear behaviour, homogeneity and isotropy were assumed for the dam wall, foundation, and fluid system. Arbitrary elastic material properties were applied to the dam wall and rock foundation based on knowledge of common engineering practice for concrete dams, whilst typical fluid properties were applied for the fluid medium.

Table 4.1: Concrete dam wall model material properties

Property and Symbol	SI Units	Concrete Wall
Static Elastic Modulus ($E_{c, \text{static}}$)	GPa	30.0
Dynamic Elastic Modulus ($E_{c, \text{dynamic}}$)	GPa	39.0
Poisson's ratio (ν_c)	-	0.22
Density (ρ_c)	kg/m ³	2400
Coefficient of expansion (α_c)	-	1.0×10^{-5}

The static elastic modulus ($E_{c, \text{static}}$) and Poisson's ratio (ν_c) material properties are used in building the stiffness matrix in the case of a static analysis. In the case of a dynamic analysis, the dynamic elastic modulus ($E_{c, \text{dynamic}}$), estimated using a relationship between the static and dynamic moduli defined in Owens (2009), is used to build the stiffness matrix instead. The density is the material parameter used in building the mass matrix whilst the coefficient of

thermal expansion (α_c) of the concrete is important in the static analysis of structures experiencing thermal loads.

Table 4.2: Rock foundation model material properties

Property and Symbol	SI Units	Rock Foundation
Static Elastic Modulus ($E_{f, \text{static}}$)	GPa	30.0
Dynamic Elastic Modulus ($E_{f, \text{dynamic}}$)	GPa	39.0
Poisson's ratio (ν_f)	-	0.22

A massless foundation was assumed in this study; thus, the foundation only plays a part in the stiffness and not the mass of the model. The assumption of the massless foundation means that energy dissipation in the foundation, which is important in a seismic analysis but not so important for the extraction of modal parameters (since there are no loads applied to the foundation boundary in this study), is not considered ((Vezi, 2014) (Chopra, 2020)).

Table 4.3: Fluid medium (water) model material properties

Property and Symbol	SI Units	Fluid Medium
Bulk Modulus (K_w)	GPa	2.2
Density (ρ_w)	kg/m ³	1000

Table 4.3 provides a summary of the material properties used for the fluid medium (water).

4.3.9 Application of Boundary Conditions

Static and Dynamic Analysis was carried out on (i) different dam wall configurations (excluding the foundation and fluid medium) in which case fixed boundary conditions were assumed (for all dam configurations) at the dam-foundation interface (Figure 4.13a), and (ii) for selected dam configurations, the full dam-foundation-fluid system was analysed, in which case a pinned boundary condition was applied to the foundation's bottom surface. Additionally, the top fluid surface pressure was assumed to be atmospheric (i.e., assigned a magnitude of zero) and a non-reflective planar acoustic impedance boundary that damps the acoustic hydrodynamic degrees of freedom moving towards the upstream to avoid imposing interferences on the solution, was applied on the upstream end surface of the fluid medium (Figure 4.13b).

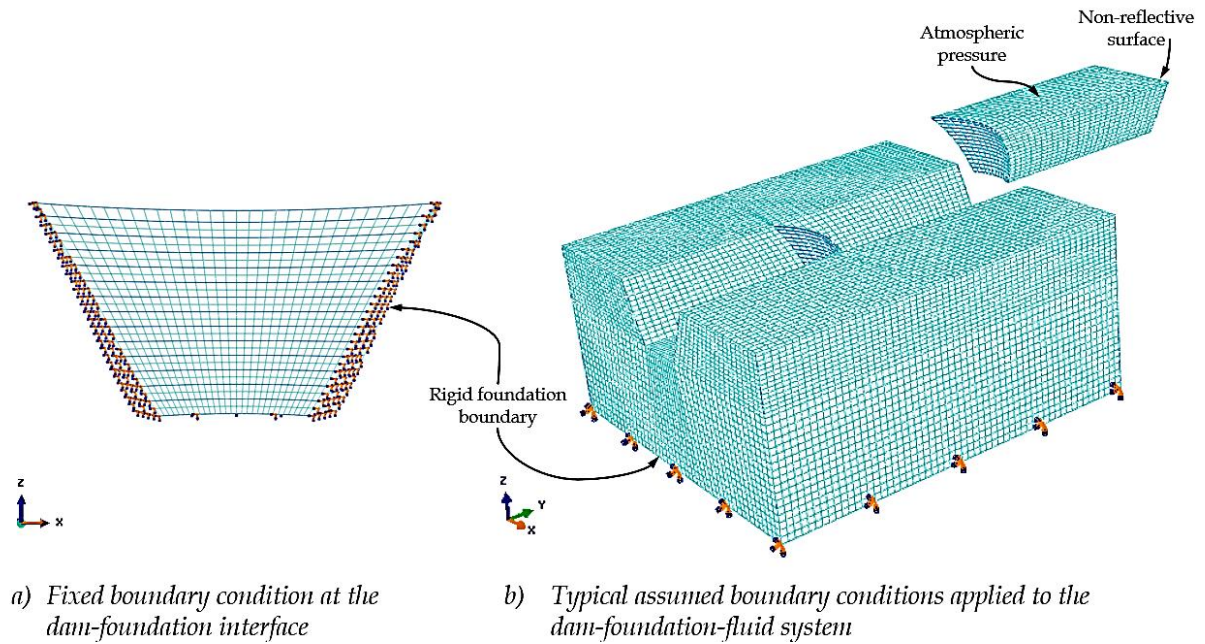


Figure 4.13: Illustration of the boundary conditions applied in this study

4.3.10 Model Updating (Calibration and Validation)

The proper calibration of the finite element model is a critical aspect of numerical modelling, as a wrongly calibrated model results in erroneous structural behaviour response predictions. The current practice in FE modelling of dams is to use static and dynamic measurements independently of each other to calibrate these models ((Buffi et al., 2017) (Daneshyar et al., 2021) (Prakash et al., 2022)). In this study, the implemented modelling updating approach consists of three steps for concrete arch dams using actual observations:

- i. Calibration of the dynamic properties using the modal properties obtained from ambient vibration testing or monitoring to update the elastic properties that relate to the stiffness of both the dam wall and rock foundation and the mass of the dam wall,
- ii. Thermal calibration, using observed climatic data as well as the temperature monitoring data of the structure and reservoir (if available) to update the thermal parameters of the model,
- iii. Deformation calibration using monitoring data from pendulum displacements, or geodetic measurements, and/or GPS systems to update the coefficient of thermal expansion and closure temperature, as the static behaviour of concrete arch dams

is largely driven by thermal loads in areas with large seasonal temperature variations (Hattingh et al., 2019).

For dam structures experiencing swelling due to ASR an additional calibration step is added, normally before and/or together with the deformation calibration but after the thermal calibration step as ASR is temperature dependent and contributes significantly to the deformation of the dam structure:

- iv. ASR calibration normally achieved using accelerated laboratory tests and/or observed site measurements to update the parameters used to define an assumed expansion curve of ASR.

Furthermore, it should be noted that water plays a significant role in all aspects of the dam behaviour. Thus, once the model parameters have been calibrated, the model is validated by comparing the behaviour of the model to the actual measured behaviour of the dam at a particular water level and at different time periods to those used for the calibration. The model is successfully validated if there is agreement between the two sets of data.

4.4 Loading Considerations for Concrete Arch Dams in Operation.

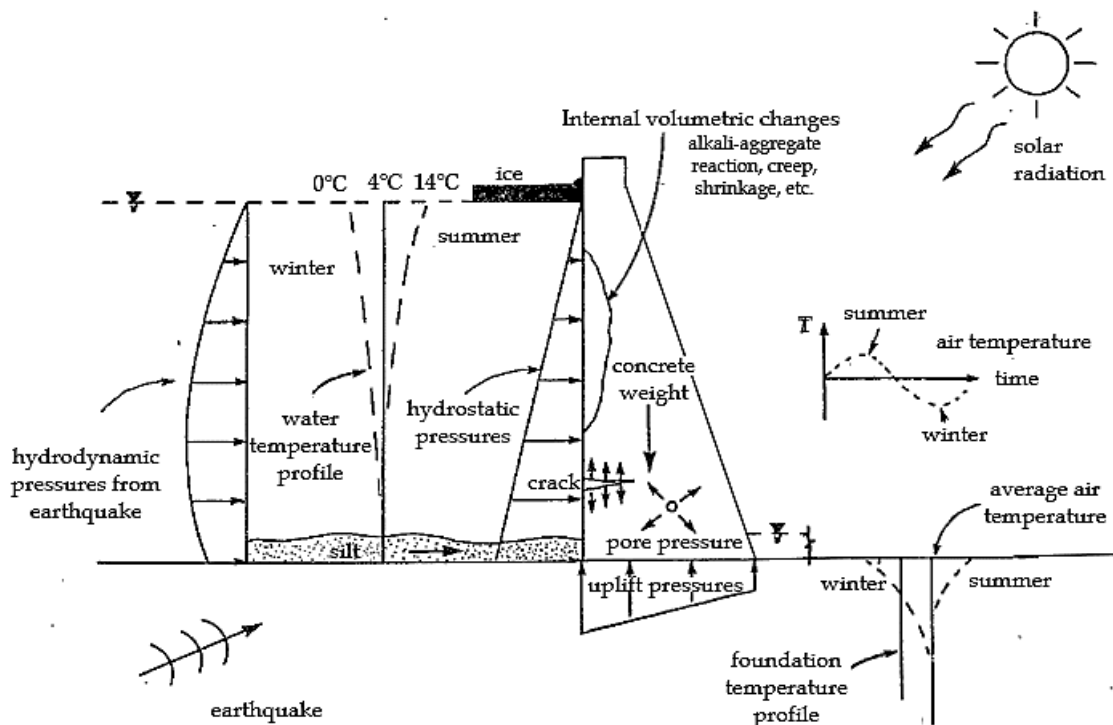


Figure 4.14: Illustration of different loads concrete dams in operation might experience (adapted from Leger et al., 1993)

A dam in service experiences different load combinations some of which vary spatially and/or temporarily. Some of these loads include gravity, ice, hydrostatic, seismic and temperature loads. Figure 4.14 illustrates the different loading experienced by concrete dams in operation. Depending on the location of the dam, the load combinations imposed on the structure may include some or all the loads shown in Figure 4.14. For example, some dams may be subject to ice loads, others subject to earthquakes, and for dams in regions with high temperature variations thermal stresses play a significant role in the behaviour and deterioration of the dam structure ((Enzell & Tollsten, 2017) (Agullo et al., 1996) (Leger et al., 1993)). The loads pertinent to this study include, gravity loads, water load, thermal load and loading due to ASR.

4.4.1 Gravity loads

Gravity loads consists of the loading due to the weight of concrete and any structural elements within/on the concrete. This loading is important for both the static and dynamic analysis of arch dams.

4.4.2 Water load

The impounded water results in significant loading on the structure for both static analysis (hydrostatic load) and dynamic analysis (hydrodynamic pressures in an earthquake and due to the fluid-structure interaction). The variation of the load is due to the quantity of the water in the reservoir, which generally varies seasonally. A full dam deflects downstream with the deflected dam wall approaching its base state as the reservoir level drops. This leads to a cyclic stress distribution in the dam. Similarly, a dam experiences its lowest natural frequencies when full and its highest frequencies at low reservoir levels. Additionally, the dam experiences higher reservoir damping when the dam is full than when it is empty (Chopra, 2020).

4.4.3 Temperature loading

Temperature loads arise due to differences between the closure temperature (base temperature state after grouting of construction joints) and the concrete temperatures during operation of the dam. Temperature analysis is consequently carried out for the construction phase until the establishment of the base state and operational thermal analysis. The latter will be the focus of this study. Temperature loading on the concrete dam during operation is dependent on the solar radiation, air temperature and reservoir temperature. The temperature analysis considerations for the performance assessment of concrete arch dams are broadly discussed in

section 4.5 below.

4.4.4 Internal Stress due to Alkali-Silica Reaction

Swelling due to Alkali-Silica Reaction (ASR) is another load that negatively affects the integrity of the dam structure as the resulting swelling is irreversible and may ultimately result in the loss of structural integrity and/or subsequent failure (refer to Chapter 2).

4.5 Heat Transfer Modelling Considerations for Concrete Arch Dams in Operation

Thermal stresses normally dominate seasonal stress variation in concrete arch dams located in areas with large seasonal temperature changes. Additionally, the concrete temperature and distribution influence the creep mechanism, the reaction kinetics, and the rate of volumetric expansion for dam structures affected by ASR. A sound understanding of the temperature field and its spatial distribution in concrete arch dams is essential for informed dam safety assessment. Heat analysis is accomplished thru three main temperature processes (i) conduction, (ii) convection and (iii) radiation. In FEM-based temperature analysis, mathematical models that require measured data to verify analytical results are used to model heating sources. Thus, it is beneficial to perform heat transfer analysis on dams with temperature monitoring data to validate these models (Nzuza, 2013).

4.5.1 Fourier Heat Conduction model

The Fourier heat conduction model assumed to govern heat transfer in concrete dam systems is given as follows:

$$\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} + \frac{\partial^2 T}{\partial z^2} + \frac{Q}{k} = \frac{1}{D_m} \frac{\partial T}{\partial t} \quad (4.1)$$

$$\Delta^2 T + \frac{Q}{k} = \frac{1}{D_m} \frac{\partial T}{\partial t} \quad (4.2)$$

where T (°C) is the 3D temperature field, Q (kJ/m³h) is the heat generation rate per unit volume, Δ is the Laplace operator for temperature, k (W/m°C) is the heat conduction coefficient and t is time,

in which:

$$D_m = \frac{k}{\rho c} \quad (4.3)$$

where ρ (kg/m^3) is the density and c ($\text{kJ/kg}^\circ\text{C}$) specific heat.

In transient heat transfer analysis for concrete dams operational for 10 years or more, the hydration process is assumed complete, thus Q_h is assumed to be negligible. Thus, the Fourier heat conduction model reduces to:

$$\Delta^2 T = \frac{1}{D_m} \frac{\partial T}{\partial t} \quad (4.4)$$

((Saetta et al, 1995) (Meri, 2011)).

Equation (4.4) is used to establish the cross-sectional temperature distribution of concrete dams in service. This model considers the spatial distribution of the sun's rays incidence at any position on the surface of the concrete. Additionally, the model considers heat transfer in the three global axis directions tracing the complex arch dam geometry (Nzuza, 2013).

The boundary conditions associated with this analysis are considered next.

4.5.2 Boundary Conditions

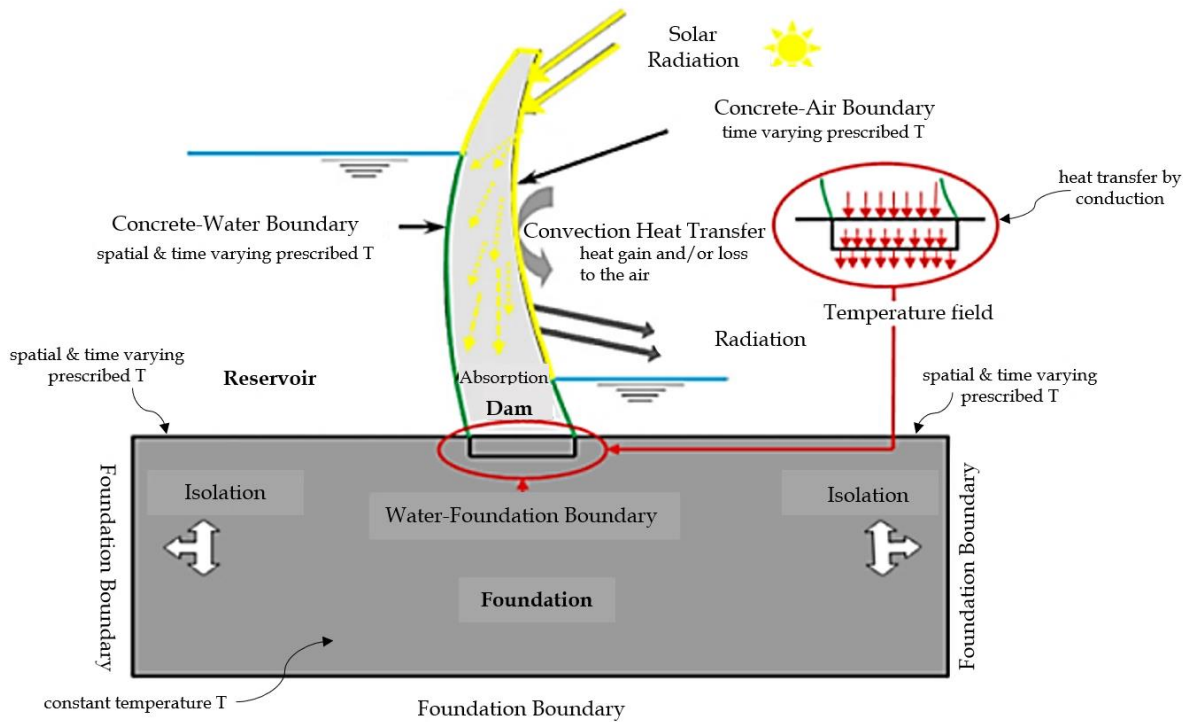


Figure 4.15: Idealised temperature contact boundaries of a typical arch dam in operation (adopted from Nzuza, 2013)

The thermal contact boundary conditions are represented by (i) the concrete-air boundary, (ii) concrete-water boundary, (iii) wall-foundation boundary and (iv) the foundation boundary. Figure 4.15 shows a graphic representation of the idealised contact temperature boundaries using a typical concrete arch dam section.

Concrete-water boundary

The concrete-water boundary refers to the interface between the upstream surface of the dam wall with the reservoir and/or downstream dam wall surface and the water in contact with it (Figure 4.15). The temperature at this boundary interface is determined using temperature models or through observations from thermometers placed in the upstream surface (Agullo et al., 1996). The main assumption is that the concrete and water temperature variations are the same at this boundary with no convection and/or radiation occurring at this boundary. For the full supply water level, the concrete-water boundary can be divided into three sections with unique temperature amplitudes determined using water temperature models such as the one by Bafong (1997), applied to each section to account for the variation in temperature along the depth of the reservoir.

Concrete-air boundary

The concrete-air boundary refers to the boundary between the dam wall surface and the surrounding air that causes cooling of the dam structure via heat transfer. The gain and loss of heat periodically varies in the dam wall due to seasonal changes. For example, in areas with winter rains and dry summers, in summer when the water level is low, the exposed surface of the dam wall is the greatest whilst in winter the reverse occurs. Consequently, the temperature field patterns applied on the upstream and downstream faces will be different. Additionally, the variation of heat loss and gain is subject to the complex geometry of concrete arch dams at this boundary interface (Ghaemian & Sheibany, 2006). The heat transfer at this boundary interface is a function of air temperature (established with temperature models e.g. equation (4.5), or experimentally by measuring or monitoring the ambient air temperature), concrete temperature, wind speed and solar radiation intensity. Thus, the boundary conditions applied at this boundary interface include: (i) heat flow from the sun (ii) convection (refer to section 4.5.3) and (iii) radiation (refer to section 4.5.4):

$$T_{\text{air}} = \left(\frac{T_{\text{max}} - T_{\text{min}}}{2} \right) \sin \left(2\pi \frac{t - \left(\frac{h_{\text{max}} + h_{\text{min}}}{2} \right)}{2(t_{\text{max}} - t_{\text{min}})} \right) + \left(\frac{T_{\text{max}} + T_{\text{min}}}{2} \right) \quad (4.5)$$

where $T_{\max}$ is the maximum daily temperature and $t_{\max}$ the corresponding time, $T_{\min}$ is the minimum daily temperature and $t_{\min}$ the corresponding time (Agullo et al., 1996).

Concrete wall-foundation boundary

The dam wall-foundation interface, also referred to as the adiabatic boundary, is analysed assuming earth temperatures with zero thermal flux at high depths. Thus, the temperature at this boundary may be defined using a constant temperature ((Ghaemian & Sheibany, 2006) (Milano & Leonardo, 2008)).

4.5.3 Convection heat transfer

The temperature difference between the concrete surface and the surrounding ambient temperature results in the exchange of heat through (heat gain or loss) to the surrounding air due to convection given by Newton's law of cooling:

$$q_c = -h_c(T_{\text{air}} - T) \quad (4.6)$$

where h_c ($\text{W}/\text{m}^2\text{K}$) is the convection coefficient, T_{air} is the temperature of the surrounding air in K and T the temperature of the concrete surface also in K. The surface temperature of a dam due to convection heat transfer normally follows the variation of the air temperature (Polivka and Wilson, 1976).

4.5.4 Solar radiation and radiative heat transfer

Solar radiation is the energy transferred from the sun through electromagnetic waves with varying wavelengths on the dam wall surfaces. It is mainly subject to the variability of global irradiance (direct radiation), diffuse irradiance (diffuse radiation), horizontal direct irradiance (reflected radiation) and sunshine duration ((Janna, 1986) (Power & Mills, 2005). Moreover, the geometry of arch dam structures affects the quantity of irradiance that gets to the dam's surfaces, determined thru the sunrays angle of incidence. Consequently, the resulting thermal loading on the dam structure is non-linear.

The solar irradiance in concrete dams is estimated by summing the global irradiance (H_o), direct irradiance (H_b), and diffuse irradiance (H_d), which are estimated from experimental data or via analytical analysis that accounts for the varying dam geometry and the terrain of the site (Power & Mills, 2005).

Solar energy in arch dams is controlled by the opaqueness of the dam surface and is affected by the radiant attributes of the concrete material that control to heat gain in the dam wall, namely: absorptivity, emissivity, reflectivity, and transmissivity (Nzuza, 2013).

Emissivity refers to the capability of surfaces to release radiant energy. The opacity of concrete surfaces in arch dams allow for the absorption and radiation of solar energy (Incropera & Dewitt, 2002).

Reflectivity also known as solar reflectance refers to the ratio of the reflected solar radiation to the total (incident) solar radiation that the concrete surface may be exposed to, varying from 0, for perfect absorbing surfaces, to 1, for surfaces that perfectly reflect (ACPA, 2002).

Absorptivity has to do with the portion of the incident radiant energy that is absorbed by the dam surface (Janna, 1986).

Transmissivity refers to the portion of the total radiant energy that passes through the film closest to an exposed surface (Nzuza, 2013).

The absorbed direct solar energy from the sun, q_s (W/m²), is given by:

$$q_s = aI_t \quad (4.7)$$

where a is the surface solar absorptivity, and I_t is the incident solar energy on the surface.

Whilst the emitted radiant energy (thermal radiation) can be measured using the Stefan-Boltzmann law:

$$q_r = eC_s(T_{air}^4 - T^4) \quad (4.8)$$

where e is the surface emissivity and C_s , the Stefan-Boltzmann constant specified as 5.669×10^{-8} W/m².

4.5.5 Finite element implementation of heat transfer

The heat conduction analysis of concrete dams is calculated in all three directions i.e. in the x-, y-, and z-directions and the analysis model (equation (4.4)) takes the form:

$$k_x \frac{\partial^2 T}{\partial x^2} + k_y \frac{\partial^2 T}{\partial y^2} + k_z \frac{\partial^2 T}{\partial z^2} = \rho c \frac{\partial T}{\partial t} \quad (4.9)$$

where T refers to the temperature of the concrete in K; k_x, k_y, k_z are the conductivity coefficients of the concrete in x-, y-, and z- directions.

The three-dimensional heat model (equation (4.9)) is the best to evaluate the temperature distribution, heat flows and thermal stressing requirements within the concrete arch dam structure. The matrix equation for the heat flow equilibrium to determine the nodal temperatures $\{T(t)\}$, (Leger et al., 1993) is expressed as:

$$[c]\{T(t)\} + [k]\{T(t)\} = \{Q(t)\}_e + \{Q(t)\}_c + \{Q(t)\}_r \quad (4.10)$$

where $[c]$ is the matrix of the heat capacity of the system that is a function of ρ and c , $[k]$ is the thermal conductivity matrix of the system, $\{Q(t)\}_e$ is the heat flux provided externally, $\{Q(t)\}_c$ is the surface-based convective heat transfer, and $\{Q(t)\}_r$ is the surface-based radiative heat transfer.

The radiation boundary condition introduces nonlinearity resulting in the solution solved for iteratively at each time step.

Thermal displacements are then computed from thermal strains ($\epsilon = \partial d / \partial n$) as follows:

$$T(x,y,z) = \int \frac{\partial d}{\partial n} \partial \Omega \quad (4.11)$$

and the thermal stresses within the dam evaluated as follows:

$$\sigma = E\epsilon \quad (4.12)$$

The assessment of thermal stresses in concrete arch dams is based on the temperature distribution, the material properties of the concrete, and the support conditions (Leger et al., 1993). Consequently, critical temperature states need to be considered when evaluating the safety of concrete dams.

4.6 ASR Modelling Considerations for Concrete Arch Dams in Operation

Different ASR modelling approaches exist as has been extensively discussed in chapter 2. The thermo-chemo-mechanical constitutive model proposed by Saouma and Perotti (2006) that integrates the reaction kinetics and diffusion process to estimate the volumetric expansion of concrete as expressed in equation (4.13), was implemented in this study:

$$\dot{\epsilon}_V^{ASR}(t, T) = \underbrace{\Gamma_t(f'_t | \omega_c, \sigma_I | COD_{max}) \Gamma_c(\bar{\sigma}, f'_c)}_{\text{Retardation}} \underbrace{g(q)}_{\text{Humidity}} \underbrace{\dot{\xi}(t, T)}_{\text{Kinetics}} \underbrace{\epsilon(\infty)}_{\text{ASR Strain}} \quad (4.13)$$

The reaction kinetics and diffusion process in the implemented model were based on the research conducted by Larive (1998). The volumetric expansion is assumed to be based on the reaction extent and observing the chemical reaction kinetics illustrated in Figure 4.16. The essential variables in defining the curve include the characteristic time (τ_c), latency time (τ_l), and the AAR ultimate strain which are normally estimated for each structure using accelerated laboratory tests.

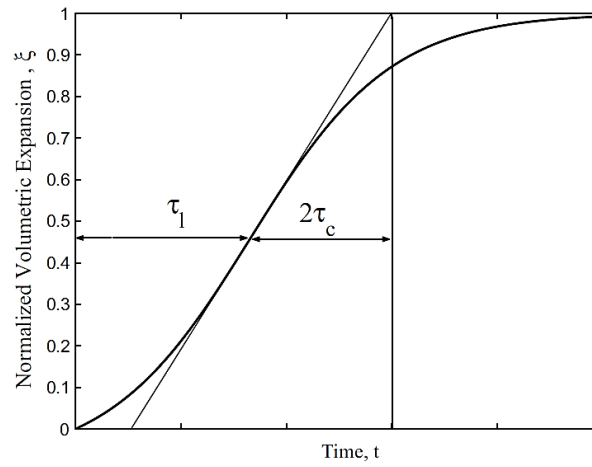


Figure 4.16: Normalised expansion curve (adopted from Saouma et al., 2014)

The constitutive model accounts for the retardation in ASR related expansion due to the presence of cracks and/or triaxial compressive stress. The development of cracks provides spaces fillable by the expanding gel, thereby relieving the expanding gel induced stresses. Whilst, as already established, ASR related expansion is sensitive to confining stress which tend to prevent or limit expansion. Both mechanisms ultimately lead to the reduction in the final volumetric expansion. Additionally, the constitutive model also accounts for the effect of stress distribution by redistributing the volumetric ASR strain to the three principal directions as a function of the respective stresses along the principal direction(s). Consequently, the model results in anisotropic ASR due to the weighting allocated to each of the principal directions (Saouma, 2014).

The resulting mechanical property deterioration of the concrete material due to ASR expansion is incorporated in the constitutive model through nonlinear time-dependent models illustrated in Figure 4.17 and defined using equations (4.14) and (4.15).

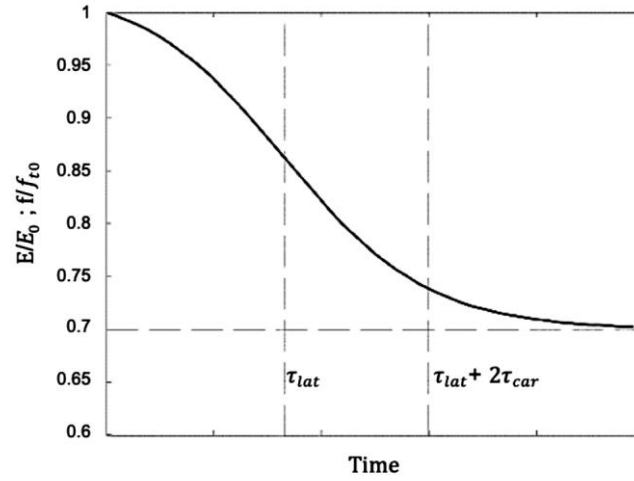


Figure 4.17: Degradation of E and f_t' (adopted from Saouma & Perotti, 2006)

$$E(t, E) = E_0 [1 - (1 - \beta_E) \xi(t, T)] \quad (4.14)$$

$$E(t, T) = f_{t,0}' [1 - (1 - \beta_f) \xi(t, T)] \quad (4.15)$$

The progression of the reaction is affected by the availability of moisture. Consequently, the effect of Humidity on the reaction is considered in the constitutive through a reduction function (equation (4.16)).

$$0 \leq g(q) \leq \frac{\epsilon(t=\infty, RH=q)}{\epsilon(t=\infty, RH=1)} \quad (4.16)$$

where $g(q)$ (equation (4.17)) is an expression as a function of the relative humidity (q) that reduces the maximum expansion as RH lowers.

$$g(q) = ae^{b \cdot q} \quad (4.17)$$

where a and b are coefficients with values of 0.0002917 and 8.156 respectively (Saouma, 2014).

Below an RH value of 0.8, it is widely accepted that ASR is severely inhibited. (Saouma, 2014). Consequently, it is essential to determine the spatial distribution of RH within a structure (i.e. $RH(t, x, y, z)$). However, the RH within a dam structure generally lies between 90-95%. Thus, $g(q)$ is assumed to be 1 within a dam irrespective of the temperature distribution (Saouma & Perotti, 2006).

The ASR modelling approach discussed here was implemented in ABAQUS using a subroutine developed by Rodriguez et al. (2011). (For further details on the implementation of the subroutine, please refer to Rodriguez et al. (2011) and Nyoni (2018)).

4.7 Dynamic Behaviour Analysis of Arch Dams under Ambient Conditions

The dynamic behaviour of arch dams has most commonly been evaluated in the consideration of seismic design and monitoring, which involves the analysis of the full dynamic problem whose solution involves the pressures and displacements in the upstream face ((Clough, 1980) (Fenves et al., 1992) (Proulx, 2004)). More recently, it is considered in structural health monitoring of aging concrete dam structures whereby the evaluation of the structure includes calibrating an FE model using ambient vibration testing/monitoring results to provide an understanding of a dam structure's current condition ((Dabre & Proulx, 2002) (Gomes & Lemos, 2020) (Calcina et al., 2014)), which is the focus of this research. This study concerns the extraction of the natural frequencies and mode shapes.

The dynamic properties intrinsic to the structural system assuming free-undamped-vibration “modes” of the dam, can be determined using equation (3.64) (here shown as equation (4.18)).

$$[\mathbf{M}]\{\ddot{\mathbf{u}}\} + [\mathbf{K}]\{\mathbf{u}\} = \{\mathbf{0}\} \quad (4.18)$$

where $[\mathbf{M}]$ & $[\mathbf{K}]$ are the mass matrix & stiffness matrix respectively, and $\mathbf{u}(x, y, z, t)$ is a relative displacement vector, the over-dots denote the derivatives of the displacement with respect to time.

whose solution

$$\{\mathbf{u}\} = \{\boldsymbol{\phi}\}e^{i\omega t} \quad (4.19)$$

where ω is the natural frequency, $\{\boldsymbol{\phi}\}$ the mass normalised mode shape and t represents time, leading to the Eigen problem:

$$([\mathbf{K}] - \omega^2[\mathbf{M}])\{\boldsymbol{\phi}\} = \{\mathbf{0}\} \quad (4.20)$$

(refer to section 3.3.2).

The dynamic behaviour of concrete arch dams involves the interaction of three continua (i.e., coupled problem) i.e., the dam structure interacts with the deformable foundation that supports it and with the water impounded in the reservoir, (dam-foundation-reservoir

system), both of which extend to large distances ((Pedro, 1999) (Chopra, 2020)). The development of procedures for the seismic analysis of the dam-foundation-reservoir system that recognise the energy loss, among other things, due to the large extent of the fluid and foundation were the subject of research between 1970 to about the late 1990s (Chopra, 2020). An expedient solution that considered a limited size of the foundation in which a massless foundation was assumed, and the hydrodynamic effects modelled by an added mass of the water moving with the dam, was developed by Clough (1980). Due to its easy implementation in commercial FE software, this modelling approach became popular. However, for seismic analysis the assumption of a massless foundation and incompressible fluid among other assumptions, meant that the proposed solution solved a very different problem from the real problem (Chopra, 2020). Considerations to determine the dynamic response of the dam-foundation-reservoir system of concrete arch dams under ambient conditions to achieve a solution for the real problem here follow:

4.7.1 Dam-Foundation System

Arch dams are constructed to transmit all their loads onto the founding (rock) material of the canyon. Consequently, the rock foundation has a substantial impact on the response of a concrete arch dam. An ideal foundation incorporates all geological characteristics and stretches a length such that the influence of the boundary does not affect the static and dynamic characteristics of the dam structure (Vezi, 2014). In studies reported by Chopra (2020), foundation sizes of $5H \times 5H \times 3H$ were used together with viscous damper boundaries on the sides and bottom of the rock foundation applied for seismic analysis. Whilst the USACE (1994), based on Clough (1980)'s solution, recommend a massless elliptical shaped flexible rock foundation with a radius equal to one dam height in the case where the foundation stiffness is greater than half the modulus of concrete, otherwise, a radius of two times the dam height is suitable. The assumption of a massless foundation leads to an overestimate of earthquake responses, which is problematic for a seismic analysis but is regarded negligible for the extraction of modal parameters under ambient conditions (Vezi, 2014). It is important to note that a massless foundation only plays a part in the stiffness and not the mass of the system (equation (4.21)).

$$\left(([\mathbf{K}]_{\text{concrete}} + [\mathbf{K}]_{\text{foundation}}) - \omega^2 [\mathbf{M}] \right) \{\boldsymbol{\phi}\} = \{0\} \quad (4.21)$$

In this study, massless flexible foundations of approximately 5H x 5H x 3H were used without the application of viscous damper boundaries (refer to section 4.3.5 and 4.3.9).

4.7.2 Dam-Fluid Interaction (Fluid-Structure Interaction Method)

The impounded water body interacts with the concrete dam wall during a dynamic event i.e., they cannot be assessed separately (decoupled) from each other (Zienkiewics & Taylor, 2000). The factors that affect this interaction include the reservoir geometry which depends on the topology of the site and energy absorption at the bottom of the reservoir due to varying layers of bare bedrock, alluvium silt and other sedimentary material (Chopra, 2020). For seismic analysis, the energy absorption at the bottom of the reservoir results in the loss of energy due to wave absorption at the bottom of the reservoir. Consequently, the fundamental resonant frequency peak of the reservoir is reduced, whilst higher resonant frequencies are effectively eliminated. However, the absorption of energy at the bottom of the reservoir does not affect the natural frequencies of the impounded water (Chopra, 2020) and is thus of little concern for ambient analysis.

As already alluded to above, the fluid-structure interaction causes hydrodynamic pressures at the upstream interface affecting the deformation of the dam wall. In turn, this affects the pressure (coupled pressure-displacement relationship) (USACE, 2003). The full expression of the fluid-structure interaction results in frequency-dependent hydrodynamic pressures taken as an added force, mass, and damping (Chopra, 1967). The added force and damping do not affect the dynamic properties (i.e., natural frequencies and mode shapes) significantly. Thus, the mass of the structure is the only variable that is changed by the fluid-structure interaction. Thus, the Eigen problem to be solved becomes (Chopra, 1967):

$$\left(([\mathbf{K}]_{\text{concrete}} + [\mathbf{K}]_{\text{foundation}}) - \omega^2 ([\mathbf{M}]_{\text{concrete}} + [\mathbf{M}]_{\text{water}}) \right) \{\boldsymbol{\Phi}\} = \{\mathbf{0}\} \quad (4.22).$$

The Westergaard method and the fluid-structure coupled FE added hydrodynamic mass models are the most widely used added mass formulations appropriate for arch dams. Both approaches first estimate the hydrodynamic pressure to be transformed to an added mass, although these methods estimate the added mass differently ((Westergaard, 1933) (Kuo, 1982) (Chopra 1967) (Rodriguez et al., 2012)). Other approaches which can be used to evaluate the fluid-structure interaction problem in arch dams include the Lagrangian and Eulerian finite element-based approaches. In the latter method, the displacements are considered parameters

in the dam wall and pressures as fluid parameters, whilst for the former approach displacements are both considered fluid variables ((Bayraktar et al., 2011) (Sevim et al., 2010)).

The dynamic evaluation of a fluid-structure interaction problem is a taxing approach due to its complexity. The preferred approach is the coupled problem setup that considers the impounded water body and arch dam, accounting for: (i) the water compressibility, and (ii) the various mechanisms of energy loss (including loss due to radiation of waves in the upstream direction, reflection & refraction at the bottom of the reservoir and/or canyons). Additionally, the preferred method uses compressible finite elements with the relevant boundary conditions applied (Vezi, 2014) (refer to section 4.3.9 for the boundary conditions applied in this study). The movement of water is governed by the three-dimensional wave equation:

$$\frac{\partial^2 \mathbf{p}}{\partial x^2} + \frac{\partial^2 \mathbf{p}}{\partial y^2} + \frac{\partial^2 \mathbf{p}}{\partial z^2} = \frac{1}{V^2} \frac{\partial^2 \mathbf{p}}{\partial t^2} \quad (4.23)$$

where $\mathbf{p}$ (x, y, z, t) is the hydrodynamic pressure (more than the hydrostatic pressure) and V is the velocity of the water pressure waves (Chopra, 2020).

Considering free-undamped vibrations, the coupled system of the dam wall and impounded water, written in the standard Galerkin variation formulation of pressures, $\mathbf{p}$, and displacements, $\mathbf{u}$, is described as:

$$\begin{bmatrix} \mathbf{M}_{\text{concrete}} & 0 \\ \rho_{\text{water}} \mathbf{Q}^T & \mathbf{M}_{\text{water}} \end{bmatrix} \begin{Bmatrix} \ddot{\mathbf{u}} \\ \ddot{\mathbf{p}} \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_{\text{concrete}} + \mathbf{K}_{\text{foundation}} & -\mathbf{Q} \\ 0 & \mathbf{K}_{\text{water}} \end{bmatrix} \begin{Bmatrix} \mathbf{u} \\ \mathbf{p} \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \quad (4.24)$$

where $\mathbf{M}$ and $\mathbf{K}$ have the usual meaning they have in structural dynamics, ρ_{water} is the density of water and $\mathbf{Q}$ is the transformation matrix that converts nodal pressures into hydrodynamic forces ((Tiliouine & Seghir, 1998) (Zienkiewicz et al., 2005)).

Equation (4.24) is clearly not symmetric nor positive definite. However, physically, the eigenvalues are observed to be real and the free vibration modes exist. It is, therefore, possible through several operations to arrive at a symmetrical form and minimise the problem to a standard eigenvalue one (Zienkiewicz et al., 2005). This was achieved by using the Lanczos formulation to obtain a symmetric equation which was then solved using the Lanczos eigen solver for the extraction of eigenvalues and corresponding eigenvectors in this study.

4.8 Current State of Practice regarding the Use of Dam Surveillance Data in the Modelling of Concrete Arch Dams

The accuracy and reliability of numerical models for structural identification, condition and safety assessments of concrete arch dams depend on how well surveillance data is incorporated in the modelling process. A lot of parameters essential for modelling are obtained from monitoring systems (ICOLD, 2013). Usually for expediency, the modelling of different physical actions on concrete dams is carried out separately and independently in a non-interacting manner assuming linear superposition of the different analysis observations. The dam engineer is therefore required to synthesize all the partial analyses results to assess the safety of a structure (ICOLD, 2001). However, there has been a tendency towards the development and use of more sophisticated models that allow for the consideration of the nonlinear effects (ICOLD, 2001) and interaction (Hariri-Ardebili & Kianoush, 2014) of some physical actions due in part to increasing computational power and technological advancements that have resulted in more digital-focused trends of managing and performance assessment of structures (Chiachio et al., 2022). For example, Hariri-Ardebili & Kianoush (2014) are among the first to report on an integrative safety assessments approach for concrete arch dams that was focussed on seismic assessment. Whilst some of the sophisticated analysis require input from the modelling of other physical process such as temperature modelling for ASR analysis (Saouma, 2014). Some experiences on static and dynamic model updating are presented in the following section as an illustration of the current state of practice regarding the incorporation of dam surveillance data in the modelling process. Additionally, some observations of modal parameters from the dynamic monitoring of dams are presented, to be used for comparison with the theoretical investigation presented in the following chapter.

4.8.1 Static monitoring data

Monitoring of parameters that relate to the static behaviour of concrete arch dams is standard practice. The parameters of interest in the context of this study include measurements of the prevailing climatic conditions at the dam site including air temperature and wind speed, dam wall and reservoir temperature measurements which are important for the thermal analysis of these structures. Additionally, the water level and displacement measurements are also important for the calibration of the structural model's response to the static loads imposed on it.

Thermal parameter model updating

The thermal analysis models require information about air temperature, wind speeds, orientation of the dam, water temperature, foundation temperature and solar radiation loads (which depend on the dam location (latitude, longitude)), time of the day, month, and year ((Mirzabozorg et al., 2014) (Daneshyar et al., 2021)). For dam structures in operation for more than ten years, the heat transfer analysis consists of a steady-state analysis followed by a transient-state analysis. From which, the temperature distribution in the dam wall is obtained and compared with the measured temperature profile in the concrete to update the dam wall's material properties. The concrete thermal material properties that are typically updated include the specific heat, thermal conductivity, convection coefficient, solar absorptivity, and emissivity ((Mirzabozorg et al., 2014) (Daneshyar et al., 2021)). Another important aspect to check is the phase difference between the measured and calculated temperature distribution.

Displacement model updating

The static related structural calibration goes beyond thermal distribution within the dam to include self-weight, hydrostatic load, silt load and irreversible phenomena such as creep and swelling ((Tatin et al, 2018) (Mirzabozorg et al., 2014)). In the displacement model updating, the dam-foundation-reservoir system's mechanical properties that influence that static behaviour of concrete arch dams are typically calibrated. These include the concrete's elastic modulus and coefficient of thermal expansion (Daneshyar et al., 2021). Mirzabozorg et al. (2014) additionally included the concrete's density, Poisson's ratio, grouting (closure temperature) and the foundation's deformation modulus and Poisson's ratio. However, Meyer & Mouvet (1995) observed that the thermal coefficient of expansion is the most sensitive parameter to update.

4.8.2 Dynamic monitoring data

Dynamic testing of dam structures began in the 1960s initially carried out using forced vibration testing (FVT) techniques (Rouse & Bouwkamp, 1967), whilst the application of ambient vibration testing (AVT) of dams started in the late 1970s to early 1980s ((Calciati et al., 1979) (Ellis et al, 1982) (Clough et al., 1984)). Up until recently, FVT remained the standard approach of dynamic testing of dam structures. The high costs associated with the heavy equipment used in the FVT technique necessitated a shift towards AVT techniques in dynamic testing of dams (Bukanya, 2020). Additionally, developments in vibration measurement technologies such as analogue to digital converters and signal processing algorithms have

made AVT a viable alternative to FVT for dam structures ((Brincker & Ventura, 2015) (Pereira et al, 2020)). Consequently, AVT is now the preferred approach to dynamic testing of dam structures (Prakash, 2022).

Table 4.4 provides a summary of the dynamic characteristics of some concrete arch dams on which dynamic tests are reported. The examples provided in Table 4.4 are not the only tests that have been reported. Some arch dam structures on which FVT tests have been reported upon include Morrow Point Dam in the USA (Mostafaei et al., 2021), Emosson & Mauvoisin dams both in Switzerland ((Darbre & Proulx, 2002)) (Proulx et al., 2004)), Baixo Sabor Dam in Portugal (Gomes et al., 2018). Arch dam structures on which AVT measurements have been reported on include Karun IV Dam in Iran (Tarinejad et al., 2016), Caldeirao dam, Castelo do Bode dam & Santa Luzia Dam in Portugal (Pereira et al, 2020), Fei-Tsui dam in Taiwan (Loh & Wu, 1996) and Tajera dam in Spain (García-Palacios et al., 2016). While no dam-foundation-water system is perfectly symmetric, for most arch dam structures, sufficient symmetry is usually present such that the mode shapes of the system are classified as either symmetric or antisymmetric (Hall, 1988). Only the dominant arch first, second and/or third symmetric or antisymmetric modes are reported in Table 4.4, those mode shapes which might be dominantly cantilever and/or both arch & cantilever dominant modes (complex modes) are not included. The modal parameters presented in Table 4.4 will be used to validate (in a general sense) the theoretical observations presented in the following chapter.

4.8.3 Incorporation of Dynamic monitoring data in

Information from dynamic measurements have been used in different ways including model seismic parameter extraction ((Tarinejad et al., 2016) (Yang et al., 2016) (Oliveira & Alegre, 2019)), model calibration ((Sevim et al., 2010) (Chopra, 2012) (García-Palacios et al., 2016)) and damage diagnosis ((Loh & Wu, 1996) (Proulx et al., 2004) (Alegre et al., 2023)). Only model calibration and damage diagnosis applications will be discussed further as seismic analysis is not part of the scope of this study.

Dynamic Model updating

Typically, measured modal parameters are compared with the calculated modal parameters to calibrate numerical model material parameters (Hariri-Ardebili & Kianoush, 2014). The material properties that are typically updated using dynamic measurement data include the dam wall stiffness, Poisson's ratio, & density and the foundation stiffness & Poisson's ratio ((Proulx et al. 2004) (Sevim et al., 2010) (García-Palacios et al., 2016)).

Table 4.4: Dynamic behaviour characteristics of concrete arch dams in operation

Dam Name (curvature)	Location	h	$l_c:h$	$b_c:h$	Arch classification	Water level below the crest (m)	Mode	Type	Frequency (Hz)	Dynamic test type (source)
Ambiesta Dam (unknown)	Italy	59.0	2.5	0.13	thin	8.0	1	A	4.1	FVT (Hall, 1988)
							2	S	4.7	
							3	S	7.1	
							4	A	8.5	
							5	S	9.3	
Berke Dam (double)	Turkey	201	1.3	0.10	thin	12.5	1	A	2.75	AVT (Sevim et al., 2010)
							2	S	3.41	
							3	S	4.78	
							4	A	5.56	
							5	S	7.94	
Cabril Dam (double)	Portugal	132	2.2	0.15	thin	7	1	A	2.30	AVT (Oliveira & Mendes, 2006)
							2	S	2.38	
							3	S	3.30	
Cahora Bassa Dam (double)	Mozambique	170	1.8	0.14	thin	0(full)	1	A	1.78	AVT (Alegre et al., 2023)
							2	S	2.20	
							3	S	2.65	
							4	A	5.56	
							5	S	7.94	
Fiastra Dam (unknown)	Italy	87.0	2.9	0.36	thick	23.0	1	S	4.72	FVT (Hall, 1988)
							2	A	5.97	
							3	S	7.87	
							4	A	9.72	
							5	S	11.62	

Dam Name (curvature)	Location	h	l_c/h	b_c/h	Arch classificat ion	Water level below the crest (m)	Mode	Type	Frequency (Hz)	Dynamic test type (source)
Kamishiiba Dam (unknown)	Japan	110	2.8	0.25	medium- thick	0 (full)	1	S	3.8	FVT (Hall, 1988)
							2	A	4.3	
							3	S	5.8	
							4	A	7.2	
							5	S	8.7	
Kolnbrein Dam (unknown)	Austria	197	3.2	0.21	medium- thick	197 (empty)	1	A	1.40	FVT (Hall, 1988)
							2	S	1.65	
							3	S	2.01	
							4	A	2.47	
							5	S	3.04	
Kouga Dam (double)	South Africa	72	4.4	0.14	thin	2.7	1	A	3.31	AVT (Moyo, 2013)
							2	S	3.63	
							3	S	4.63	
							4	A	6.07	
Kurobe Dam (unknown)	Japan	186	2.8	0.22	medium- thick	25.0	1	S	2.0	FVT (Hall, 1988)
							2	A	2.4	
							3	S	3.6	
Liuxihe Dam (double)	China	78.0	3.3	0.28	medium- thick	18.0	1	S	6.6	FVT (Hall, 1988)
							2	A	8.4	
							3	S	10.8	
							4	A	13.2	
Morrow Point Dam (double)	USA	142	1.5	0.11	thin	48.0	1	S	3.67	FVT (Mostafaei et al., 2021)
							2	A	3.73	
							3	S	4.52	
							4	S	-	
							5	A	6.67	

4.8 Current State of Practice regarding the Use of Dam Surveillance Data in the Modelling of Concrete Arch Dams

Dam Name (curvature)	Location	h	$l_c:h$	$b_c:h$	Arch classificat ion	Water level below the crest (m)	Mode	Type	Frequency (Hz)	Dynamic test type (source)
Monticello Dam (double)	USA	93.0	3.4	0.32	thick	6.0	1	S	3.12	FVT (Hall, 1988)
							2	A	3.55	
							3	S	4.70	
							4	A	6.00	
Nagawado Dam (unknown)	Japan	155	2.3	0.23	medium- thick	24.0	1	A	3.3	FVT (Hall, 1988)
							2	S	3.6	
							3	S	4.7	
							4	A	5.5	
Punta Gennarta Dam (double)	Italy	57	4.5	0.17	thin	24.3	1	A	4.0	AVT (Calcina et al., 2014)
							2	S	4.8	
							3	-	6.2	
							4	-	7.3	
Quan Shuri Dam (double)	China	80.0	2.6	0.11	thin	4.5	1	A	3.95	AVT (Hall, 1988)
							2	S	4.00	
							3	S	6.90	
							4	S	7.52	
							5	A	8.20	
Roode Elsberg Dam (double)	South Africa	72	3.8	0.13	thin	0(full)	1	A	3.05	AVT (Vezi, 2014)
							2	S	3.23	
							3	S	4.43	
							4	A	5.62	
							5	S	6.50	
Saveh Dam (double)	Iran	128	2.1	0.20	thin	47.5	1	A	3.91	AVT (Mivehchi et al., 2003)
							2	S	4.39	
							3	S	5.76	
							4	S	6.25	
							5	A	7.61	

Dam Name (curvature)	Location	h	$l_c:h$	$b_c:h$	Arch classificat ion	Water level below the crest (m)	Mode	Type	Frequency (Hz)	Dynamic test type (source)
Sazanamigawa Dam (unknown)	Japan	67.0	1.9	0.13	thin	0 (full)	1	A	4.3	FVT (Hall, 1988)
							2	S	5.5	
							3	S	6.8	
							4	A	8.7	
Schlegeis Dam (unknown)	Austria	131	5.5	0.25	medium- thick	37.0	1	A	2.15	FVT (Hall, 1988)
							2	S	2.29	
							3	S	2.75	
							4	A	3.29	
							5	S	3.94	
Shahid Rajaee Dam (double)	Iran	138	3.1	0.19	thin	18	1	A	1.46	AVT (Mivehchi et al., 2003)
							2	S	2.27	
							3	S	2.44	
							4	A	2.93	
							5	S	3.58	
Techi Dam (double)	Taiwan	181	1.6	0.11	thin	17.0	1	A	2.64	AVT (Hall, 1988)
							2	S	3.27	
							3	S	4.60	
							4	A	4.87	
							5	S	5.74	
Tonoyama Dam (unknown)	Japan	64.5	2	0.19	thin	15.5	1	A	5.9	FVT (Hall, 1988)
							2	S	6.2	
							3	S	10.5	
Xiang Hong Dian Dam (single)	China	87.5	4	0.45	thick	16.5	1	S	3.94	AVT (Hall, 1988)
							2	A	4.25	
							3	S	5.05	
							4	A	5.95	
							5	S	6.87	

Damage diagnosis

Modal parameters of a structure depend on its structural mass and stiffness (Gaftoi et al., 2021). Consequently, any change in either of the structural properties will result in a change in the structure's modal properties. Consequently, vibration measurements potentially provide a non-destructive technique to assess the structural and material characteristic changes of a structure, enabling the monitoring of the safety of ageing dam structures (Gaftoi et al., 2021). Alegre et al. (2023) used this idea to evaluate the effect of ASR on an arch dam that has been in operation for about 50 years.

4.9 Research Study's Methodology Overview and Rationale

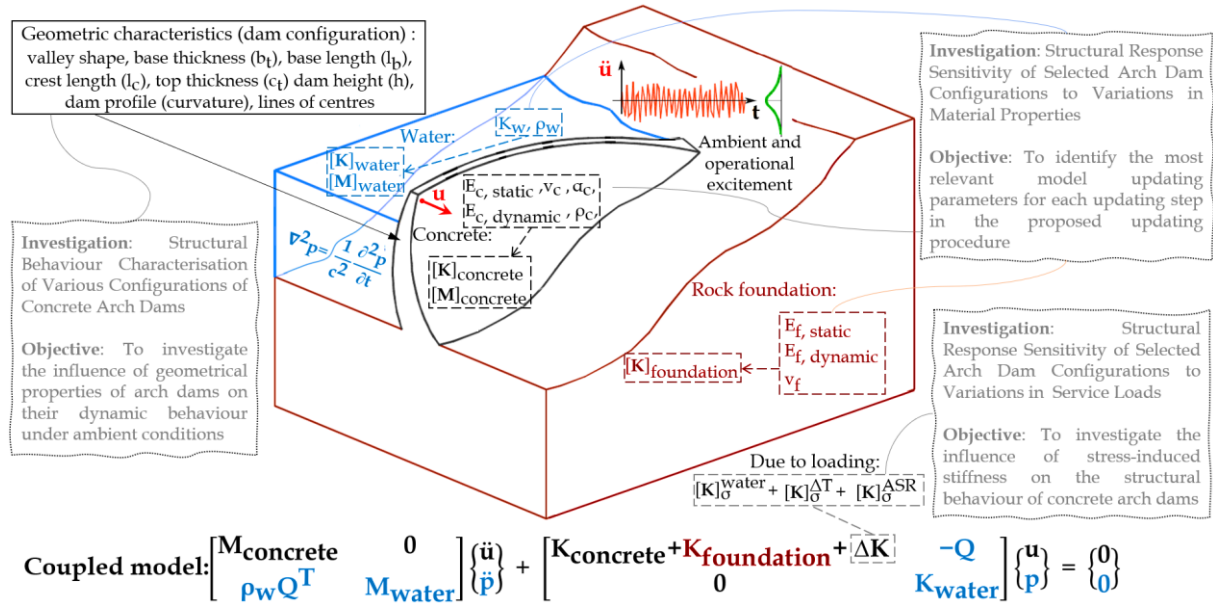


Figure 4.18: Overview of study methodology approach (adapted from Oliveira et al., 2012)

Arch dams are complicated structures with many variables to carefully consider in their analysis as has been discussed in the preceding sections of this chapter. Figure 4.18 illustrates the different aspects considered in this study using the example of the coupled fluid-structure interaction formulation for the dynamic analysis used in this study. From Figure 4.18, the importance of the geometric characteristics and material properties in building the model's mass and stiffness matrices used in determining the structure's response to different load cases is apparent. To meet the objectives of this study:

- i. The influence of geometric characteristics on the structural behaviour of concrete arch dams is investigated and the results presented in Chapter 5 of this dissertation,

- ii. The sensitivity of the structural behaviour of concrete arch dams to variations in loading conditions and material properties is the subject of Chapter 6,
- iii. Based on the observations from Chapters 5 and 6, a finite element model updating procedure for concrete arch dams affected by ASR is proposed and implemented on a case study dam (Chapter 7).

4.10 Chapter Summary

In this Chapter, a broad overview and description of critical components of the methodology by which this study was undertaken is provided. The aspects of the study's methodology that are covered include the model development (including geometry development, application of the material properties and boundary conditions), load application and the general updating required for operational concrete arch dams (which is the main focus of this study). The key contribution of this chapter is to highlight the interaction of these different components of FEM based assessment of concrete arch dams, in relation to their implementation to meet the objectives of this study.

A key observation to highlight is the role surveillance data plays in the modelling process. In addition to the use of surveillance data to monitor the structural integrity of dams particularly in the consideration of damage diagnosis, the critical use surveillance data plays towards achieving reliable and accurate models of the current state of behaviour of dams through calibration has been further illustrated and reiterated in this Chapter. The current state of practice concerning modelling (and by implication model calibration) of physical actions on concrete arch dams is to carry them out separately and independently in a non-interacting manner assuming linear superposition of the different analysis observations. Except for some sophisticated nonlinear analysis where there is an unavoidable interaction between different physical actions affecting the behaviour of concrete arch dams such as ASR progression which is a function of the dam temperature distribution.

Additionally, it has been illustrated in the foregoing discussion that the modelling of concrete arch dams to achieve realistic, representative, reliable and accurate models of operational concrete arch dams is a complex problem as the behaviour of real physical systems of arch dams is influenced by the complex interaction of the dam-foundation-reservoir system components with each other and with their surrounding (result in varying load conditions). There is a need for holistic approaches towards the assessment of concrete arch dams that

consider the complex dam system and its interaction with different physical actions. Integrative safety assessment approaches that combine static behaviour with dynamic behaviour assessment of concrete arch dams provide a possible approach to achieving accurate representation and evaluation of the complex behaviour of concrete arch dams, which is the objective of this study.

Additional observations which will be useful for comparison with the observations in the later chapters of this thesis concerns the parameters that are generally updated for different types of analysis. Generally, the parameters which are updated in the thermal analysis include the concrete thermal material properties namely: specific heat, thermal conductivity, convection coefficient, solar absorptivity, and emissivity. Whilst the calibration of properties using displacement measurements following a thermal analysis has been reported to be carried out for the concrete's elastic modulus, coefficient of thermal expansion, concrete's density, Poisson's ratio, grouting (closure temperature) as well as the foundation's deformation modulus and Poisson's ratio. The parameters normally updated using dynamic monitoring data include: the concrete's stiffness, Poisson's ratio, & density and the foundation stiffness & Poisson's ratio. However, there seems to be no sensitivity study which has been carried out or that has been reported to give guidance as to which parameters are the best to update for each analysis, only inferences for some cases have been made. One of the objectives of this study is to evaluate the sensitivity of the response of arch dams to variations in these parameters (refer to Chapter 6).

One other aspect which has not been sufficiently investigated concerns the evaluation of the dynamic behaviour of concrete arch dam. Although numerous dynamic tests have been carried out on concrete arch dams, there has been no systematic evaluation to characterise the dynamic behaviour of these structures and the parameters that influence the dynamic behaviour of these structure. This study will be among the first studies to systematically, investigate and characterise the dynamic behaviour of various concrete arch dam configurations in an effort to understand the parameters that influence their structural behaviour.

4.11 References

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Chapter 5

CHARACTERISATION OF THE STRUCTURAL BEHAVIOUR OF VARIOUS CONFIGURATIONS OF CONCRETE ARCH DAMS

5.1 Introduction

A comprehensive understanding of the operational structural behaviour of arch dams that includes a comprehension of their form, function, interaction with their surrounding and loading is critical to the development of improved design and health monitoring assessment procedures of these structures (Vezi, 2014) as indicated in Chapter 4. The aim of this study is to improve structural health monitoring assessment of concrete arch dams using numerical models. To this end, the influence of the form of an arch dam on its structural behaviour is investigated in this chapter to meet the objectives (particularly objective (i) (refer to section 1.3)) of this study. The influence of the loading and the interaction of arch dams with their surrounding on their structural behaviour is investigated in the following chapter. Different forms of arch dams have been constructed around the world over the years. Some of the earliest types of arch dam structures were arch-gravity type structures constructed using masonry and, in some cases, together with earth or concrete cores, built by the Romans. Improvements in analysis techniques and understanding of material behaviour has led to the construction of various configurations of arch dams on varied sites (Pedro, 1999). In the preceding chapter, it was observed that the form of an arch dam is generally described based on the site type, the thickness and curvature classification of the arch dam configuration (i.e., geometric characteristics of the dam). This study will be limited to investigating the arch dam configurations described in section 4.2.5.

Additionally, this study does not attempt to cover all possible configurations. The purpose of this study is to understand the general characterisation of the behaviour of various concrete arch dams assuming similar founding conditions and excluding the influence of variations in water level, which are investigated in the following chapter. It is acknowledged that trying to simplify design conditions that are usually distinct for each scenario leads to the lack of design accuracy and optimisation. However, such an approach could have merit in cases where the objective is simply to understand the structural behaviour of different arch dam options and details are not too important, such as in preliminary design stages of new structures.

This chapter proceeds as follows: the methodology aspects specific to this study are discussed first. After which the dynamic behaviour characterisation of the various configurations followed by the static behaviour characterisation of the same configurations are considered next. The chapter ends with a summary discussion of and the implication of the observations.

5.2 Methodology

The model development, meshing and material properties assigned to the models have already been discussed in chapter 4. The aspects relevant to this study are here summarised.

5.2.1 Concrete Arch Dam Configurations Considered in this Study

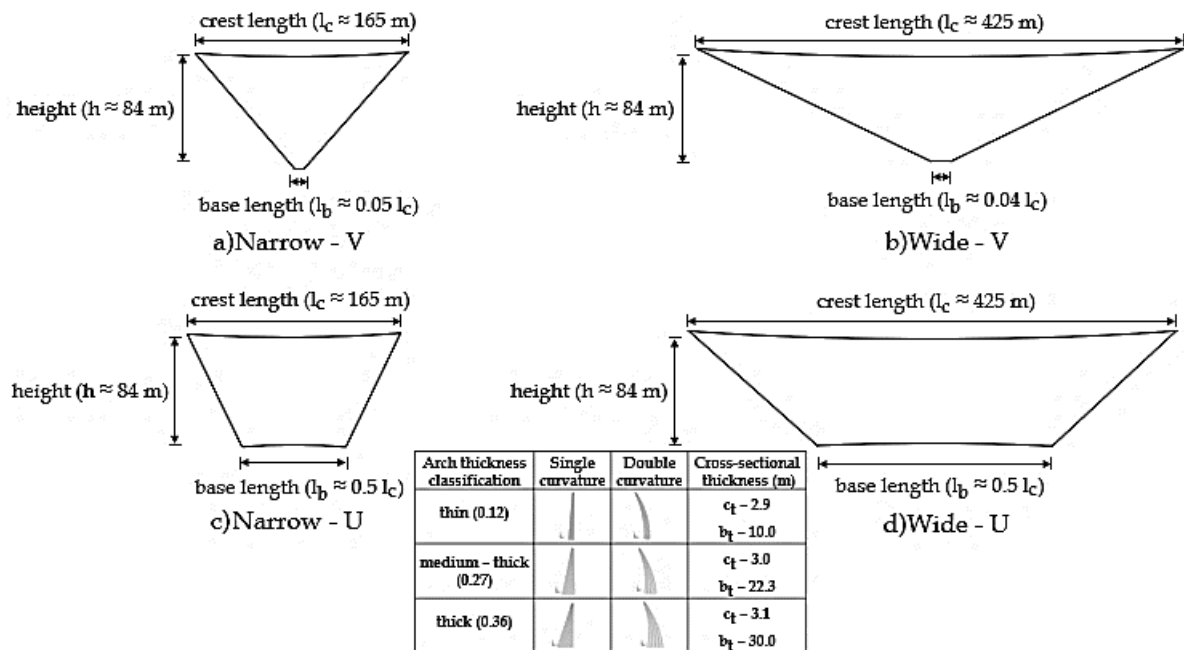


Figure 5.1: Geometric properties of the various arch dam configurations considered in this study

Figure 5.1 shows the different arch dam configurations considered in this study. The description of each configuration's geometric characteristics is based on the four valley types described in Chapter 4. For each valley type, three dam configurations which fall under the three dam thickness classification were developed for both single and double curvature arch dam configurations as follows: (i) thin arch with $b_t: h$ ratio of 0.12, (ii) medium-thick arch with $b_t: h$ ratio of 0.27, and (iii) thick arch with $b_t: h$ ratio of 0.36 (Figure 5.1). Additionally, the crest thickness/width (c_t) and the base width (b_t) are as described in Figure 5.1 below.

5.2.2 Modelling Consideration

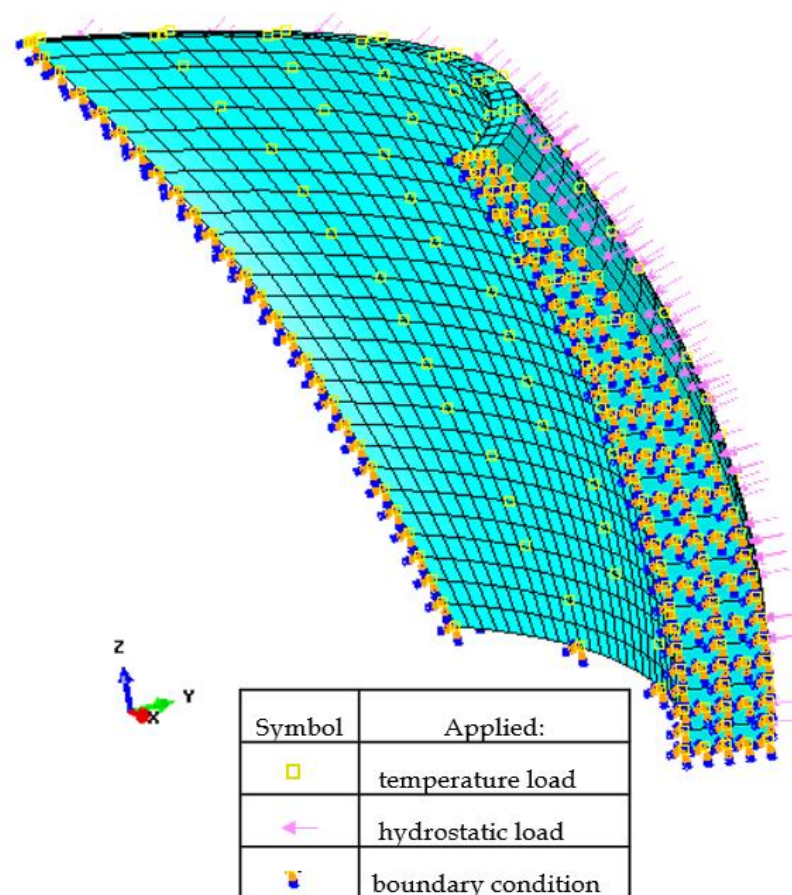


Figure 5.2: Illustration of the applied boundary and load conditions for the developed arch dam configurations

Only the dam wall structure of the different configurations, assuming monolithic structures, are considered in this investigation. A fixed boundary condition (illustrated in Figure 5.2) was assumed at the dam structure-rock foundation interface and the models were meshed using 20-noded isoparametric (C3D20R) (illustrated in Figure 5.2) elements seeded at optimised seed sizes for each configuration. The concrete material properties (i.e., $E_{c, static} = 30$ GPa,

$E_{c, \text{dynamic}} = 39 \text{ GPa}$, $\nu_c = 0.22$, $\rho_c = 2400 \text{ kg/m}^3$, and $\alpha_c = 1.0 \times 10^{-5}$ (Table 4.1)) are assumed to be the same for all arch dam configurations.

The Lanczos method was used to extract the Eigen properties of each dam configuration assuming an empty dam scenario for the dynamic response analysis. Whilst, for the static response analysis under thermal loading and assuming a closure temperature of 15°C , a thermal load, $\Delta T = 10^\circ\text{C}$, was applied throughout the dam cross-section (illustrated in Figure 5.2) for all dam configurations to characterise and compare their static response. Under normal operating conditions, the temperature distribution varies spatially throughout the dam structure's cross-section at any instance, due to factors such as heat transfer, the sun's trajectory, dam configuration and water level among other factors. Applying a constant temperature load, although not likely to ever occur in reality, is a conservative approach which could be used to define the bounds of the expected thermal displacements if one applies the maximum change in temperature expected. Additionally, the static response of each configuration assuming a full dam hydrostatic load scenario was investigated in a separate load step. The static response from the temperature load and hydrostatic load were considered independently of each other (Figure 5.2 shows both loads together for illustration purposes).

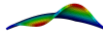
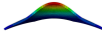
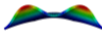
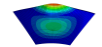
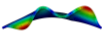
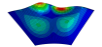
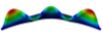
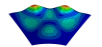
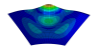
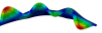
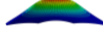
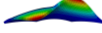
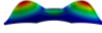
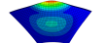
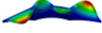
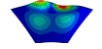
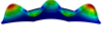
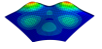
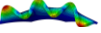
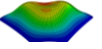
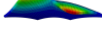
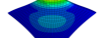
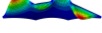
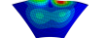
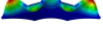
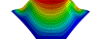
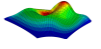
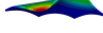
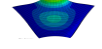
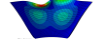
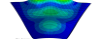
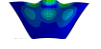
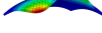
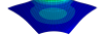
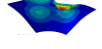
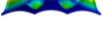
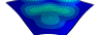
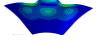
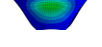
5.3 Dynamic Behaviour Characterisation of Various Arch Dam Configurations

Table 5.1 to Table 5.4 shows the first 10 theoretical mode shapes obtained for the arch dam configurations considered in this discussion. In some cases, the maximum number of mode shapes that could be extracted from the theoretical analysis were less than 10 mode shapes.

5.3.1 Narrow-U shaped valley configurations

Table 5.1 shows the theoretical modes obtained for all the narrow-U shaped valley dam configurations. For the thin arch dam classification for both single curvature and double curvature dam configurations, the first mode shape is the first antisymmetric mode followed by the first symmetric mode. Whilst for the medium-thick and thick classified arch dam configurations, the first mode shape is the first symmetric mode followed by the first antisymmetric mode shape. The third, fourth, fifth, sixth and seventh mode shape sequence is the same for all narrow-U shaped dam configurations (despite the arch dam curvature class or dam thickness class). The first complex mode (cantilever dominant mode shape) appears as the fourth vibration mode shape and the second complex mode (both cantilever & arch dominant mode shape) appears as the sixth vibration mode shape for all dam configurations

Table 5.1: Narrow-U shaped dam configuration(s) theoretical vibration modal parameters

Dam Curvature	Arch class	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Single	thin 	A  4.02 Hz	S  4.20 Hz	S  5.91 Hz	C  6.95 Hz	A  7.69 Hz	C  8.63 Hz	S  10.18 Hz	C  11.55 Hz	C  12.46 Hz	A  13.01 Hz
	medium-thick 	S  5.46 Hz	A  6.43 Hz	S  8.98 Hz	C  10.61 Hz	A  11.83 Hz	C  13.76 Hz	S  15.19 Hz	C  17.90 Hz	A  18.75 Hz	C  18.99 Hz
	thick 	S  6.27 Hz	A  7.90 Hz	S  10.90 Hz	C  12.61 Hz	A  14.22 Hz	C  16.26 Hz	S  17.98 Hz	C  19.34 Hz	C  19.46 Hz	
Double	thin 	A  4.54 Hz	S  4.63 Hz	S  6.86 Hz	C  7.93 Hz	A  8.92 Hz	C  9.99 Hz	S  11.39 Hz	C  12.60 Hz	C  13.81 Hz	A  14.09 Hz
	medium-thick 	S  5.53 Hz	A  6.09 Hz	S  8.59 Hz	C  10.15 Hz	A  11.17 Hz	C  13.00 Hz	S  14.18 Hz	C  16.60 Hz	A  17.48 Hz	C  17.66 Hz
	thick 	S  6.41 Hz	A  7.60 Hz	S  10.46 Hz	C  12.00 Hz	A  13.47 Hz	C  15.30 Hz	S  16.85 Hz	C  18.84 Hz		

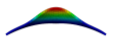
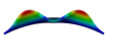
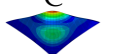
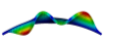
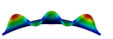
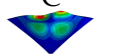
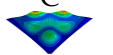
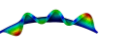
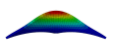
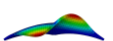
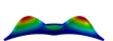
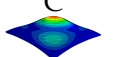
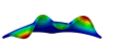
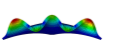
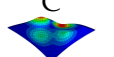
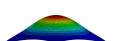
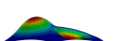
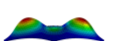
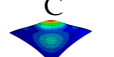
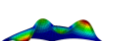
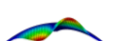
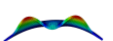
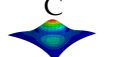
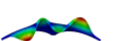
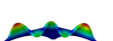
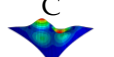
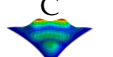
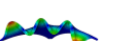
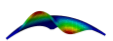
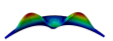
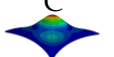
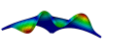
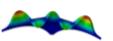
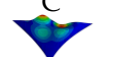
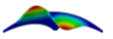
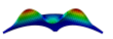
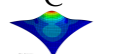
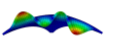
A – antisymmetric mode
C – complex mode
S – symmetric mode

under consideration in this discussion. Mode eight and mode nine are complex modes (which are qualitatively similar) for the thin arch dam configurations for both the single curvature and double curvature dam configurations, with the tenth mode appearing as the third antisymmetric mode in the vibration mode sequence. Whilst for the medium-thick classified arch dam configurations mode eight is a complex mode followed by the third antisymmetric mode as the ninth mode of vibrations. The tenth mode for the medium-thick single curvature and double curvature dam configurations is a complex mode, although they are not qualitatively the same. The tenth mode shape for medium-thick double curvature configurations is similar to the ninth mode shape of the thin arched dam configuration in the same curvature class, whereas the tenth mode shape for the medium-thick single curvature configurations is similar to the eighth mode shape of the thick arched dam configuration in the same curvature class. A maximum of nine mode shapes were obtained for the thick arched single curvature dam configuration, in which the last two mode shapes are complex modes. Whilst only eight mode shapes were obtained for the thick arched double curvature dam configuration in which the last mode shape is a complex mode.

5.3.2 Narrow-V shaped valley configurations

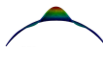
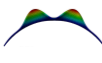
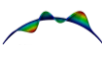
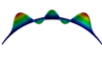
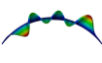
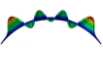
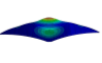
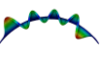
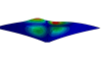
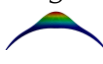
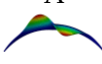
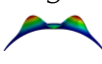
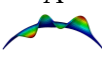
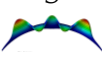
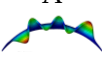
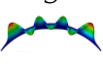
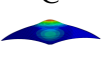
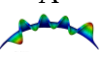
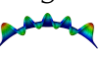
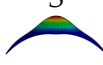
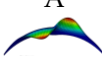
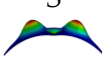
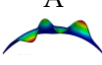
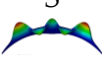
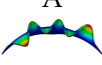
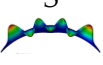
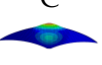
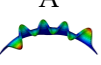
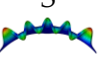
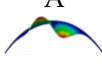
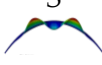
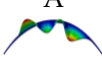
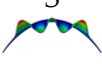
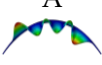
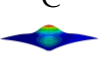
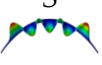
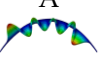
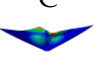
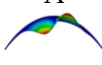
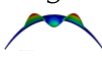
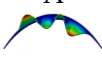
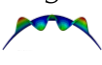
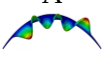
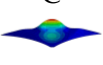
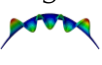
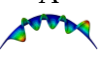
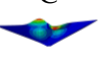
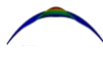
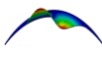
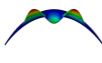
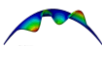
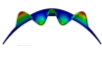
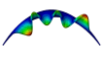
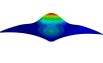
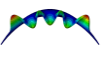
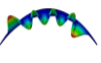
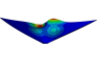
Table 5.2 shows the theoretical modes obtained for all the narrow-V valley dam configurations. The sequence of the vibration modes is the same for all configurations here discussed. The first mode shape is the first symmetric mode followed by the first antisymmetric mode shape. Similarly to the narrow-U shaped dam configuration(s) modes, the third vibration mode is the second symmetric mode, the fourth vibration mode the first complex mode (cantilever dominant mode shape) and the fifth vibration mode is the second antisymmetric mode. Only five vibration modes were obtained for both single curvature and double curvature thick arch dam classified configurations, whilst for the medium-thick classified dam configurations only seven vibration mode shapes were obtained in which the sixth mode shape is the third symmetric mode shape and the seventh vibration mode the second complex mode (dominant in both the cantilever & arch). Two more additional mode shapes were obtained for the thin double curvature arch dam configuration namely, a third complex mode as the eighth vibration mode and the third antisymmetric mode as the ninth vibration mode. In addition to these two modes, a tenth vibration mode was obtained as the fourth complex mode shape in the case of the thin single curvature arch dam configuration.

Table 5.2: Narrow-V shaped dam configuration(s) theoretical vibration modal parameters

Dam Curvature	Arch class	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Single	thin	S 	S 	A 	S 	C 	A 	S 	C 	C 	A 
		4.58 Hz	4.95 Hz	6.85 Hz	8.70 Hz	9.02 Hz	11.72 Hz	12.02 Hz	14.58 Hz	14.83 Hz	17.01 Hz
	medium-thick	S 	S 	A 	S 	C 	A 	S 	C 		
		6.18 Hz	7.84 Hz	10.48 Hz	13.49 Hz	13.58 Hz	17.07 Hz	18.44 Hz			
	thick	S 	S 	A 	S 	C 	A 				
		7.26 Hz	9.59 Hz	12.66 Hz	15.97 Hz	16.21 Hz					
Double	thin	S 	S 	A 	S 	C 	A 	S 	C 	C 	A 
		5.28 Hz	6.25 Hz	8.60 Hz	10.82 Hz	11.29 Hz	14.30 Hz	15.29 Hz	17.60 Hz	17.65 Hz	
	medium-thick	S 	S 	A 	S 	C 	A 	S 	C 		
		6.36 Hz	7.96 Hz	10.67 Hz	13.57 Hz	13.76 Hz	17.14 Hz	18.70 Hz			
	thick	S 	S 	A 	S 	C 	A 				
		7.61 Hz	9.90 Hz	13.01 Hz	16.17 Hz	16.54 Hz					

A - antisymmetric mode
C - complex mode
S - symmetric mode

Table 5.3: Wide-V shaped dam configuration(s) theoretical vibration modal parameters

Dam Curvature	Arch class	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Single	thin 	 2.76 Hz	 2.83 Hz	 3.45 Hz	 3.79 Hz	 4.38 Hz	 5.03 Hz	 5.77 Hz	 6.05 Hz	 6.60 Hz	 7.31 Hz
	medium-thick 	 4.14 Hz	 4.70 Hz	 5.53 Hz	 6.39 Hz	 7.33 Hz	 8.35 Hz	 9.42 Hz	 10.01 Hz	 10.56 Hz	 11.76 Hz
	thick 	 4.99 Hz	 5.86 Hz	 6.90 Hz	 7.97 Hz	 9.12 Hz	 10.31 Hz	 11.57 Hz	 12.07 Hz	 12.88 Hz	 14.24 Hz
Double	thin 	 3.07 Hz	 3.30 Hz	 4.23 Hz	 4.93 Hz	 5.79 Hz	 6.69 Hz	 6.90 Hz	 7.65 Hz	 8.66 Hz	 8.80 Hz
	medium-thick 	 4.12 Hz	 4.62 Hz	 5.58 Hz	 6.51 Hz	 7.52 Hz	 8.59 Hz	 9.40 Hz	 9.71 Hz	 10.88 Hz	 11.60 Hz
	thick 	 5.07 Hz	 5.90 Hz	 7.05 Hz	 8.21 Hz	 9.42 Hz	 10.67 Hz	 11.43 Hz	 11.97 Hz	 13.30 Hz	 13.95 Hz

A - antisymmetric mode
C - complex mode
S - symmetric mode

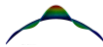
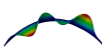
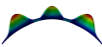
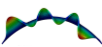
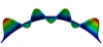
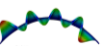
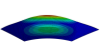
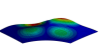
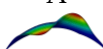
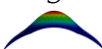
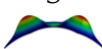
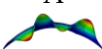
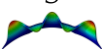
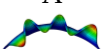
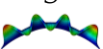
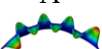
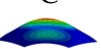
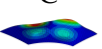
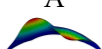
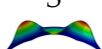
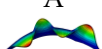
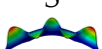
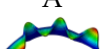
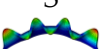
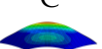
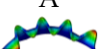
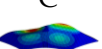
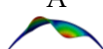
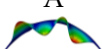
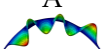
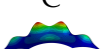
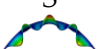
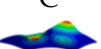
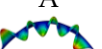
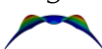
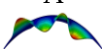
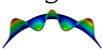
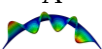
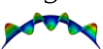
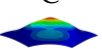
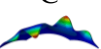
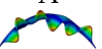
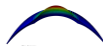
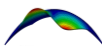
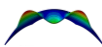
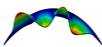
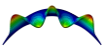
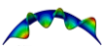
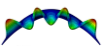
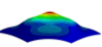
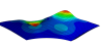
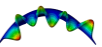
5.3.3 Wide-V shaped valley configurations

Table 5.3 shows the theoretical modes obtained for all the wide-V valley dam configurations, from which it can be observed that the first six vibration mode sequence is the same for all dam configurations in this valley classification. Similarly, to the narrow-V configurations, the first vibration mode is the first symmetric mode followed by the first antisymmetric mode. There after the vibration modes interchange between the symmetric and antisymmetric modes till the third antisymmetric mode shape appearing as the sixth vibration mode of these dam configurations. In the case of the single curvature dam configurations, the seventh vibration mode is the fourth symmetric mode and the eight-vibration mode the first dominant cantilever complex mode. Whereas for the double curvature dam configurations the seventh vibration mode is the first dominant cantilever complex mode followed by the fourth symmetric mode. The ninth vibration mode is the fourth antisymmetric mode shape for all dam configurations. The tenth vibration mode for all double curvature dam configurations and only the thin single curvature arch dam configuration, is the second complex (dominant in both the cantilever & arch) mode shape. For the medium-thick and thick single curvature arch dam configurations, the tenth vibration mode is the fifth symmetric mode shape.

5.3.4 Wide-U shaped valley configurations

Table 5.4 shows the theoretical modes obtained for all the wide-U valley dam configurations. Similarly to the narrow-U shaped results, the medium-thick and thick arch configurations for both the single curvature and double curvature arch types have similar mode shapes up to at least the seventh vibration mode. The medium-thick arch dam configuration vibration mode sequence begins with the first antisymmetric mode followed by the first and second symmetric modes. The fourth, fifth, sixth and seventh vibration modes are the second antisymmetric, third symmetric, third antisymmetric and fourth symmetric modes respectively. The eighth vibration mode for the single curvature medium-thick dam configuration is the fourth antisymmetric mode followed by two complex modes. Whilst for the double curvature medium-thick dam configuration, the eighth and ninth vibration modes are complex modes and the tenth vibration mode, the fourth antisymmetric mode. On the other hand, for the thick arch dam configurations, the modes interchange between the symmetric and antisymmetric modes until the seventh vibration mode, which is the fourth symmetric mode. The eighth vibration mode is the first dominant cantilever complex mode for both the single and double curvature configurations. The ninth and tenth vibration modes for the double curvature thick arch configuration are the second complex (dominant in both the cantilever & arch) mode

Table 5.4: Wide-U shaped dam configuration(s) theoretical vibration modal parameters

Dam Curvature	Arch class	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Single	thin 	 2.36 Hz	 2.37 Hz	 2.89 Hz	 2.95 Hz	 3.39 Hz	 3.83 Hz	 4.44 Hz	 5.12 Hz	 5.32 Hz	 5.50 Hz
	medium-thick 	 3.76 Hz	 3.77 Hz	 4.25 Hz	 4.88 Hz	 5.70 Hz	 6.59 Hz	 7.57 Hz	 8.62 Hz	 8.78 Hz	 9.24 Hz
	thick 	 4.48 Hz	 4.61 Hz	 5.26 Hz	 6.13 Hz	 7.14 Hz	 8.21 Hz	 9.36 Hz	 10.57 Hz	 10.57 Hz	 11.11 Hz
Double	thin 	 2.36 Hz	 2.46 Hz	 3.35 Hz	 3.46 Hz	 4.26 Hz	 5.03 Hz	 5.76 Hz	 5.90 Hz	 6.28 Hz	 6.76 Hz
	medium-thick 	 3.59 Hz	 3.70 Hz	 4.28 Hz	 4.86 Hz	 5.74 Hz	 6.66 Hz	 7.64 Hz	 8.08 Hz	 8.63 Hz	 8.71 Hz
	thick 	 4.55 Hz	 4.56 Hz	 5.30 Hz	 6.19 Hz	 7.26 Hz	 8.37 Hz	 9.52 Hz	 9.85 Hz	 10.49 Hz	 10.75 Hz

A - antisymmetric mode
C - complex mode
S - symmetric mode

shape and the fourth antisymmetric mode whilst for the single curvature thick arch configuration these mode shapes are swapped for the ninth and tenth vibration modes. In the case of the double curvature thin arch configuration, the vibration mode sequence is similar to that of the medium-thick arch configurations in the same valley type under consideration, up to the sixth vibration mode shape. The seventh vibration mode is the first complex mode, which is qualitatively unlike all the other first complex modes. The fourth symmetric and antisymmetric modes are the eighth and tenth vibration modes respectively. Whilst the ninth vibration mode is the second complex (dominant in both the cantilever & arch) mode shape. The sequence of the single curvature thin arch configuration is mostly the direct opposite of the double curvature thin arch vibration mode shape sequence. The single curvature thin arch vibration modes begin with the first symmetric mode shape followed by the first and second antisymmetric modes. The second symmetric mode appears as the fourth vibration mode, which for all the other configurations considered in this study, it always appeared as the third vibration mode. In addition, this is the only sequence in which the second antisymmetric mode appears before the second symmetric mode. The fifth to eighth vibration modes interchange between the symmetric and antisymmetric mode shapes, whilst the ninth and tenth vibration modes are the first complex mode (cantilever dominant mode shape) and second complex (dominant cantilever & arch mode) mode shapes respectively.

5.3.5 Theoretical mode shape sequence validation

The first step was to validate the observed theoretical mode shapes (Table 5.1 to Table 5.4). The linear dynamic behaviour of a structure is normally assumed to be governed by only a few of the lower modes. Additionally, field test can normally pick the first six natural frequencies and mode shapes. Consequently, for dams the lower modes are of greatest importance (Makha, 2012). Table 5.5 provides a summary of only the first five arch dominant modes (symmetric and antisymmetric modes) for easy comparison with the observed field vibration measurements (Table 4.4). Although the natural frequency values from the field observations (Table 4.4) cannot be compared to those in Table 5.5 owing to differences in characteristics of the individual dam-foundation-reservoir systems, it is assumed that the mode shape sequence in a general sense can be compared between the two data sets. For the dam configurations which have similar valley and thickness classifications as the theoretical configurations considered in this study, as well as all other dam configurations which do not directly fall under the valley classification (defined by the $l_c: h$ ratio), but have the same thickness classification, good correlation of the vibration mode sequences is observed.

Table 5.5: Summary of theoretical modal parameter characteristics of various concrete arch dam configurations

Valley Shape ($l_c: h$)	Arch class ($b_c: h$)	Mode	Single Curvature		Double Curvature	
			Type	Frequency (Hz)	Type	Frequency (Hz)
Narrow-U (2.0)	thin (0.12)	1	A	4.02	A	4.54
		2	S	4.20	S	4.63
		3	S	5.91	S	6.86
		4	A	7.69	A	8.92
		5	S	10.18	S	11.39
	medium-thick (0.27)	1	S	5.46	S	5.53
		2	A	6.43	A	6.09
		3	S	8.98	S	8.59
		4	A	11.83	A	11.17
		5	S	15.19	S	14.18
	thick (0.36)	1	S	6.27	S	6.41
		2	A	7.90	A	7.60
		3	S	10.90	S	10.46
		4	A	14.22	A	13.47
		5	S	17.98	S	16.85
Narrow-V (2.0)	thin (0.12)	1	S	4.58	S	5.28
		2	A	4.95	A	6.25
		3	S	6.85	S	8.60
		4	A	9.02	A	11.29
		5	S	11.72	S	14.30
	medium-thick (0.27)	1	S	6.18	S	6.36
		2	A	7.84	A	7.96
		3	S	10.48	S	10.67
		4	A	13.58	A	13.76
		5	S	17.07	S	17.14
	thick (0.36)	1	S	7.26	S	7.61
		2	A	9.59	A	9.90
		3	S	12.66	S	13.01
		4	A	16.21	A	16.54
Wide-V (5.1)	thin (0.12)	1	S	2.76	S	3.07
		2	A	2.83	A	3.30
		3	S	3.45	S	4.23
		4	A	3.79	A	4.93
		5	S	4.38	S	5.79
	medium-thick (0.27)	1	S	4.14	S	4.12
		2	A	4.70	A	4.62
		3	S	5.53	S	5.58
		4	A	6.39	A	6.51
		5	S	7.33	S	7.52
	thick (0.36)	1	S	4.99	S	5.07
		2	A	5.86	A	5.90
		3	S	6.90	S	7.05
		4	A	7.97	A	8.21
		5	S	9.12	S	9.42
Wide-U (5.1)	thin (0.12)	1	S	2.36	A	2.36
		2	A	2.37	S	2.46
		3	A	2.89	S	3.35
		4	S	2.95	A	3.46
		5	S	3.39	S	4.26
	medium-thick (0.27)	1	A	3.76	A	3.59
		2	S	3.77	S	3.70
		3	S	4.25	S	4.28
		4	A	4.88	A	4.86
		5	S	5.70	S	5.74
	thick (0.36)	1	S	4.48	S	4.55
		2	A	4.61	A	4.56
		3	S	5.26	S	5.30
		4	A	6.13	A	6.19
		5	S	7.14	S	7.26

A – antisymmetric
S – symmetric

The only exceptions include the Morrow Point dam in the USA. It is classified as a thin double curvature arch dam in a valley that is classified as a narrow valley. Its vibration mode sequence differs in the fourth mode and fifth mode shape from the thin double curvature arch configuration for the narrow V case in Table 5.5. The other two exceptions are Quan Shuri dam in China and Saveh dam in Iran. Both dams are classified as thin double curvature arch dam structures located in valleys that are classified in-between narrow and wide. They have the same vibration mode sequence that differs in the fourth and fifth mode shapes from both narrow and wide U thin double curvature arch configuration mode sequence in Table 5.5. This observation could highlight the uniqueness of each structural layout, founding conditions and their influence on the structural behaviour of dams. It is clear from these observations that the combined interaction of the dam wall thickness, crest length and valley shape influence the dynamic behaviour of concrete arch dams.

5.3.6 Influence of the curvature type on the dynamic characteristics of arch dams

Curvature allows for the optimisation of the arch shape and thickness. As can be seen in Table 5.5 above, the curvature has no influence on the sequence of the first five arch dominant modes in all cases except for the wide U thin arch configurations in which the sequence of these modes is different for the single curvature and double curvature dam configurations. To understand the influence of the curvature type on the natural frequency values, Table 5.6 shows the relative change (as a percentage) in natural frequencies of the first five corresponding dominant arch modes between the single curvature and double curvature dam configurations. The average percentage relative change and standard deviation calculated from these five natural frequency values for each configuration, do not vary greatly with those calculated for all the natural frequency values discussed in sections 5.3.1 to 5.3.4 (refer to the Appendix for further details). Thus, the average percentage relative difference and standard deviation values of the first five modes analysed in Table 5.6 are considered representative of the percentage relative change and standard deviation of the percentage relative change of the natural frequency values due to curvature for all the theoretical natural frequency values reported in this study. Also, since the vibration mode sequences is not the same for all configurations, the notation 'arch mode' is used to denote symmetric or antisymmetric modes (only) depending on the valley classification and arch classification vibration mode sequence. A positive percentage relative change means that the double curvature configuration natural frequency is higher than the corresponding single curvature configuration natural frequency. On the other hand, a negative percentage relative change means that the double curvature

configuration natural frequency is less than the corresponding single curvature configuration natural frequency.

Table 5.6: Relative change in natural frequencies of single curvature vs double curvature configurations (as a percentage)

Valley Shape	Arch class	Arch Mode 1 (%)	Arch Mode 2 (%)	Arch Mode 3 (%)	Arch Mode 4 (%)	Arch Mode 5 (%)	Average (%)	Standard deviation (%)
 Narrow-U	thin	13	10	16	16	12	13	2
	medium-thick	1	-5	-4	-6	-7	-4	3
	thick	2	-4	-4	-5	-6	-3	3
 Narrow-V	thin	15	26	26	25	22	23	4
	medium-thick	3	2	2	1	0	1	1
	thick	5	3	3	2	-	3	1
 Wide-V	thin	11	17	23	30	32	23	8
	medium-thick	0	-2	1	2	3	1	2
	thick	2	1	2	3	3	2	1
 Wide-U	thin	4	0	20	14	26	13	10
	medium-thick	-5	-2	1	0	1	-1	2
	Thick	2	-1	1	1	2	1	1

The natural frequency values for thin arch double curvature configurations are higher as compared to their corresponding single curvature natural frequency values for all valley types. For, the U-shaped (both narrow and wide) valley types, the average percentage increase in frequency due to the curvature class for the thin configuration modes is about the same at 13% (with the wide valley type under the U classified valley showing greater variation in the individual percentage relative difference values per mode). Whilst for the V-shaped valley types, the average percentage relative increase in frequency due to curvature for the thin configurations is about 23% on average (in which the wide valley type under the V classified valley type shows greater variation). The influence of curvature on the medium-thick and thick arch dam configurations varies for each valley type. Generally (on average), the U-shaped valley medium-thick and thick arch configuration natural frequency values for the double curvature configurations are slightly less than those of the corresponding single curvature configuration natural frequency values except for the thick wide-U arch dam configuration. Whilst for the V-shaped valley medium-thick and thick arch configurations, minimal increase in the natural frequency values due to curvature is observed when compared to the observed change in natural frequency values for the corresponding thin arch

configurations. The thin arch configurations in both the wide-U and wide-V valley types (wide valley class) show the highest variability in the percentage relative change in frequency per mode.

5.3.7 Influence of the valley type on the dynamic characteristics of arch dams

From the preceding discussion (section 5.3.1 to 5.3.4), the vibration mode sequence starts with the first symmetric mode followed by the first antisymmetric for all the V-shaped valley configurations. Whilst for the U-shaped valley configurations it varies depending on the arch thickness class. In general, for the U-shaped dam configurations, the vibration mode sequence begins with the first antisymmetric mode followed by the first symmetric mode for the thin arch configurations except for the wide-U single curvature thin arch configuration. Then as the $b_t: h$ increases, the first vibration mode tends to shift from the first antisymmetric mode shape to the first symmetric mode shape and the converse happens for the second vibration mode. Also, the first complex mode occurs within the first five modes for the narrow valley classified arch configurations whilst for the wide classified arch dam configurations, the first complex modes appear after at least the sixth vibration mode. Additionally, narrow-V arch configurations have higher natural frequencies as compared to narrow-U arch configurations, which in turn have higher natural frequency values than wide-V configurations, with the wide-U configurations having the lowest frequency values for the same mode shape, an artefact of the interplay between stiffness, mass, and valley shape.

Table 5.7: Relative change in natural frequencies as the crest length (l_c) increases (i.e., between narrow-V and wide-V arch configurations) (as a percentage)

Arch class	Arch curvature	Arch Mode 1 (%)	Arch Mode 2 (%)	Arch Mode 3 (%)	Arch Mode 4 (%)
thin	single	-40	-43	-50	-58
	double	-42	-47	-51	-56
medium-thick	single	-33	-40	-47	-53
	double	-35	-42	-48	-53
thick	single	-31	-39	-45	-51
	double	-33	-40	-46	-50

Keeping all other factors constant, as the crest length (l_c) increases, the natural frequency decreases. This is illustrated by a decrease in natural frequencies for the corresponding mode shapes between the narrow-V and the wide-V configurations. The percentage relative decrease

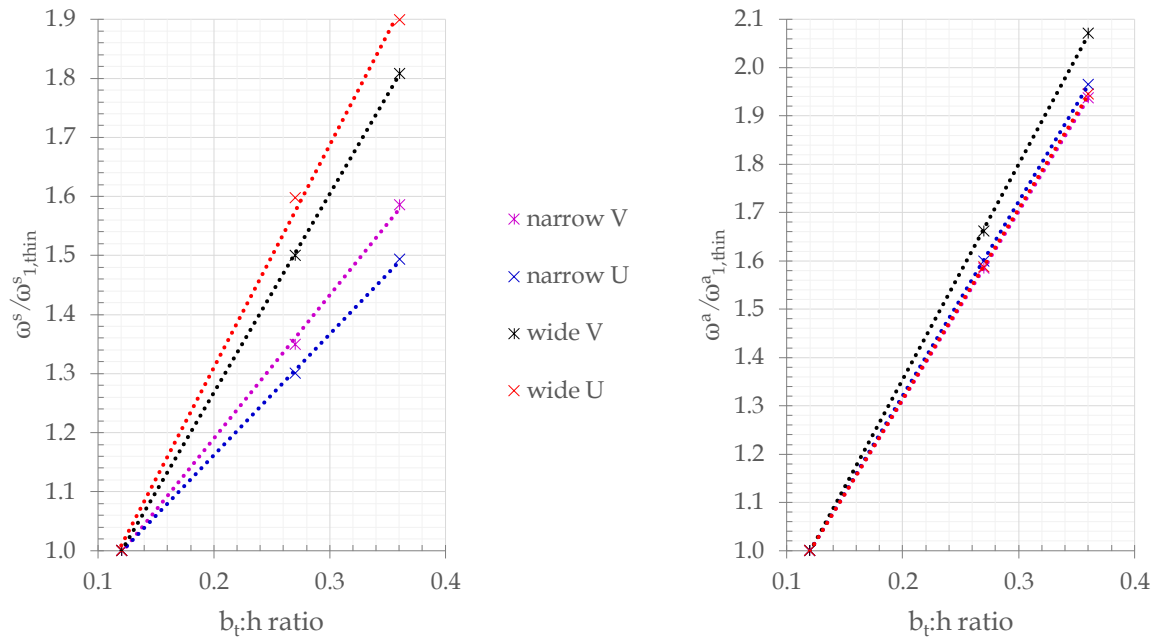
(change) in the natural frequencies between the first four dominant arch mode shapes of the narrow-V configurations and the wide-V configurations varies between -30% to -60% (Table 5.7). The relative decrease in natural frequencies increase from the lowest to highest mode for the first four modes illustrated in Table 5.7.

Table 5.8: Relative change in natural frequencies as the base length (l_b) decreases (i.e., between U- and V- shaped arch configurations) (as a percentage)

Valley Type	Arch class	Arch curvature	Arch Mode 1 (%)	Arch Mode 2 (%)	Arch Mode 3 (%)	Arch Mode 4 (%)
Narrow	thin	single	23	9	16	17
		double	38	14	25	27
	medium-thick	single	13	22	17	15
		double	15	31	24	23
	thick	single	16	21	16	14
		double	19	30	24	23
Wide	thin	single	17	19	43	31
		double	40	25	26	42
	medium-thick	single	25	10	11	31
		double	29	11	30	34
	thick	single	11	27	31	30
		double	11	29	33	33

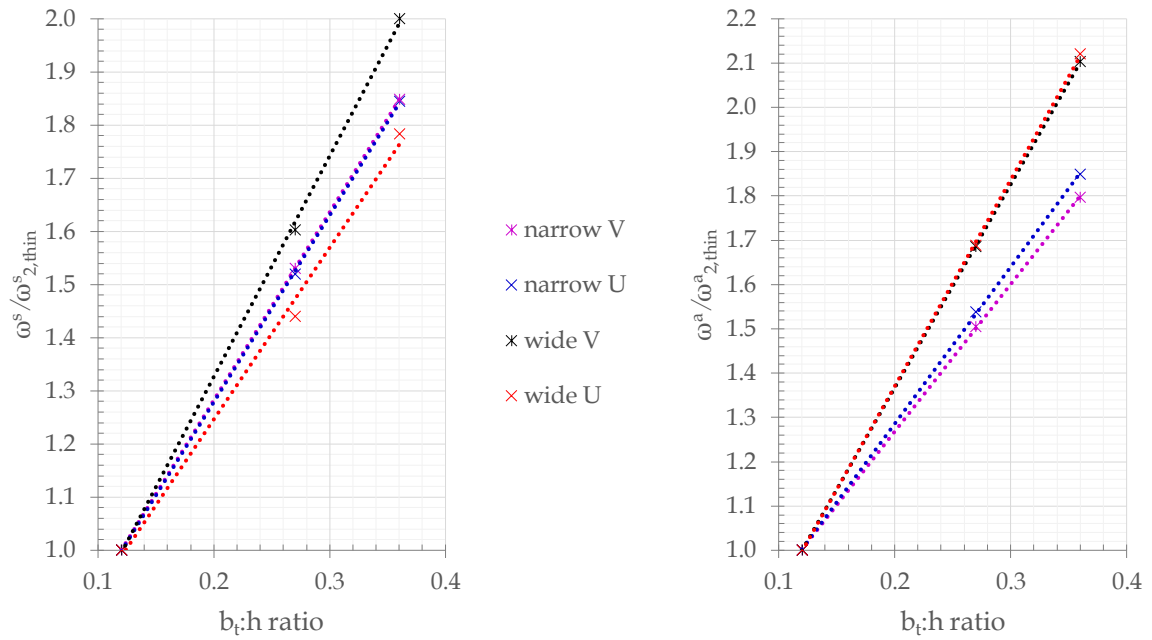
On the other hand, keeping all other factors constant, as the base length (l_b) decreases, the natural frequency increases evidenced by an increase in natural frequency values between the corresponding four dominant arch modes of the narrow-U shaped & the narrow-V shaped configurations and the wide-U shaped & the wide-V shaped configurations. For these configurations the percentage relative increase (change) in the frequency values varies between 10% to 40% (Table 5.8) depending on the mode shape and arch dam configuration. In general, the relative increase in natural frequencies is greater for double curvature arch configurations relative to the single curvature arch configurations. Also, as the crest length (l_c) and/or base length (l_b) decreases, the total number of extracted theoretical mode shapes decreases (refer to Table 5.1 and Table 5.2).

5.3.8 Influence of the arch thickness classification on the dynamic characteristics of arch dams



a) First symmetric mode

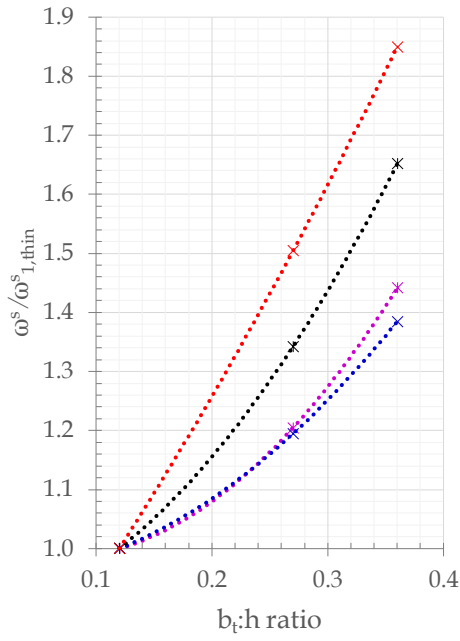
b) First antisymmetric mode



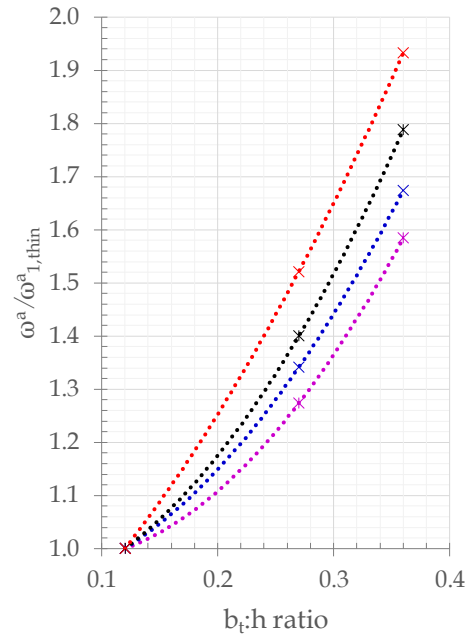
c) Second symmetric mode

d) Second antisymmetric mode

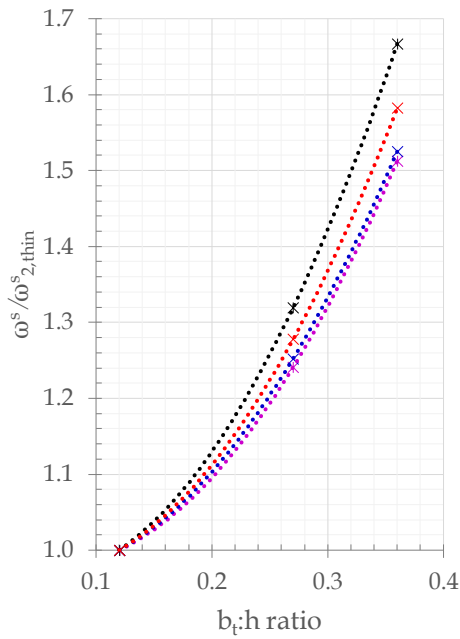
Figure 5.3: $\omega/\omega_{x,thin}$ versus thickness classification ($b_t:h$ ratio) graphs for single curvature configurations



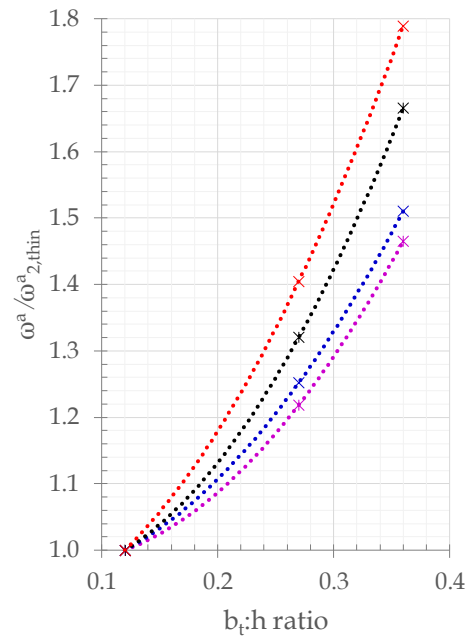
a) First symmetric mode



b) First antisymmetric mode



c) Second symmetric mode



d) Second antisymmetric mode

Figure 5.4: $\omega/\omega_{x,thin}$ versus thickness classification ($b_t:h$ ratio) graphs for double curvature configurations

In addition to the already noted observation that as $b_t:h$ increases the first vibration mode tends to shift towards a symmetric first vibration mode for the arch dam configuration(s)

noted above (section 5.3.7), as $b_t: h$ increases the total number of extracted theoretical modes decreases (refer to Table 5.1 and Table 5.2). Also, as $b_t: h$ increases the natural frequency for all observed vibration modes of each corresponding mode shape per valley and curvature classification increases (Figure 5.3 and Figure 5.4), which implies that with increasing $b_t: h$, the increase in stiffness has a greater effect than the increase in mass. Figure 5.3 and Figure 5.4 illustrates graphically the relation of natural frequency (presented as a ratio of the thin classified natural frequencies) to $b_t: h$ for the first four arch dominant vibration modes for single curvature and double curvature configurations respectively. There is clearly, a strong correlation between natural frequency and the thickness classification. In general, the rate of increase in natural frequency due to $b_t: h$ is constant for single arch configurations (Figure 5.3) whilst for double curvature arch configurations, the rate of change in natural frequency increases as the arch thickness classification increase (Figure 5.4). (Linear and parabolic relationships between the natural frequency and $b_t: h$ ratios were assumed for all single curvature and double curvature configurations respectively.) Additionally, the rate of change in natural frequencies with increasing $b_t: h$ varies depending on the mode shape and valley characteristics (both the shape (U or V and width (narrow or wide) of the valley).

5.4 Static Behaviour Characterisation of Various Arch Dam Configurations

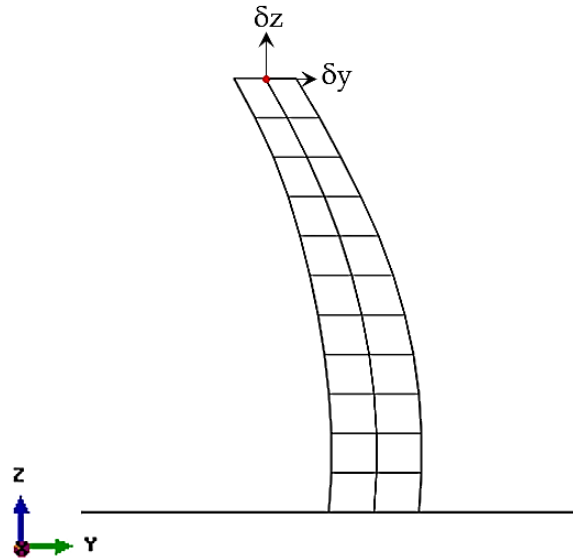


Figure 5.5: Illustration of crown cantilever node used to extract the displacement values shown in Table 5.9

Figure 5.5 shows an illustration of a model's crown cantilever cross-section showing the typical node location from which the model's global y- and z- direction displacements (which correspond to the radial displacement and vertical displacement (at the crown cantilever)

respectively) were extracted for the different arch dam configurations and are presented in Table 5.9. Table 5.9 shows the crown cantilever deflected shape due to uniformly applied temperature load(s), full water level hydrostatic pressure and the deflected shape along the crest length due to the full dam hydrostatic load.

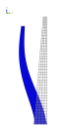
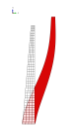
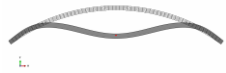
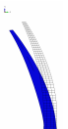
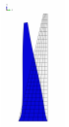
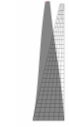
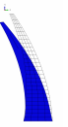
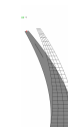
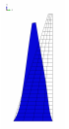
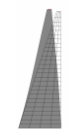
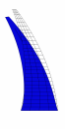
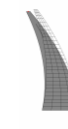
5.4.1 Observations of the static behaviour due to thermal loading

As expected in a linear elastic analysis, the extracted absolute displacement values are the same for the low and high thermal load in both directions for all configurations since the same low and high temperature change was applied. For these displacement values (thermal displacements), the radial displacements decrease as the thickness classification (i.e., $b_t: h$) increases for all valley type classifications, whilst the vertical displacements response varies based on the curvature class as the thickness classification increases for all valley types. In general, the vertical displacements remain relatively constant for single curvature dam configurations even as the thickness class in each valley type changes, whereas a slight decrease in the vertical displacements with increasing thickness class (in general) is observed for the double curvature dam configurations for each valley type. The only exceptions are the narrow-U shaped valley double curvature configurations where the vertical displacements are unaffected by a change in $b_t: h$ and the wide-U shaped thin double curvature arch dam configurations whose vertical displacements are lower compared to the other thickness classifications in the same valley type class. The thermal vertical displacements are generally higher for double curvature arch dam configurations as compared to single curvature arch dam configuration vertical displacements. On the other hand, the radial displacements start of higher for double curvature dam configurations but generally end up lower than the corresponding single curvature configurations as $b_t: h$ increases for the narrow-U and wide-V valley dam configurations. Whilst the radial displacements are lower for the double curvature arch configurations for the narrow-V and wide-U valley dam configurations.

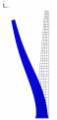
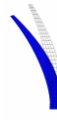
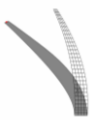
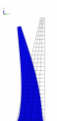
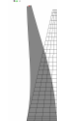
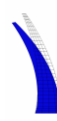
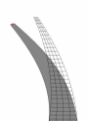
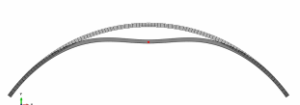
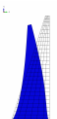
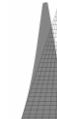
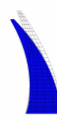
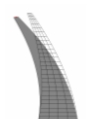
5.4.2 Observations of the static behaviour due to hydrostatic load

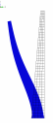
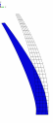
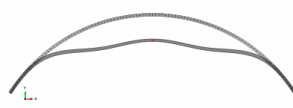
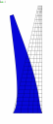
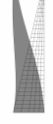
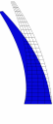
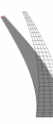
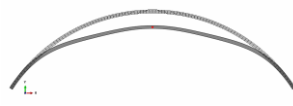
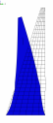
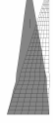
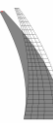
The radial displacements due to the hydrostatic load are generally higher for single curvature dam configurations as compared to double curvature configurations and tend to decrease as the dam thickness class increases for each valley type in all cases. It can also be observed from the crest level displacement profile that the deflection extends further towards the foundation more in single curvature dam configurations as compared to double curvature configurations and for U-shaped valley configurations as compared V-shaped valley configurations.

Table 5.9: Summary of theoretical static behaviour characteristics of the various concrete arch dam configurations

Valley Shape	Arch class	Single Curvature				Double Curvature			
		Low temp (mm)	High temp (mm)	Hydrostatic (mm)	Crest level displacement profile due to hydrostatic load	Low temp (mm)	High temp (mm)	Hydrostatic (mm)	Crest level displacement profile due to hydrostatic load
Narrow-U 	thin								
	δy	-17	17	-39		-20	20	-32	
	δz	-7	7	5		-13	13	-5	
	medium-thick								
	δy	-15	15	-21		-15	15	-19	
	δz	-7	7	4		-13	13	-4	
	thick								
	δy	-13	13	-14		-12	12	-12	
	δz	-7	7	3		-13	13	-3	

Valley Shape	Arch class	Single Curvature				Double Curvature			
		Low temp (mm)	High temp (mm)	Hydrostatic (mm)	Crest level displacement profile due to hydrostatic load	Low temp (mm)	High temp (mm)	Hydrostatic (mm)	Crest level displacement profile due to hydrostatic load
Narrow-V	thin								
	δy	-17	17	-35		-15	15	-23	
	δz	-6	6	4		-12	12	-7	
	medium-thick								
	δy	-12	12	-15		-10	10	-13	
	δz	-6	6	2		-11	11	-4	
	thick								
	δy	-9	9	-10		-7	7	-8	
	δz	-6	6	2		-10	10	-2	

Valley Shape	Arch class	Single Curvature				Double Curvature			
		Low temp (mm)	High temp (mm)	Hydrostatic (mm)	Crest level displacement profile due to hydrostatic load	Low temp (mm)	High temp (mm)	Hydrostatic (mm)	Crest level displacement profile due to hydrostatic load
	thin								
	δy	-26	26	-95		-29	29	-77	
	δz	-7	7	8		-17	17	-20	
	medium-thick								
	δy	-18	18	-37		-17	17	-35	
	δz	-7	7	5		-15	15	-10	
	thick								
	δy	-14	14	-22		-11	11	-20	
	δz	-7	7	4		-13	13	-6	

Valley Shape	Arch class	Single Curvature				Double Curvature			
		Low temp (mm)	High temp (mm)	Hydrostatic (mm)	Crest level displacement profile due to hydrostatic load	Low temp (mm)	High temp (mm)	Hydrostatic (mm)	Crest level displacement profile due to hydrostatic load
 Wide-U	thin								
	δy	-26	26	-64		-24	24	-58	
	δz	-8	8	7		-14	14	-5	
 Medium-thick	medium-thick								
	δy	-22	22	-39		-19	19	-34	
	δz	-7	7	6		-16	16	-8	
 Thick	thick								
	δy	-18	18	-25		-15	15	-22	
	δz	-7	7	5		-15	15	-5	

The hydrostatic load results are positive vertical displacements (i.e. upwards) for single curvature dam configurations and negative vertical displacements (i.e., downwards) for double curvature dam configurations in all cases. As the dam thickness class increases, the vertical displacements tend to decrease for both single and double curvature dam configurations, apart from the wide-U shaped double curvature dam configuration displacements.

5.4.3 Influence of the valley type on the static behaviour characteristics of arch dams

In all cases except for the hydrostatic vertical displacements, the narrow-V shaped configurations have the lowest displacements followed by narrow-U dam configurations. The wide-V and wide-U configurations exchange about midway of the $b_t:h$ range used in this study except for the single curvature configuration thermal displacements and double curvature hydrostatic vertical displacements.

Keeping all other factors constant, as the crest length (l_c) increases, both radial and vertical thermal, as well as the hydrostatic displacements increase (illustrated by an increase in displacements between the narrow V and the wide V configurations in Table 5.9). The relative increase (expressed as a percentage) in thermal displacements varies between 20% to 90% whilst the percentage relative increase related to hydrostatic displacements is about 4 times (on average) the percentage relative increase related to thermal displacements (Table 5.10). The vertical displacement relative changes are lower than for the radial displacements in all cases presented in Table 5.10.

Table 5.10: Relative change in displacements as the crest length (l_c) increases (i.e., between narrow-V and wide-V arch configurations) (as a percentage)

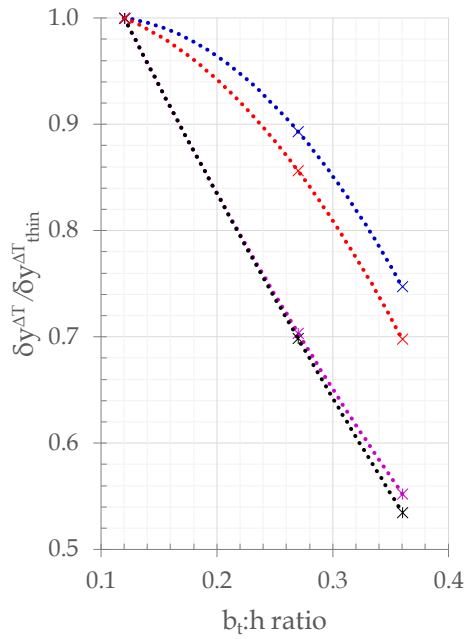
Arch class	Arch curvature	Thermal (radial) (%)	Thermal (vertical) (%)	Hydrostatic (radial) (%)	Hydrostatic (vertical) (%)
thin	single	53	23	173	126
	double	87	43	243	191
medium-thick	single	52	19	143	121
	double	65	36	173	147
thick	single	48	19	129	113
	double	68	36	159	136

Table 5.11 gives the relative change (expressed as a percentage) in displacements between the U valley shaped and V valley shaped configurations under each narrow and wide valley classification. A negative percentage relative difference means that the V-shaped configuration has a lower displacement than its corresponding U-shaped configuration and vice versa for the positive percentage difference. Keeping all other factors constant, as the base length (l_b) decreases, the thermal and hydrostatic displacements also decrease for the narrow valley configurations except for the thin double curvature vertical hydrostatic displacement that show an increase. Whilst, for the wide valley configurations, as the base length (l_b) decreases, the hydrostatic displacements increase for both single and double curvature thin arch configurations. Then as $b_t: h$ increases the percentage relative difference due to hydrostatic load as the base length (l_b) decreases tends to become negative (i.e., the displacements decrease). Thermal displacements follow the same trend as the hydrostatic displacements except for the single curvature thin configuration vertical displacements.

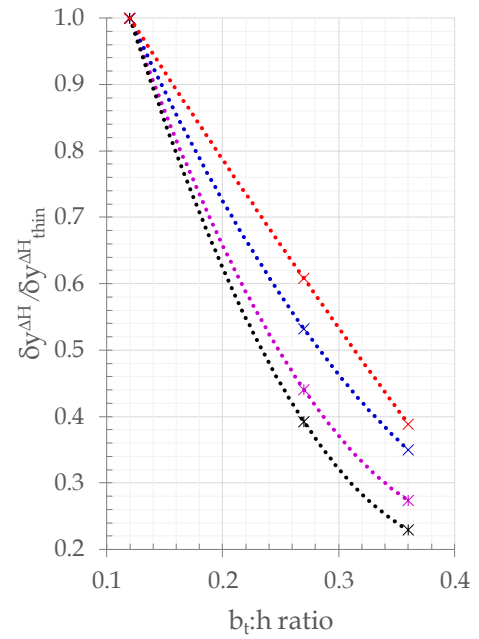
Table 5.11: Relative change of displacements with decreasing base length (l_b) (as a percentage)

			Thermal radial (%)	Thermal vertical (%)	Hydrostatic radial (%)	Hydrostatic vertical (%)
Narrow	thin	Single	-1	-20	-11	-27
		Double	-23	-5	-30	27
	medium-thick	Single	-22	-17	-26	-32
		Double	-32	-17	-33	-5
	thick	Single	-27	-16	-30	-34
		Double	-42	-22	-38	-17
Wide	thin	Single	1	-11	20	16
		Double	22	20	32	275
	medium-thick	Single	-18	-6	-5	-9
		Double	-9	-4	3	37
	thick	Single	-23	-4	-13	-15
		Double	-23	-11	-10	10

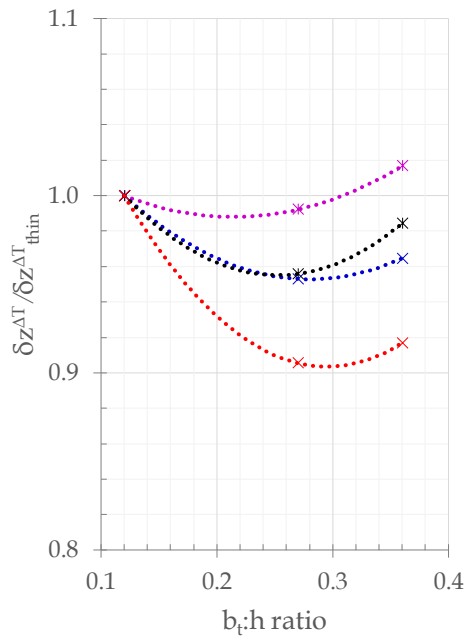
5.4.4 Influence of the arch thickness classification on the static behaviour characteristics of arch dams



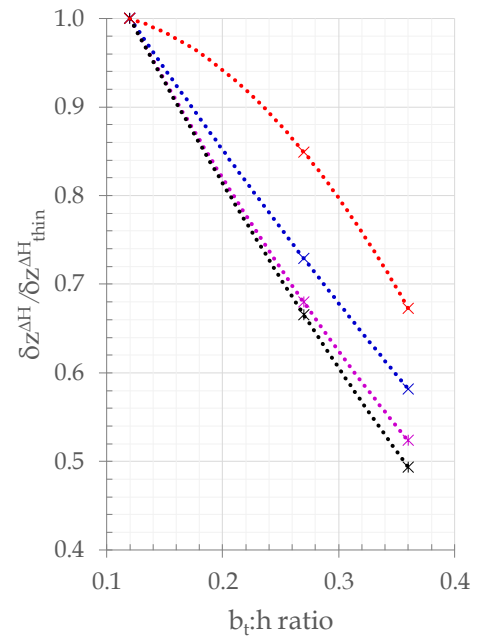
a) Thermal radial displacement



b) Hydrostatic radial displacement

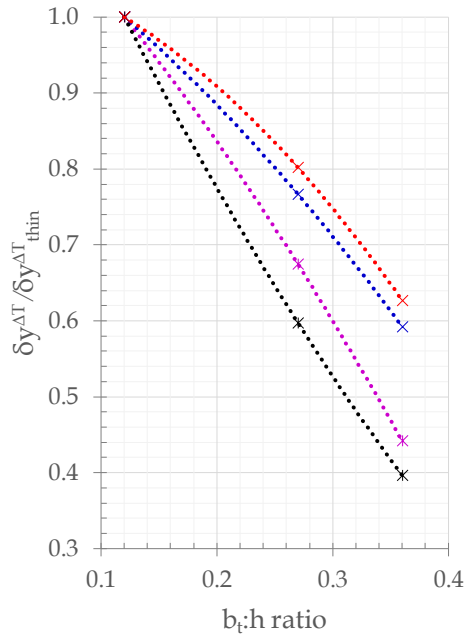


c) Thermal vertical displacement

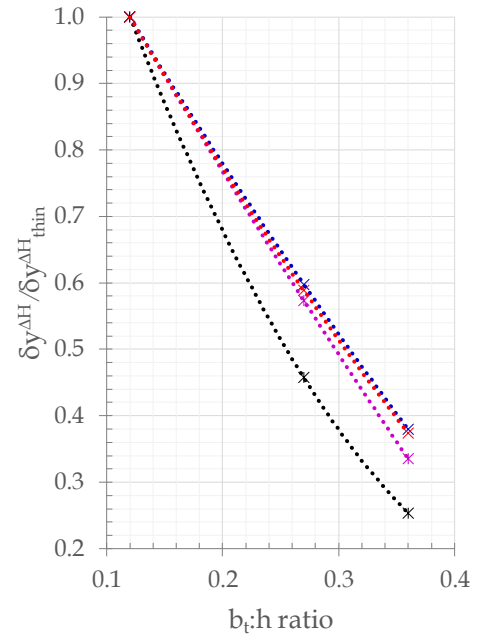


d) Hydrostatic vertical displacement

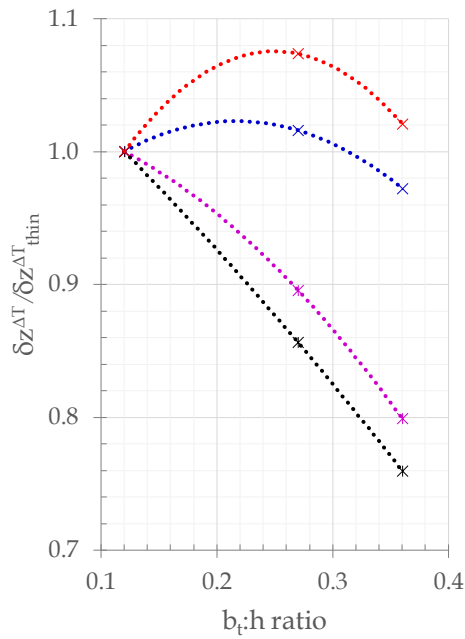
Figure 5.6: δ/δ_{thin} versus thickness classification ($b_t:h$ ratio) graphs for single curvature configurations



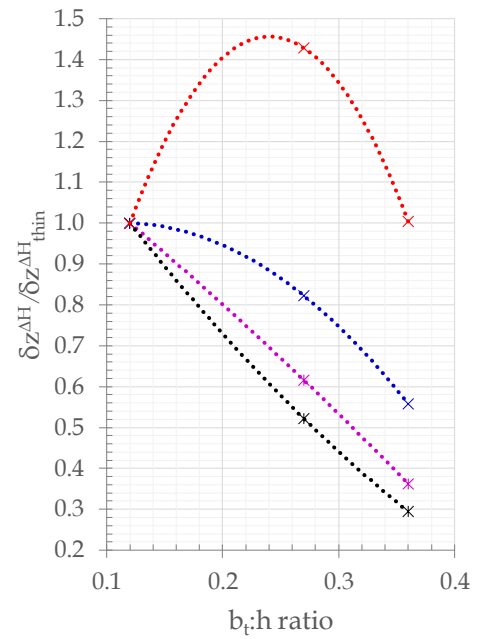
a) Thermal radial displacement



b) Hydrostatic radial displacement



c) Thermal vertical displacement



d) Hydrostatic vertical displacement

Figure 5.7: δ/δ_{thin} versus thickness classification ($b_t:h$ ratio) graphs for double curvature configurations

Figure 5.6 and Figure 5.7 illustrate graphically the relationship of displacements as a ratio of the thin classified arch configuration displacement values (due the thermal and hydrostatic loads) to $b_t: h$ for single curvature and double curvature configurations respectively. As $b_t: h$ increases the displacements generally decrease with exception of some vertical displacements. The rate of change in displacements with increasing $b_t: h$ varies depending on the displacement type and valley characteristics (both the shape (U or V and width (narrow or wide) of the valley) for all the investigated valley types.

5.5 Chapter Summary

This chapter presents the characterisation of the structural behaviour (both dynamic and static behaviour) of various configurations of concrete arch dams with the aim of understanding the influence of geometric parameters on the behaviour of these structures. The geometric parameters of concrete arch dams are generally a function of the site type (narrow-, wide-, U- & V- shaped valley types), type of arch (single or double curvature) and classification of arch dam (thin, medium-thick & thick).

The site type and arch classification influence both the natural frequency and mode shape characteristics. In general, the natural frequency increases from a wide valley type to a narrow valley type, from a U-shaped valley to a V-shaped valley and as the thickness class increases. The mode shape sequence for all V-shaped arch configurations starts with symmetric mode followed by an antisymmetric mode, whilst it varies for U-shaped valleys. Moreover, as the thickness class increases, the first mode shape tends to a symmetric mode for the thin arch configurations whose first mode is an antisymmetric mode. The arch type influences the variation in modal parameters across the site type and/or arch classification. For example, the rate of increase in natural frequency as the thickness class changes is constant for single curvature arch configurations whereas it varies for double curvature arch configurations.

In contrast, the arch type seems to have a greater influence on the static behaviour of the investigated arch dam configurations affecting both the direction and magnitudes of displacements. In general, the static displacements decrease as the arch class increases. However, there seems to be no clear pattern of the influence of geometric parameters on the static behaviour of the various configurations investigated in this study (as compared to the observed dynamic behaviour). Consequently, it may be difficult to estimate the static behaviour response of concrete arch dams based solely on the geometric characteristics of

concrete arch dams without measurements or performing numerical analysis to get a sense of a structure's response.

Although the behaviour of the various arch dam configurations investigated in this study can be generalised to varying extents for the dynamic and static behaviour, the structural behaviour of a particular concrete arch dam is unique to that structure due to its layout and configuration. Thus, modelling the dam geometry as closely as practically possible is of paramount importance.

Additionally, based on the observation from the characterisation of the behaviour of the various configuration, ten arch dam configurations consisting of both single and double curvature arch configurations are selected for further study namely: the thin narrow U, thin narrow V, thin wide U, thick narrow U, and thick wide U arch configurations. These models have been selected such that the effect of the valley type (i.e., narrow, wide, U-shaped, and V-shaped) and the thickness classification on the subsequent results can be evaluated.

5.6 References

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Chapter 6

SENSITIVITY OF THE STRUCTURAL BEHAVIOUR OF ARCH DAMS TO LOADING AND MATERIAL PROPERTY VARIATION

6.1 Introduction

It has been already established that this study focuses on the development of a finite element model updating procedure for concrete arch dams affected by ASR using dam surveillance data including dynamic monitoring, specifically, ambient vibration monitoring (AVM), data in the model updating procedure. The aim is to improve the structural health monitoring assessment of concrete arch dams using numerical models. The influence of the loading and the interaction of arch dams with their surrounding on their structural behaviour is investigated in this chapter. To meet the objectives of this research, particularly objectives (ii) & (iii) (section 1.3), the sensitivity of the structural behaviour of selected configurations of concrete arch dams to various parameters due to the interaction of arch dams with their surroundings, load effects and to variations in material properties is investigated and the findings presented in this chapter. The aim in this chapter is to arrive at a selection of the most effective parameters in improving the numerical models of these structures.

Generally, the model updating approach consists of three elements, namely: (i) selecting responses to be used as reference data, (ii) selecting of updating variables, and (iii) model tuning (Brownjohn & Xia, 2000). It has already been established in the preceding sections of this thesis that the structural response parameters of interest in this study include displacements (δ), natural frequencies (ω) and mode shapes (ψ). The selection of variables to update is a crucial step, the details of which are specified in section 3.2.3. Selecting updating

variables that most effectively improve the numerical model of the structure ensures that the updating problem is well-conditioned (Brownjohn & Xia, 2000). The most effective updating parameters are normally identified through a sensitivity analysis (Ereiz et al., 2022) which is the focus of this chapter. Model tuning will be considered in the forthcoming chapter (Chapter 7).

This chapter is organised as follows: the methodology aspects specific to this chapter are discussed first. After which, the sensitivity analysis of the behaviour of the selected arch dam configurations to load effects and then to material property variations are presented next. The chapter ends with a summary discussion of and the implication of the observations.

6.2 Methodology

The theoretical development relevant to this chapter and the model development (including meshing and application of the boundary conditions) have already been described in chapter 3 and chapter 4 respectively. The aspects relevant to this study are here summarised:

6.2.1 Concrete Arch Dam Configurations Considered in this Study

In addition to the dam configurations considered in chapter 5 (for which the foundation and fluid medium were not modelled), ten arch dam configurations were selected for the sensitivity study in which the dam-foundation-reservoir interaction was considered for further study as indicated in Chapter 5 (section 5.5). The ten selected configurations comprise both single curvature and double curvature arch dam configurations namely: (i) thin arch narrow-U shaped valley, (ii) thin arch narrow-V shaped valley, (iii) thin arch wide-U shaped valley, (iv) thick arch narrow-U shaped valley, and (v) thick arch wide-U shaped valley.

6.2.2 Sensitivity Analysis

Sensitivity analysis that determines the rate of change in a structural response in relation to the variation in a structural parameter (sensitivity coefficient), allows for the identification of the variables the structural model is responsive and/or unresponsive to, towards the selection of the most suitable variables for model tuning (Brownjohn & Xia, 2000). As was discussed in section 3.2.4, for the selected responses and parameters, the absolute sensitivity of response, i , to change in updating parameter, j , is given by equation (6.1) in building the sensitivity matrix, $[S]$ and illustrated in Figure 6.1 assuming a linear relationship between the response and parameter:

$$S_{ij} = \left[\frac{\partial z_i}{\partial \theta_j} \right] \quad (6.1)$$

where $i=1,2,\dots,m$ for m responses and $j=1,2,\dots,p$ for p parameters.

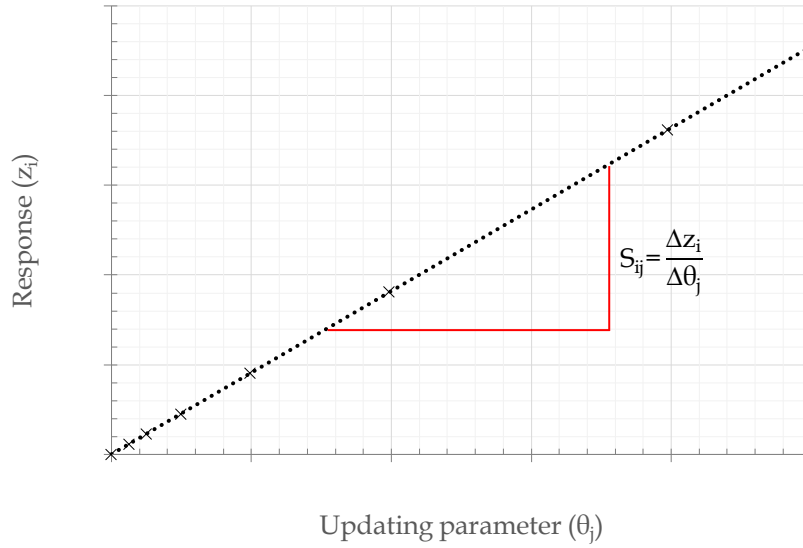


Figure 6.1: Illustration of the sensitivity of response i to change in updating parameter j

The sensitivity matrix can be directly derived for all physical element characteristics (material, geometrical, boundary, etc.) depending on the proportionality of the property to mass and stiffness (Brownjohn & Xia, 2000), as well as for load effects (non-physical properties) (Wittrick, 1962) as illustrated in section 3.3.2. Concerning the latter, the sensitivity of the selected responses to load effects (stress-stiffening) is mainly considered for the stress induced by ASR assuming a maximum volumetric expansion of up to 0.5% (which is equivalent to a thermally applied (illustrated in Figure 6.2) expansion with a temperature increase of up to 500 °C for $\alpha_c = 1.0 \times 10^{-5}$) (Saouma, 2014). Operational temperature (assuming ambient temperature variation from about 0 °C to 50 °C) induced stresses are considered in comparison with the ASR induced stresses for all the arch dam configurations considered in the previous chapter (Chapter 5).

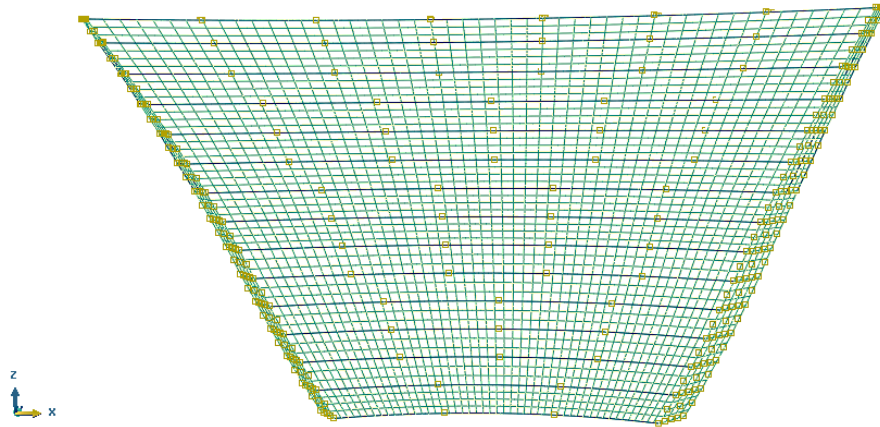


Figure 6.2: Illustration of applied thermal load

Additionally, the sensitivity of the selected responses to variation in the water load is also investigated, firstly for the hydrostatic load related stress-stiffness (applied as shown in Figure 6.3(a)) and secondly for the added mass effect on the dynamic response of the selected arch dam configuration (in which the foundation and reservoir was included) (Figure 6.3(b)). The hydrostatic load sensitivity was analysed for all the selected arch dam configurations assuming similar boundary conditions applied on just the dam wall (Figure 4.13, section 4.3.9), whilst for the added mass investigation, the full dam-foundation-reservoir interaction was modelled (Figure 6.3(b)) and the boundary conditions applied as in section 4.3.9.

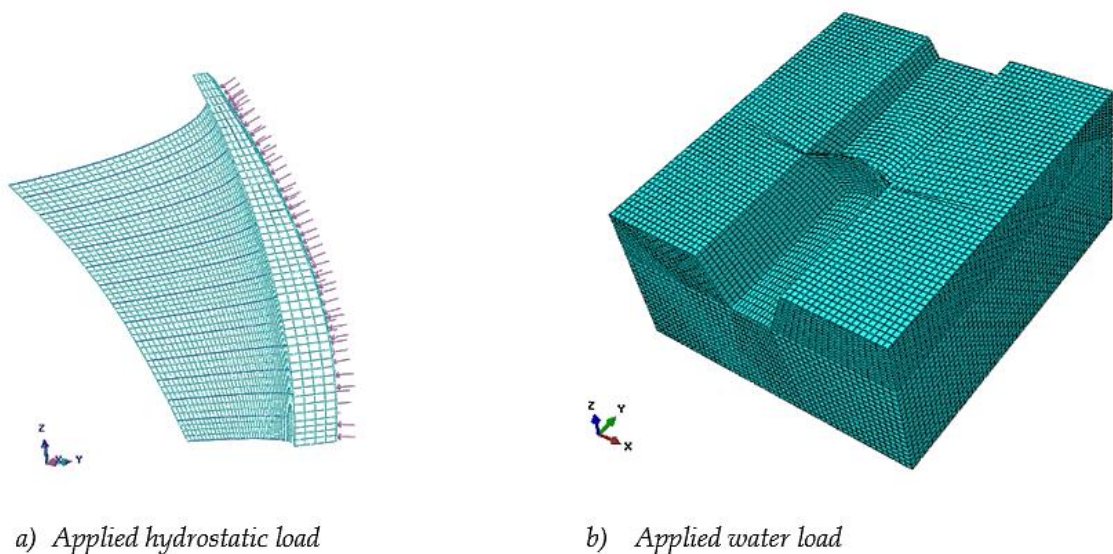


Figure 6.3: Illustration of the applied hydrostatic and added water load

Concerning the sensitivity investigation of the physical properties (Figure 6.4), only the sensitivity of the selected responses to variation in material properties (of the selected arch dam configurations including the dam-foundation interaction) are discussed in this chapter. For the material properties defined for both the rock foundation and concrete wall (section 4.3.8), the sensitivity of the selected responses to variations in the ratio of the foundation material property/concrete wall material property is considered. Additionally, the sensitivity of the selected responses to variation in the coefficient of thermal expansion of the concrete (α_c) (varied between 5×10^{-6} and 20×10^{-6} (Owens, 2009)) and concrete density (variation in concrete density between 1900 kg/m^3 and 2600 kg/m^3 for normally classified concrete (Owens, 2009)), keeping all other factors constant, is presented in this chapter considering just the dam wall model as in the stress-stiffness investigation.

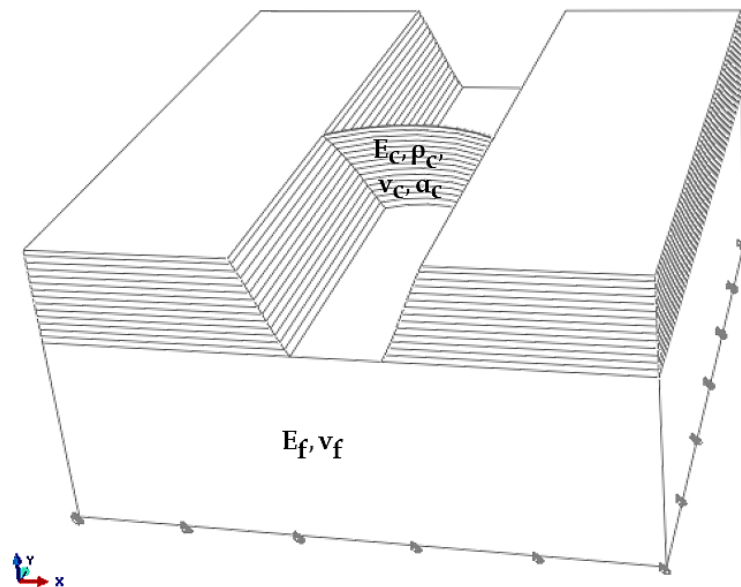


Figure 6.4: Material properties varied in the sensitivity study

6.2.3 Modelling Consideration

The modelling considerations for the arch dam configurations without the foundation and/or fluid are as specified in section 5.2.2. For the arch dam configurations for which the foundation and/or fluid is modelled, the boundary conditions, dam-foundation-reservoir interaction and elements used for meshing are as defined in section 4.3. Similarly to the structural behaviour characterisation study, the Lanczos method was used to extract the Eigen properties of each dam configuration. In addition, under normal operating conditions, the temperature

distribution varies spatially throughout the dam structure's cross-section at any instance due to factors such as heat transfer, the sun's trajectory, dam configuration and water level among other factors. Applying a constant temperature load, although not likely to ever occur, is a conservative approach which could be used to define the bounds of the expected thermal displacements if one applies the maximum change in temperature expected.

6.2.4 Presentation of the sensitivity analysis results

For easy comparison across the different arch dam configurations and to eliminate the influence of other parameters on the analysis such as the influence of the stiffness ratio in the analysis for which the foundation is include, the frequency and displacement are presented as ratios as follows:

- i. For the analysis results in which a fixed boundary condition at the structure-foundation interface is assumed and for sensitivity analysis of the variation of stiffness, the natural frequency results are presented as a ratio of the (respective symmetric and antisymmetric) fundamental frequency (ω^s_1 & ω^a_1 respectively) assuming fixed boundary conditions. Similarly, the displacement results are presented as a ratio of the displacement at $\Delta T = 10^\circ\text{C}$ ($\delta_{\Delta T=10^\circ\text{C}}$) assuming fixed boundary condition.
- ii. To eliminate the influence of the stiffness ratio on the sensitivity analysis of Poisson's ratio, the frequency and displacements are presented normalised to the lowest Poisson's ratio relation investigated in this study.

6.3 Structural Response Sensitivity to Variation in Selected Service Loads

The sensitivity of the selected responses to ambient temperature conditions is first discussed leading up to the ASR related sensitivity analysis in which the ASR related volumetric strains were induced by applying a temperature load. The objective is to find the relationship between the development of stress-stiffness due to ASR-related volumetric expansion and its impact on the structural behaviour of concrete arch dams. Next, the sensitivity of the ten selected configurations' response to the hydrostatic stress effects on both the dynamic and static response is considered. Lastly, the added mass effect due to the water load on the dynamic response of the selected arch dam configurations, including the dam-foundation-fluid interaction is then investigated.

6.3.1 Variation of natural frequencies with temperatures

Figure 6.5 and Figure 6.6 show the variation of the natural frequencies (expressed as a ratio of the relevant fundamental frequency (refer to section 6.2.4)) with thermal loading. The rate of change (i.e., sensitivity) in natural frequencies to the applied thermal loading is constant for each presented case. However, the sensitivity of the natural frequencies to the applied thermal load varies for different configuration depending on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. Table 6.1 lists the individual sensitivities (as a percentage) corresponding to the graphs in Figure 6.5 and Figure 6.6.

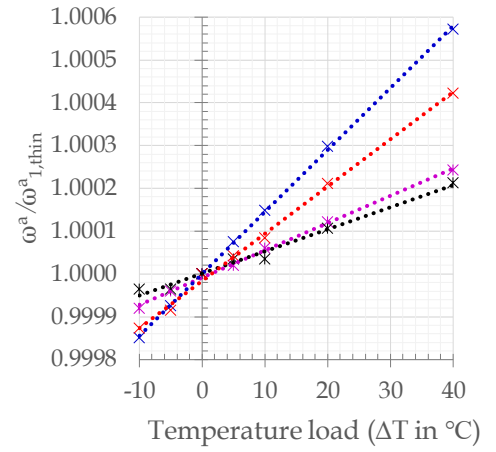
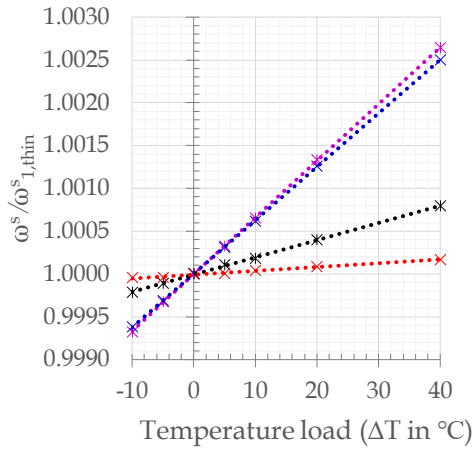
The first symmetric mode shape for the thin narrow-V shaped single curvature arch configuration is the most sensitive to the applied load followed by the thin narrow-U shaped single arch configuration symmetric mode (Table 6.1). Whereas for the double curvature arch configurations, the first symmetric mode shape for the thin narrow-U shaped arch is the most sensitive to the applied load, followed by that of the thin narrow-V shaped arch configuration (Table 6.1). The symmetric modes are generally more sensitive than the antisymmetric modes. In some cases, there seems to be a difference in sensitivities from the graphic illustration point of view, but when the actual sensitivities are compared the difference is insignificant, such as the thick double arch narrow U, narrow V, and wide V configurations, as the frequencies are only sensitive to the 4th decimal place.

Table 6.1: Sensitivity of the first two arch modes to the applied thermal load ([Hz/°C] %)

Arch curvature	Arch class	Arch Mode	Narrow U 	Narrow V 	Wide V 	Wide U 
single	thin	1 st -S	0.006	0.007	0.002	0.0004
		1 st -A	0.001	0.0006	0.0005	0.001
	medium-thick	1 st -S	0.003	0.002	0.0003	-0.00007
		1 st -A	-0.0004	-0.001	-0.001	-0.0004
	thick	1 st -S	0.001	0.0005	-0.0001	0.0001
		1 st -A	-0.0008	-0.001	-0.001	-0.0008
double	thin	1 st -S	0.009	0.007	0.005	0.0005
		1 st -A	-0.0004	-0.002	-0.002	0.003
	medium-thick	1 st -S	0.005	0.002	0.0007	-0.0009
		1 st -A	-0.001	-0.003	-0.003	-0.0005
	thick	1 st -S	0.002	0.0006	-0.0002	-0.0002
		1 st -A	-0.002	-0.002	-0.002	-0.001

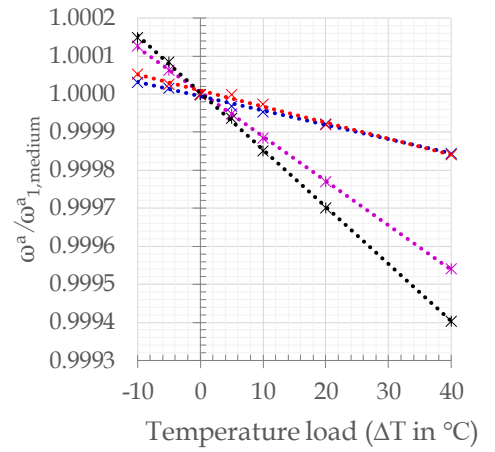
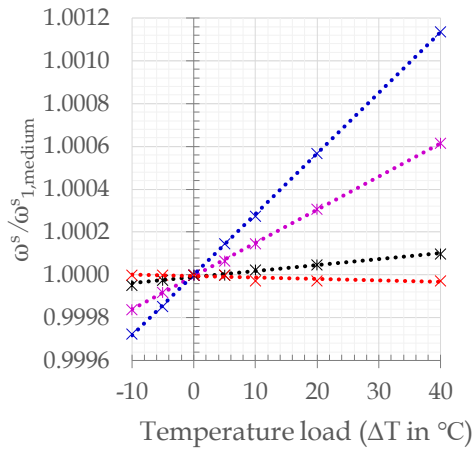
A – antisymmetric mode

S – symmetric mode



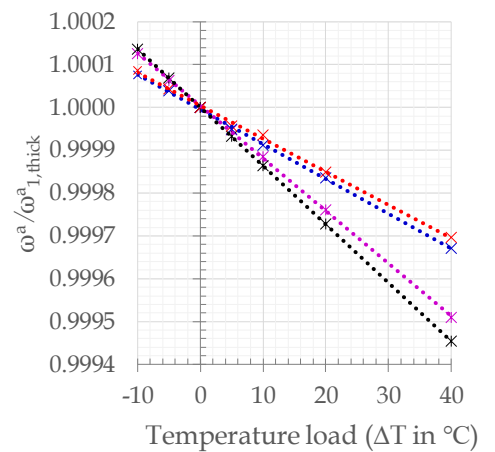
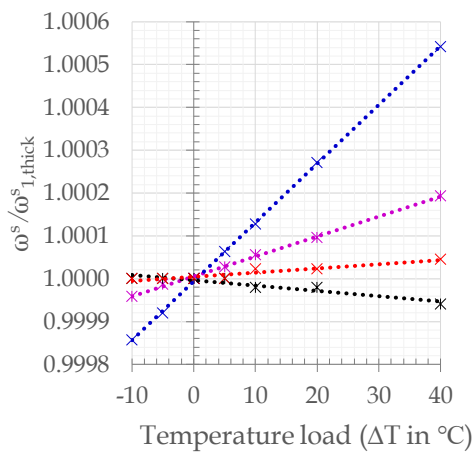
b) Thin arch first symmetric mode

b) Thin arch first antisymmetric mode



c) Medium-thick arch first symmetric mode

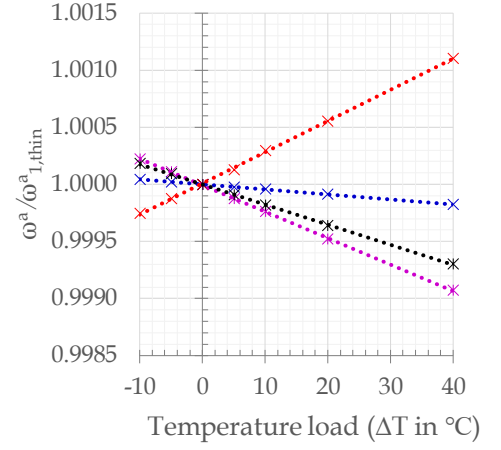
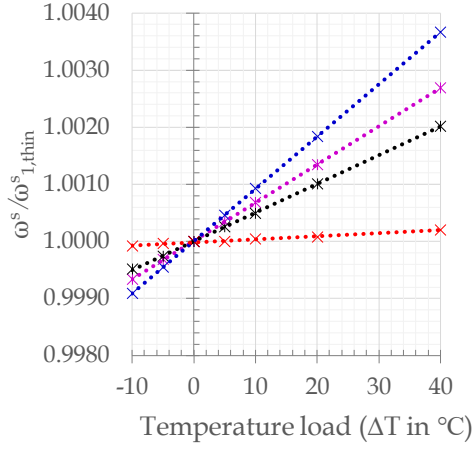
d) Medium-thick arch first antisymmetric mode



e) Thick arch first symmetric mode

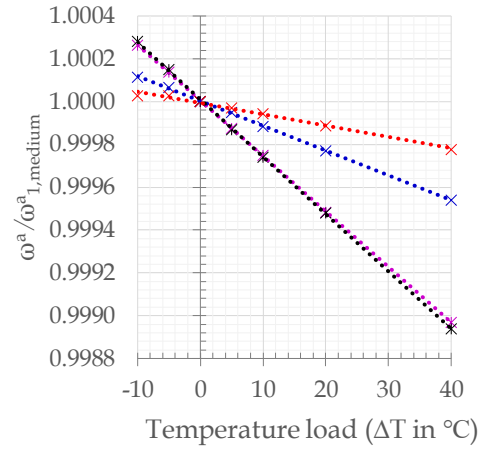
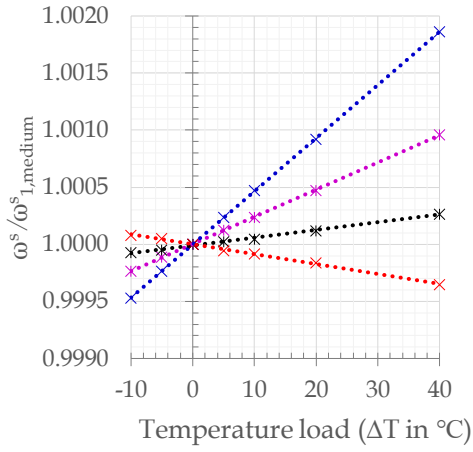
f) Thick arch first antisymmetric mode

Figure 6.5: ω/ω_1 versus applied thermal load graphs for single curvature arch configurations



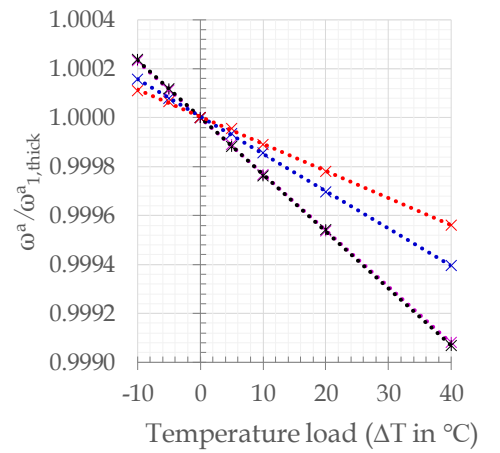
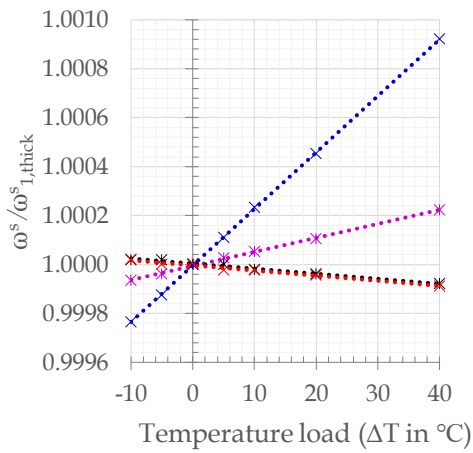
a) Thin arch first symmetric mode

b) Thin arch first antisymmetric mode



c) Medium-thick arch first symmetric mode

d) Medium-thick arch first antisymmetric mode

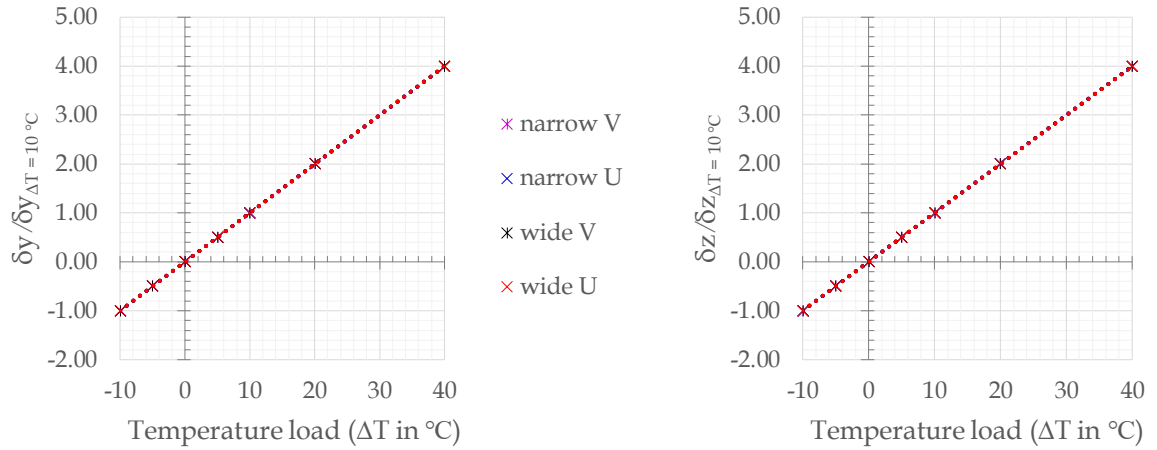


e) Thick arch first symmetric mode

f) Thick arch first antisymmetric mode

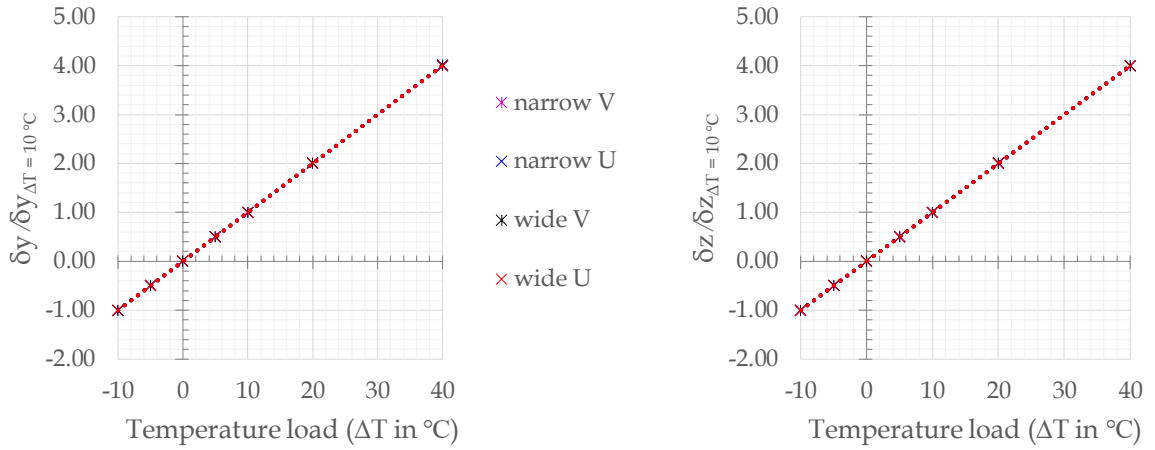
 Figure 6.6: ω/ω_1 versus applied thermal load graphs for double curvature arch configurations

6.3.2 Variation of displacements with temperatures



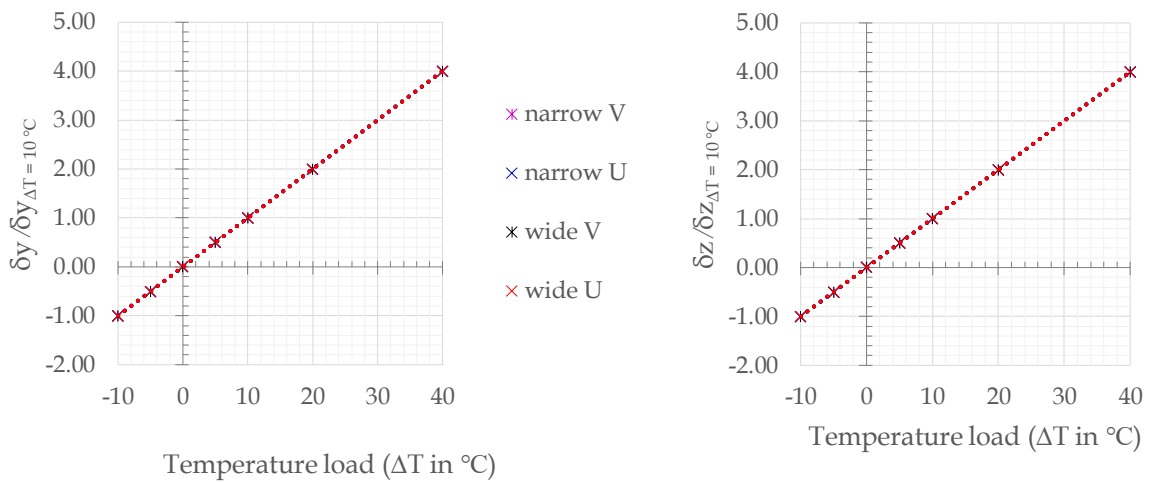
a) Thin arch radial displacements

b) Thin arch vertical displacements



c) Medium-thick arch radial displacements

d) Medium-thick arch vertical displacements



e) Thick arch radial displacements

f) Thick arch vertical displacements

Figure 6.7: Typical $\delta/\delta_{\Delta T=10^{\circ}\text{C}}$ versus applied thermal load graphs for arch dams

Figure 6.7 shows the typical variation of displacements (expressed as a ratio of the displacements for $\Delta T = 10^\circ\text{C}$ as detailed in section 6.2.4). The sensitivity ($[\text{mm}/^\circ\text{C}] \%$) of the displacements to the applied thermal load remains constant, equal to 10%, and is the same for all the investigated arch configurations irrespective of the type of displacement (radial or tangential), valley type, arch thickness, and arch dam curvature classifications.

6.3.3 Variation of natural frequencies with ASR related volumetric expansion

Figure 6.8 and Figure 6.9 show the variation of the natural frequencies (expressed as a ratio of the relevant fundamental frequencies determined assuming an equivalent volumetric strain for $\Delta T = 10^\circ\text{C}$) with volumetric expansion expressed (as a percentage) for the first symmetric and first antisymmetric modes for the thin, medium-thick, and thick classified single curvature and double curvature arch configurations respectively. Similarly to the dynamic response sensitivity to variations in thermal loading due to ambient temperatures, the rate of change (i.e., sensitivity) in natural frequencies to volumetric expansion is constant for each investigated configuration. However, the sensitivity of the natural frequencies to the applied ASR related volumetric expansion varies across the investigated configurations depending on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. Table 6.2 lists the individual sensitivities (as a percentage) corresponding to the graphs in Figure 6.8 and Figure 6.9.

Table 6.2: Sensitivity of natural frequencies to ASR related volumetric expansion ($[\text{Hz} / (\epsilon_v (\%))] \%$)

Arch curvature	Arch class	Arch Mode	 Narrow U	 Narrow V	 Wide V	 Wide U
single	thin	1 st -S	2.48	2.70	0.75	0.13
		1 st -A	0.55	0.23	0.15	0.39
	medium-thick	1 st -S	1.23	0.69	0.12	-0.04
		1 st -A	-0.15	-0.5	-0.6	-0.15
	thick	1 st -S	0.61	0.21	-0.05	0.04
		1 st -A	-0.35	-0.54	-0.58	-0.31
double	thin	1 st -S	3.59	2.92	2.02	0.19
		1 st -A	-0.17	-0.94	-0.67	1.07
	medium-thick	1 st -S	2.00	1.12	0.3	-0.37
		1 st -A	-0.47	-1.13	-1.10	-0.20
	thick	1 st -S	1.04	0.3	-0.07	-0.1
		1 st -A	-0.64	-1.06	-0.99	-0.45

A - antisymmetric
S - symmetric

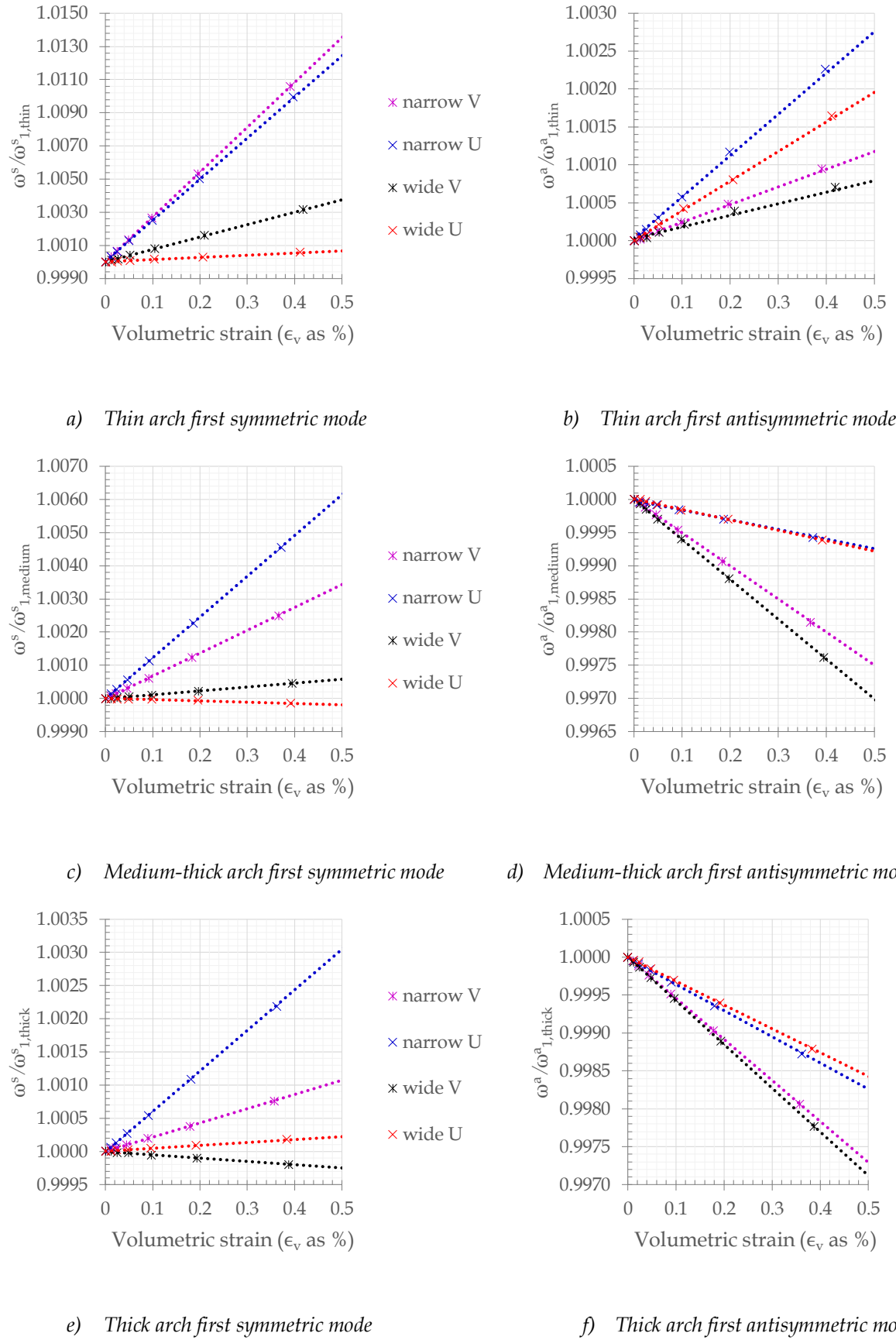
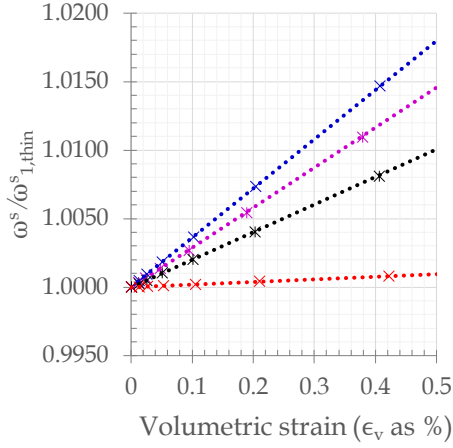
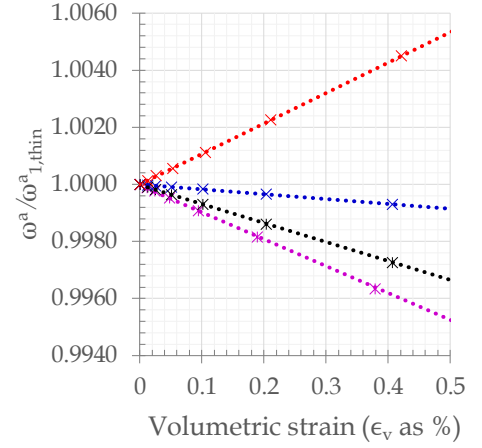


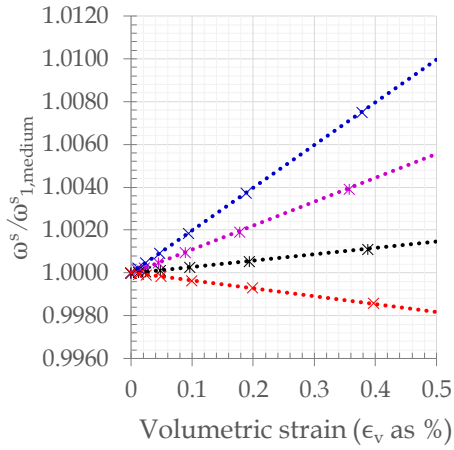
Figure 6.8: ω/ω_1 versus ϵ_v graphs for single curvature arch configurations



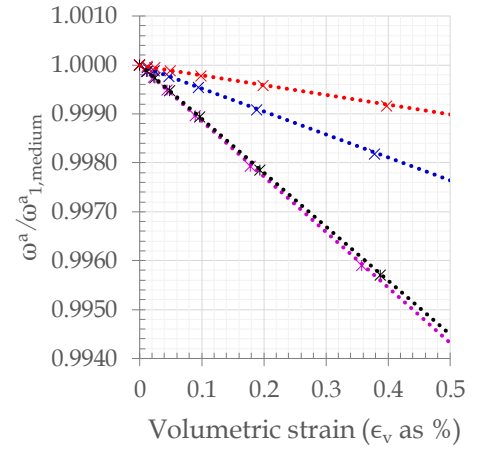
a) Thin arch first symmetric mode



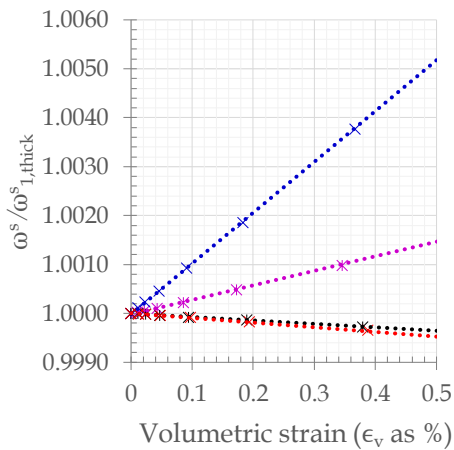
b) Thin arch first antisymmetric mode



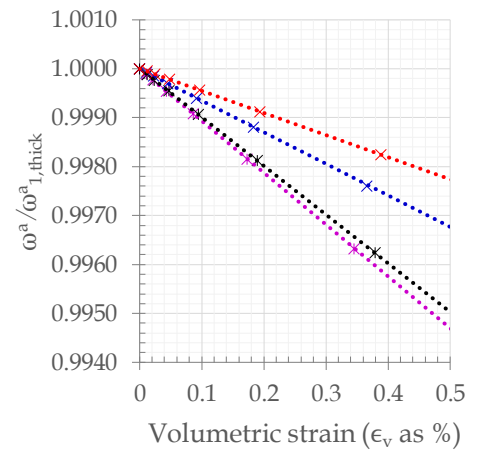
c) Medium-thick arch first symmetric mode



d) Medium-thick arch first antisymmetric mode



e) Thick arch first symmetric mode



f) Thick arch first antisymmetric mode

 Figure 6.9: ω/ω_1 versus ϵ_v graphs for double curvature arch configurations

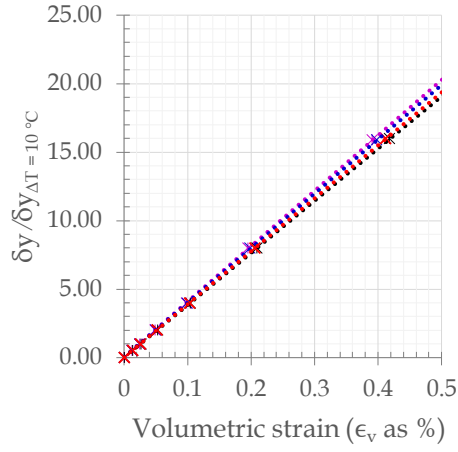
The most sensitive single curvature arch configuration modal parameters to volumetric expansion corresponds to the thin narrow-V shaped arch symmetric mode followed by the thin narrow-U shaped arch configuration mode (Table 6.2). Whilst the thin narrow-U shaped arch configuration's symmetric mode is the most sensitive for the double curvature arch configurations, followed by the thin narrow-V shaped arch symmetric mode (Table 6.2). The symmetric modes are generally more sensitive than the antisymmetric modes.

6.3.4 Variation of displacements with ASR related volumetric expansion

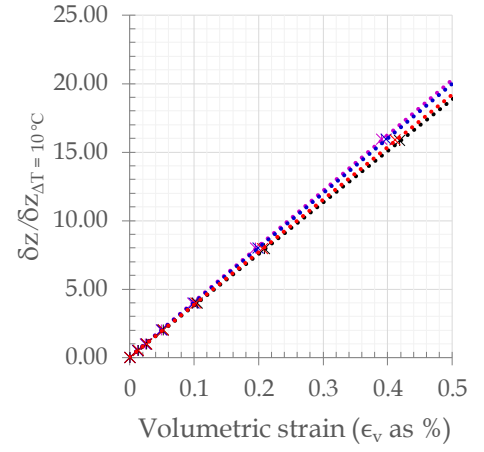
Table 6.3: Sensitivity of the radial and vertical displacements to volumetric strain ($[mm / (\epsilon_v (\%))] \%$)

Arch curvature	Arch class	Displacement type	Narrow U	Narrow V	Wide V	Wide U
single	thin	radial	3971.1	4044.5	3808.1	3866.8
		vertical	3991.6	4048.7	3758.6	3830.7
	medium-thick	radial	4286.8	4371.7	4068.8	4088.8
		vertical	4275.4	4335.1	4015.6	4044.8
	thick	radial	4422.8	4495.2	4147.9	4176.1
		vertical	4399.2	4458.3	4105.6	4138.9
double	thin	radial	3876.4	4212.6	3936.5	3764.9
		vertical	3866.2	4172.1	3874.3	3742.8
	medium-thick	radial	4212.8	4513.9	4165.5	4016.5
		vertical	4193.7	4461.8	4109.9	3993.7
	thick	radial	4380.2	4688.6	4272.3	4131.8
		vertical	4350.9	4626.6	4217.1	4087.7

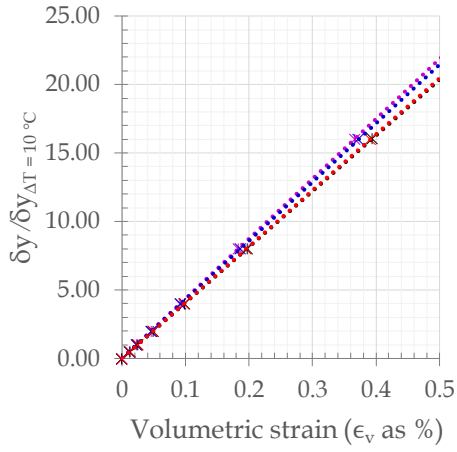
Figure 6.10 and Figure 6.11 show the variation of the displacements (expressed as a ratio of the displacements determined assuming an equivalent volumetric strain for $\Delta T = 10^\circ C$) with volumetric expansion expressed (as a percentage). Whilst Table 6.3 lists all the sensitivities (as a percentage) corresponding to graphs in Figure 6.10 and Figure 6.11. The rate of change (i.e., sensitivity) in displacements to volumetric expansion is constant for each arch dam configuration, showing some variation depending on the types of displacement (radial or tangential), valley type, arch thickness, and arch dam curvature classifications for the presented cases (Figure 6.10 and Figure 6.11). However, the narrow V arch configurations consistently have the highest sensitivities for all the displacement types across all the investigated configurations Table 6.3). On the other hand, the wide V configuration show the least displacement sensitivity for all single curvature arch configuration (Table 6.3) whilst the



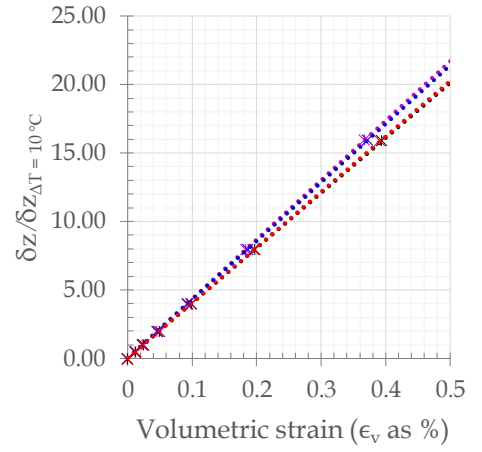
a) Thin arch radial displacements



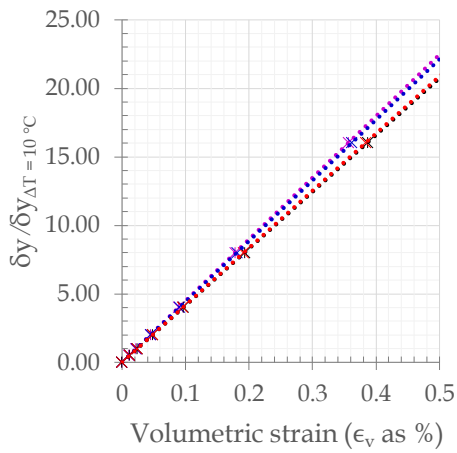
b) Thin arch vertical displacements



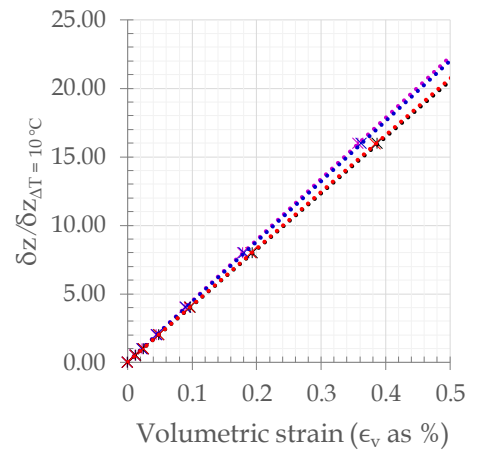
c) Medium-thick arch radial displacements



d) Medium-thick arch vertical displacements

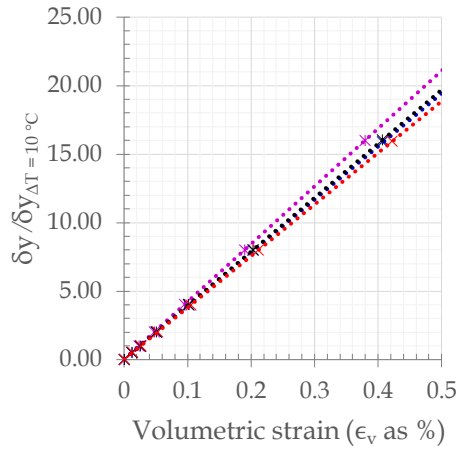


e) Thick arch radial displacements

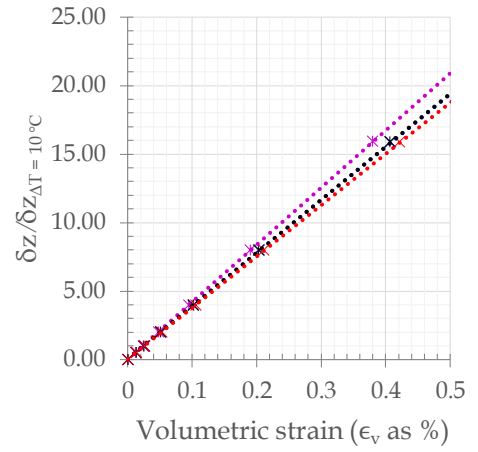


f) Thick arch vertical displacements

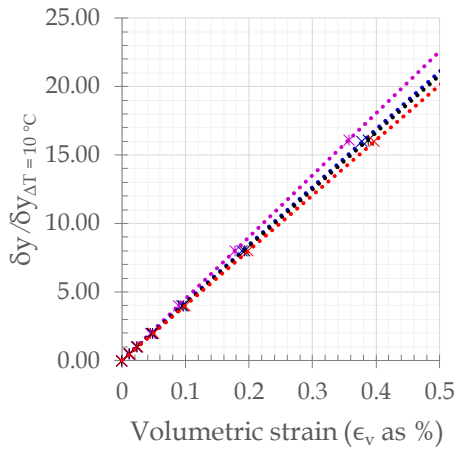
Figure 6.10: $\delta/\delta_{\Delta T=10^\circ C}$ versus ϵ_v graphs for single curvature arch configurations



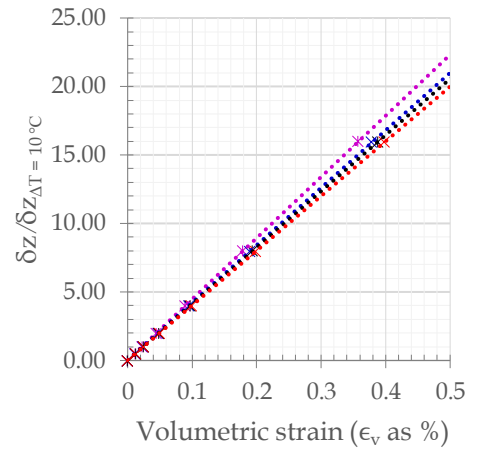
a) Thin arch radial displacements



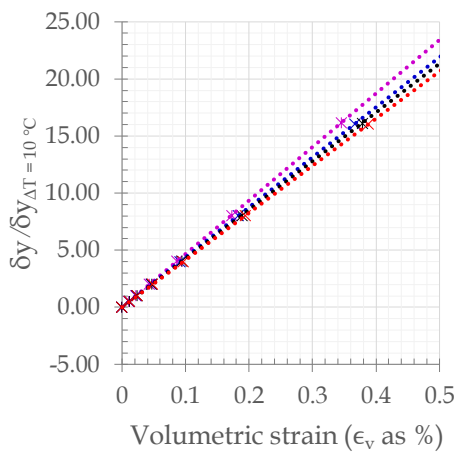
b) Thin arch vertical displacements



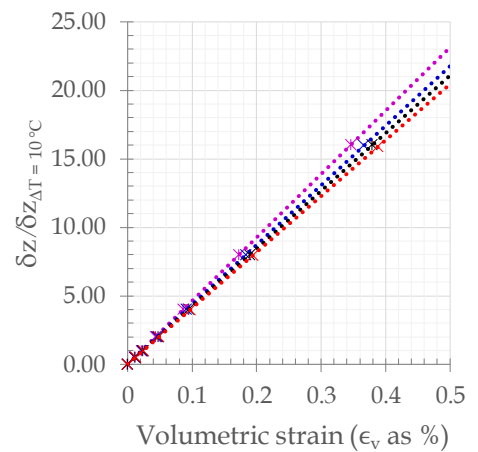
c) Medium-thick arch radial displacements



d) Medium-thick arch vertical displacements



e) Thick arch radial displacements



f) Thick arch vertical displacements

Figure 6.11: $\delta/\delta_{\Delta T=10^\circ C}$ versus ϵ_v graphs for double curvature arch configurations

wide U configurations consistently show the least displacement sensitivity to volumetric expansion for all double curvature arch configurations (Table 6.3).

As illustrated in Figure 6.3a, the hydrostatic load was applied on dam configurations in which the dam wall-foundation interface was modelled assuming a fixed boundary condition. The corresponding results are here presented starting with the dynamic response results followed by the static response observations next:

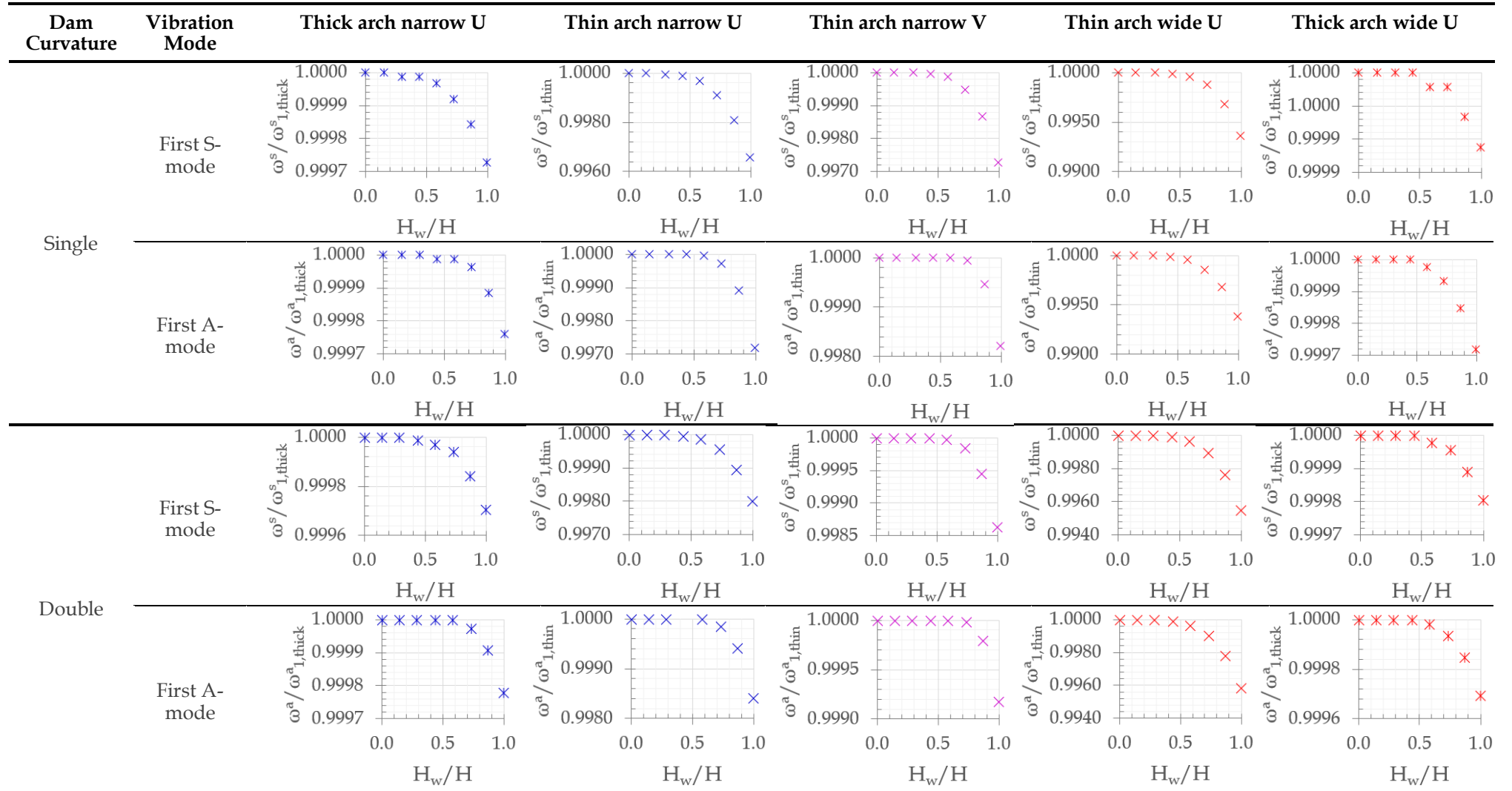
6.3.5 Variation of natural frequencies with hydrostatic loading

Table 6.4 shows the effect of the hydrostatic load on the natural frequencies of the ten selected arch dam configurations. For all the configurations, the rate of change (i.e., sensitivity) in the natural frequency ratio of the first symmetric and antisymmetric modes decreases as the water level decreases (approaching the frequency value for the empty dam) till there is no change in natural frequency with any further decrease in water level between an H_w/H value of 0.15 & 0.7 depending on the arch dam configuration and mode shape (Figure 6.12 and Figure 6.13). The total decrease in natural frequencies due to the hydrostatic loading varies depending on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. Generally, the investigated arch configuration antisymmetric modes reach their corresponding empty dam frequency values at much higher water levels (about an H_w/H of 0.7 depending on the arch dam configuration) as compared with the symmetric mode shape frequencies (which reach the corresponding empty dam frequency value below an H_w/H of 0.5) for both single curvature and double curvature arch configurations (Figure 6.12 and Figure 6.13). Additionally, the rate of change in natural frequency (for both symmetric and antisymmetric modes) generally increases from U- to V- shaped valley types. The difference in the rate of change in natural frequencies generally decreases as the thickness class increases (Figure 6.12 and Figure 6.13).

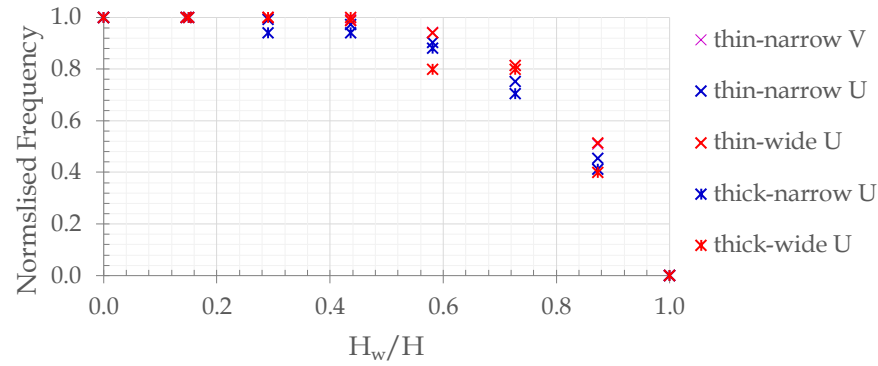
6.3.6 Variation of displacements with hydrostatic loading

Table 6.5 shows the effect of the hydrostatic load on the displacements (expressed as a ratio) for the ten selected arch dam configurations. Similarly to the dynamic response, the rate of change in the displacement ratios (both radial and vertical) decreases as the water level decreases until there is no change in displacements below an H_w/H of 0.5 for all radial displacements and only the vertical displacements for the single curvature arch configurations. The double curvature arch configuration vertical displacements generally start

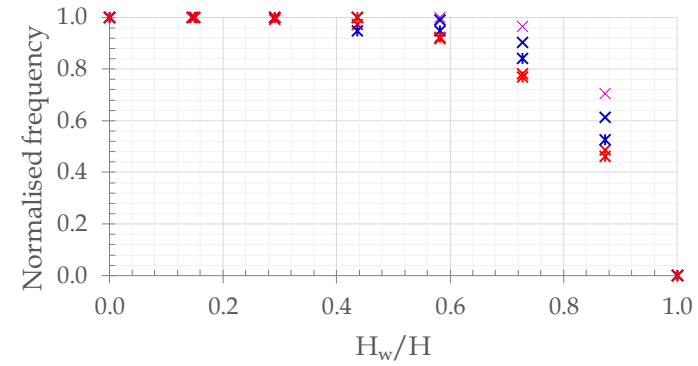
Table 6.4: Effect of hydrostatic load on the dynamic behaviour of selected arch dam configurations



A, a - antisymmetric mode
S, s - symmetric mode

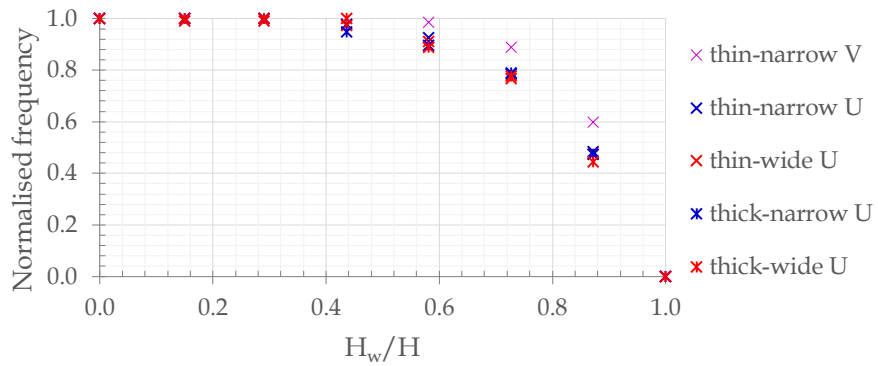


a) First symmetric mode

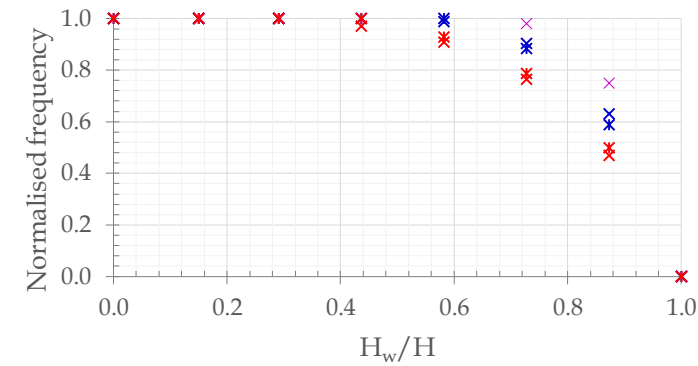


b) First antisymmetric mode

Figure 6.12: Normalised natural frequencies versus hydrostatic load for selected single curvature arch configurations



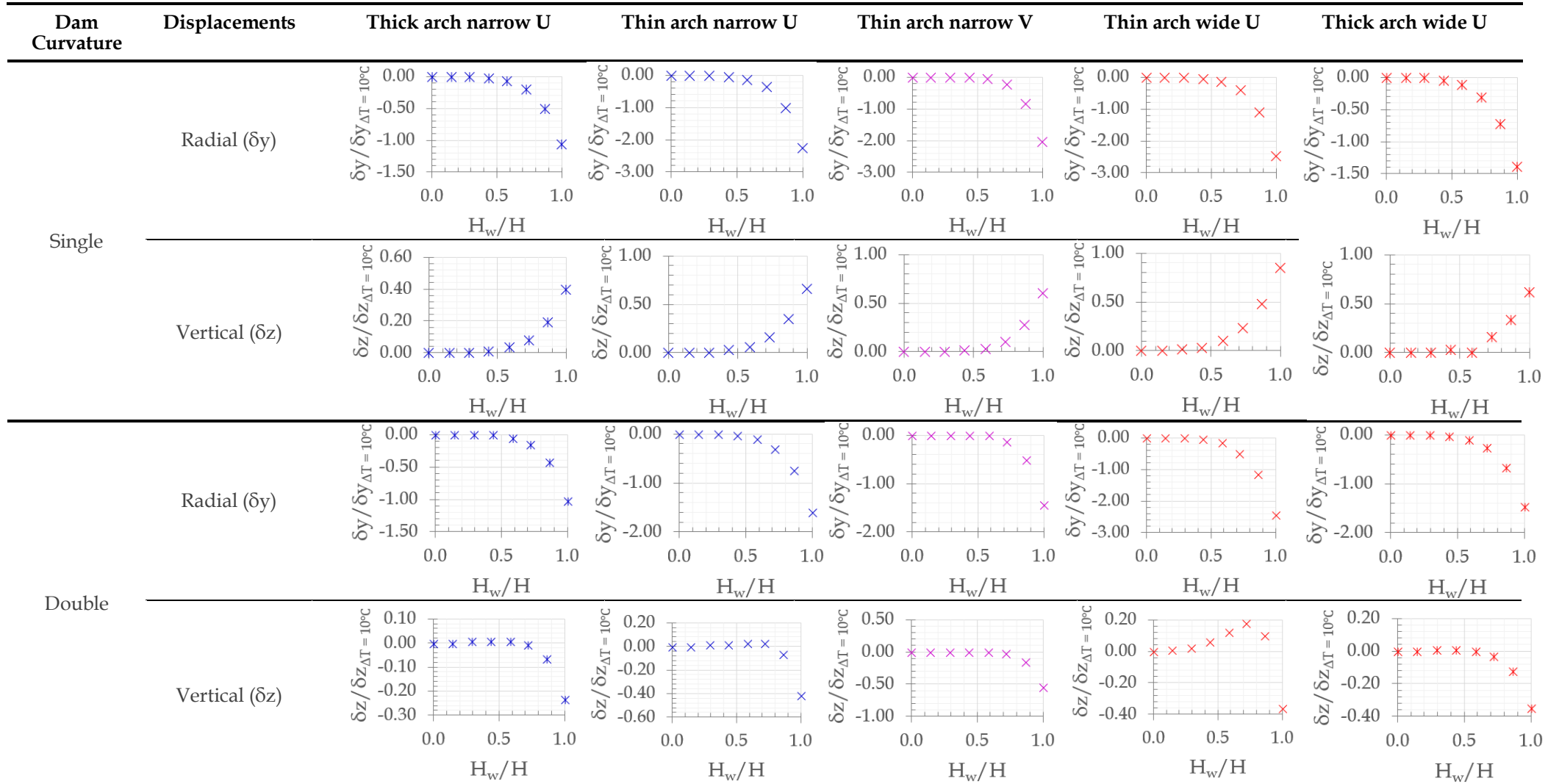
a) First symmetric mode

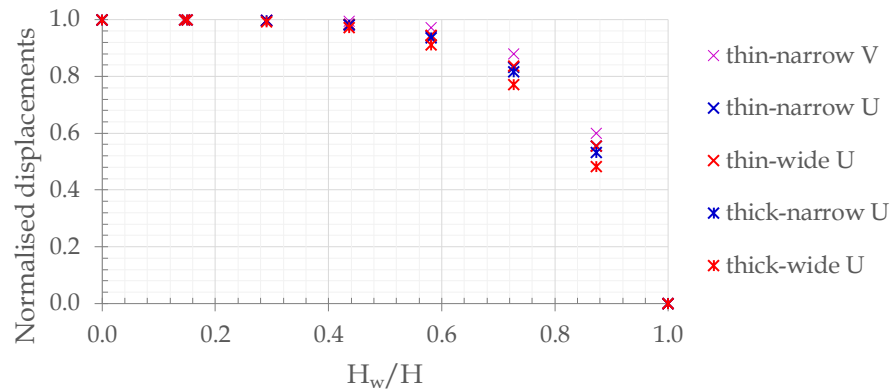


b) First antisymmetric mode

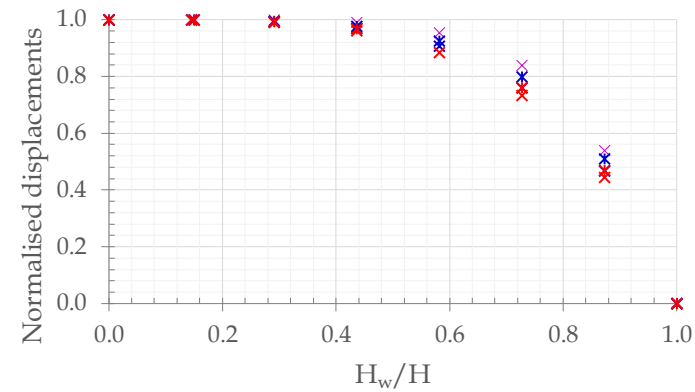
Figure 6.13: Normalised natural frequencies versus hydrostatic load for selected double curvature arch configurations

Table 6.5: Effect of hydrostatic load on the static behaviour of selected arch dam configurations



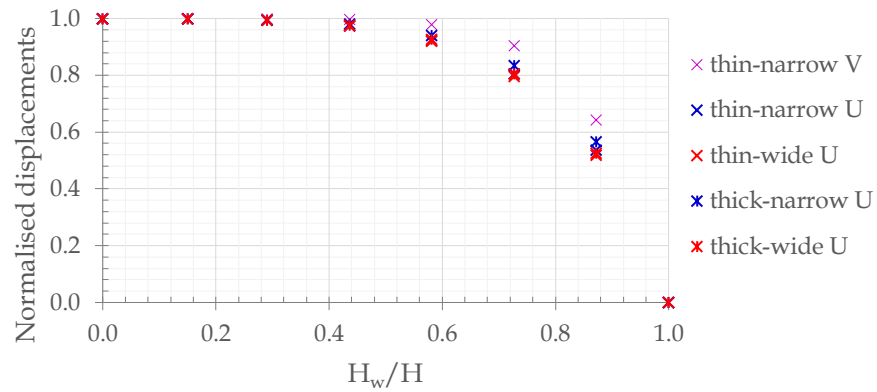


a) Radial (y-) displacements

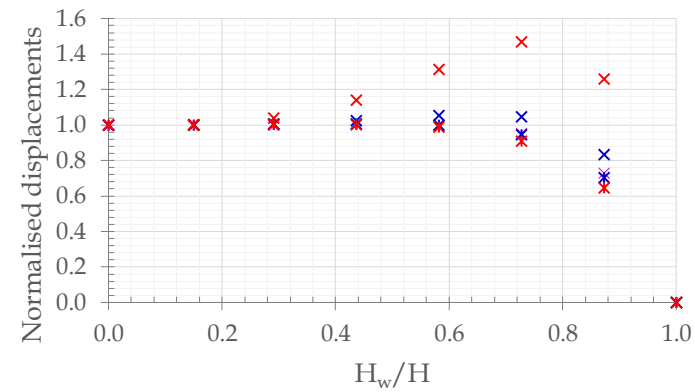


b) Vertical (z-) displacements

Figure 6.14: Normalised displacements versus hydrostatic load for selected single curvature arch configurations



a) Radial (y-) displacements



b) Vertical (z-) displacements

Figure 6.15: Normalised displacements versus hydrostatic load for selected double curvature arch configurations

to be constant at higher water levels. Additionally, the thin U-shaped double curvature arch configuration vertical displacements have a different graph profile compared to the rest of the configurations. The total change in displacements due to the hydrostatic loading varies depending on the type of displacement, valley type, arch thickness, and arch dam curvature classifications (refer to the Appendix for further details). Also, the rate of change in displacements (for both single and double curvature configurations) generally increases from wide to narrow and from U- to V- shaped valley types except for the thin double curvature U shaped valley arch configurations (Figure 6.14 and Figure 6.15).

As illustrated in Figure 6.3b, the effect of added mass due to water level was investigated for a system of the selected arch dam configurations consisting of the dam, foundation, and reservoir. An $E_f/E_c = 1$ and $v_f/v_c = 1$ was implemented throughout the analysis. Consequently, it became necessary to compare the dynamic behaviour of the selected arch dam configurations with the different boundary conditions implemented throughout this study. Additionally, the dynamic response of the full dam case was considered in comparison. After which, the sensitivity results of the dynamic behaviour of the selected arch dam configurations to variation in water level, considering the added mass effect, will be discussed next:

6.3.7 Influence of the dam-foundation-fluid system on the dynamic behaviour of arch dams

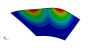
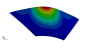
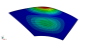
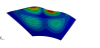
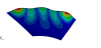
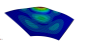
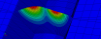
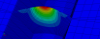
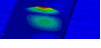
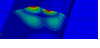
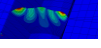
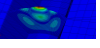
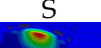
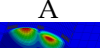
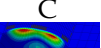
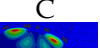
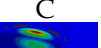
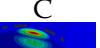
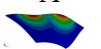
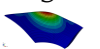
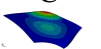
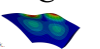
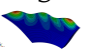
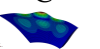
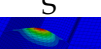
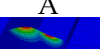
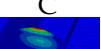
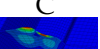
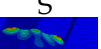
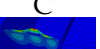
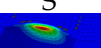
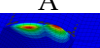
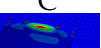
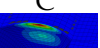
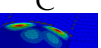
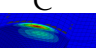
Table 6.6 to Table 6.10 provide a summary of the theoretical modal parameters considering different boundary conditions and the dam-foundation-fluid system interaction(s). As can be seen from these results, some of the selected arch dam configuration(s) mode shape sequences changed as the dam-foundation interface/interaction changed (such as the thick narrow-U & wide-U arch configurations), whilst other configuration mode shape sequences remained the same (such as the thin narrow-U & narrow-V arch configurations). The mode shape sequences of all the selected configurations changed with the addition of the full dam fluid medium (i.e., including the fluid-structure interaction approach). The difference in natural frequency values between the fixed boundary condition (i.e., $E_f/E_c = \infty$) and $E_f/E_c = 1$ is discussed in detail in section 6.5.1 whilst the influence of the water level is discussed in the section that follows. There is clearly an influence of the boundary condition and water level (i.e., dam-foundation-fluid system and its interaction) on the modal parameters (both natural frequency values and mode shape sequence) of the selected arch dam configurations.

Table 6.6: Thick narrow-U shaped dam configuration(s) theoretical vibration modal parameters

Dam Curvature	Conditions	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Single	Fixed boundary, empty dam	 S 6.27 Hz	 A 7.90 Hz	 S 10.90 Hz	 C 12.61 Hz	 A 14.22 Hz	 C 16.26 Hz	 S 17.98 Hz	 C 19.34 Hz	 C 19.46 Hz	
	$E_f/E_c=1$, empty dam	 S 5.62 Hz	 A 7.40 Hz	 S 10.37 Hz	 C 11.03 Hz	 A 13.56 Hz	 C 13.77 Hz	 C 14.00 Hz	 C 15.08 Hz	 S 17.47 Hz	 C 18.52 Hz
	$E_f/E_c=1$, full dam	 S 3.88 Hz	 S 4.68 Hz	 S 5.47 Hz	 A 5.73 Hz	 S 6.75 Hz	 C 7.82 Hz	 S 8.31 Hz	 C 8.39 Hz	 C 8.74 Hz	 C 9.79 Hz
Double	Fixed boundary, empty dam	 S 6.41 Hz	 A 7.60 Hz	 S 10.46 Hz	 C 12.00 Hz	 A 13.47 Hz	 C 15.30 Hz	 S 16.85 Hz	 C 18.84 Hz		
	$E_f/E_c=1$, empty dam	 S 5.87 Hz	 A 7.26 Hz	 S 10.15 Hz	 C 10.73 Hz	 A 13.29 Hz	 C 13.51 Hz	 C 14.21 Hz	 C 15.14 Hz	 S 16.77 Hz	 C 17.65 Hz
	$E_f/E_c=1$, full dam	 S 4.36 Hz	 S 4.71 Hz	 S 5.55 Hz	 A 5.95 Hz	 S 7.05 Hz	 C 7.67 Hz	 S 8.54 Hz	 C 8.64 Hz	 C 8.75 Hz	 C 9.86 Hz

A - antisymmetric mode
C - complex mode
S - symmetric mode

Table 6.7: Thin narrow-U shaped dam configuration(s) theoretical vibration modal parameters

Dam Curvature	Conditions	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Single	Fixed boundary, empty dam	A 	S 	S 	C 	A 	C 	S 	C 	C 	A 
		4.02 Hz	4.20 Hz	5.91 Hz	6.95 Hz	7.69 Hz	8.63 Hz	10.18 Hz	11.55 Hz	12.46 Hz	13.01 Hz
	$E_f/E_c=1$, empty dam	A 	S 	S 	C 	A 	C 	S 	C 	C 	A 
		3.88 Hz	4.02 Hz	5.74 Hz	6.66 Hz	7.54 Hz	8.36 Hz	10.03 Hz	11.18 Hz	11.93 Hz	12.79 Hz
	$E_f/E_c=1$, full dam	S 	A 	S 	C 	C 	C 	C 	A 	C 	C 
		2.47 Hz	2.67 Hz	4.00 Hz	4.34 Hz	4.70 Hz	5.66 Hz	5.75 Hz	5.99 Hz	7.11 Hz	7.62 Hz
Double	Fixed boundary, empty dam	A 	S 	S 	C 	A 	C 	S 	C 	C 	A 
		4.54 Hz	4.63 Hz	6.86 Hz	7.93 Hz	8.92 Hz	9.99 Hz	11.39 Hz	12.60 Hz	13.81 Hz	14.09 Hz
	$E_f/E_c=1$, empty dam	S 	A 	S 	C 	A 	C 	S 	C 	C 	A 
		4.40 Hz	4.43 Hz	6.78 Hz	7.58 Hz	8.98 Hz	9.79 Hz	11.66 Hz	12.28 Hz	13.63 Hz	14.48 Hz
	$E_f/E_c=1$, full dam	S 	A 	S 	C 	S 	C 	C 	A 	C 	C 
		3.05 Hz	3.33 Hz	4.54 Hz	4.97 Hz	5.28 Hz	6.10 Hz	6.92 Hz	7.37 Hz	7.46 Hz	7.76 Hz

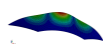
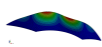
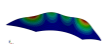
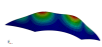
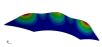
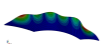
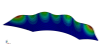
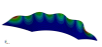
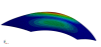
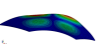
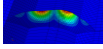
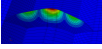
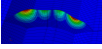
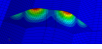
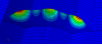
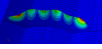
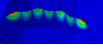
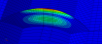
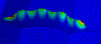
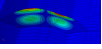
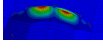
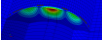
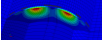
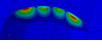
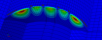
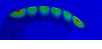
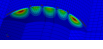
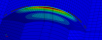
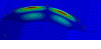
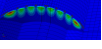
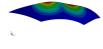
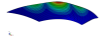
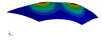
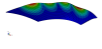
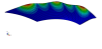
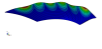
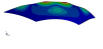
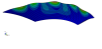
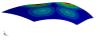
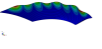
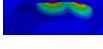
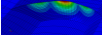
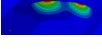
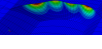
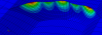
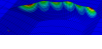
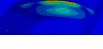
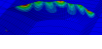
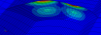
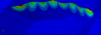
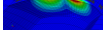
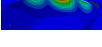
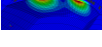
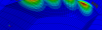
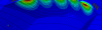
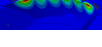
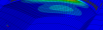
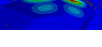
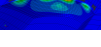
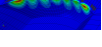
A – antisymmetric mode
C – complex mode
S – symmetric mode

Table 6.8: Thin narrow-V shaped dam configuration(s) theoretical vibration modal parameters

Dam Curvature	Conditions	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Single	Fixed boundary, empty dam	 S 4.58 Hz	 A 4.95 Hz	 S 6.85 Hz	 C 8.70 Hz	 A 9.02 Hz	 S 11.72 Hz	 C 12.02 Hz	 C 14.58 Hz	 A 14.83 Hz	 C 17.01 Hz
	$E_f/E_c=1$, empty dam	 S 4.35 Hz	 A 4.73 Hz	 S 6.60 Hz	 C 8.20 Hz	 A 8.72 Hz	 S 11.37 Hz	 C 11.41 Hz	 C 13.78 Hz	 A 14.43 Hz	 C 16.00 Hz
	$E_f/E_c=1$, full dam	 S 2.80 Hz	 A 3.39 Hz	 S 4.93 Hz	 C 5.17 Hz	 C 5.58 Hz	 C 6.58 Hz	 A 6.74 Hz	 C 7.91 Hz	 C 8.33 Hz	 S 8.98 Hz
Double	Fixed boundary, empty dam	 S 5.28 Hz	 A 6.25 Hz	 S 8.60 Hz	 C 10.82 Hz	 A 11.29 Hz	 S 14.30 Hz	 C 15.29 Hz	 C 17.60 Hz	 A 17.65 Hz	
	$E_f/E_c=1$, empty dam	 S 4.96 Hz	 A 5.88 Hz	 S 8.17 Hz	 C 9.98 Hz	 A 10.74 Hz	 S 13.63 Hz	 C 14.19 Hz	 C 16.19 Hz	 A 16.86 Hz	 C 18.31 Hz
	$E_f/E_c=1$, full dam	 S 3.63 Hz	 A 4.61 Hz	 C 5.59 Hz	 C 6.12 Hz	 S 6.63 Hz	 C 7.26 Hz	 C 8.44 Hz	 A 8.84 Hz	 C 9.96 Hz	 C 10.74 Hz

A – antisymmetric mode
C – complex mode
S – symmetric mode

Table 6.9: Thin wide-U shaped dam configuration(s) theoretical vibration modal parameters

Dam Curvature	Conditions	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Single	Fixed boundary, empty dam	S  2.36 Hz	A  2.37 Hz	A  2.89 Hz	S  2.95 Hz	S  3.39 Hz	A  3.83 Hz	S  4.44 Hz	A  5.12 Hz	C  5.32 Hz	C  5.50 Hz
	$E_f/E_c=1$, empty dam	A  2.28 Hz	S  2.28 Hz	A  2.83 Hz	S  2.90 Hz	S  3.35 Hz	A  3.82 Hz	S  4.46 Hz	C  5.16 Hz	A  5.18 Hz	C  5.35 Hz
	$E_f/E_c=1$, full dam	A  1.45 Hz	S  1.48 Hz	S  1.77 Hz	A  1.87 Hz	S  2.25 Hz	A  2.69 Hz	S  3.22 Hz	C  3.42 Hz	C  3.65 Hz	A  3.81 Hz
Double	Fixed boundary, empty dam	A  2.36 Hz	S  2.46 Hz	S  3.35 Hz	A  3.46 Hz	S  4.26 Hz	A  5.03 Hz	C  5.76 Hz	S  5.90 Hz	C  6.28 Hz	A  6.76 Hz
	$E_f/E_c=1$, empty dam	A  2.25 Hz	S  2.39 Hz	S  3.28 Hz	A  3.40 Hz	S  4.20 Hz	A  4.97 Hz	C  5.60 Hz	S  5.83 Hz	C  6.11 Hz	A  6.72 Hz
	$E_f/E_c=1$, full dam	A  1.59 Hz	S  1.72 Hz	S  2.24 Hz	A  2.50 Hz	S  3.15 Hz	A  3.82 Hz	C  3.93 Hz	C  4.44 Hz	C  4.49 Hz	S  4.54 Hz

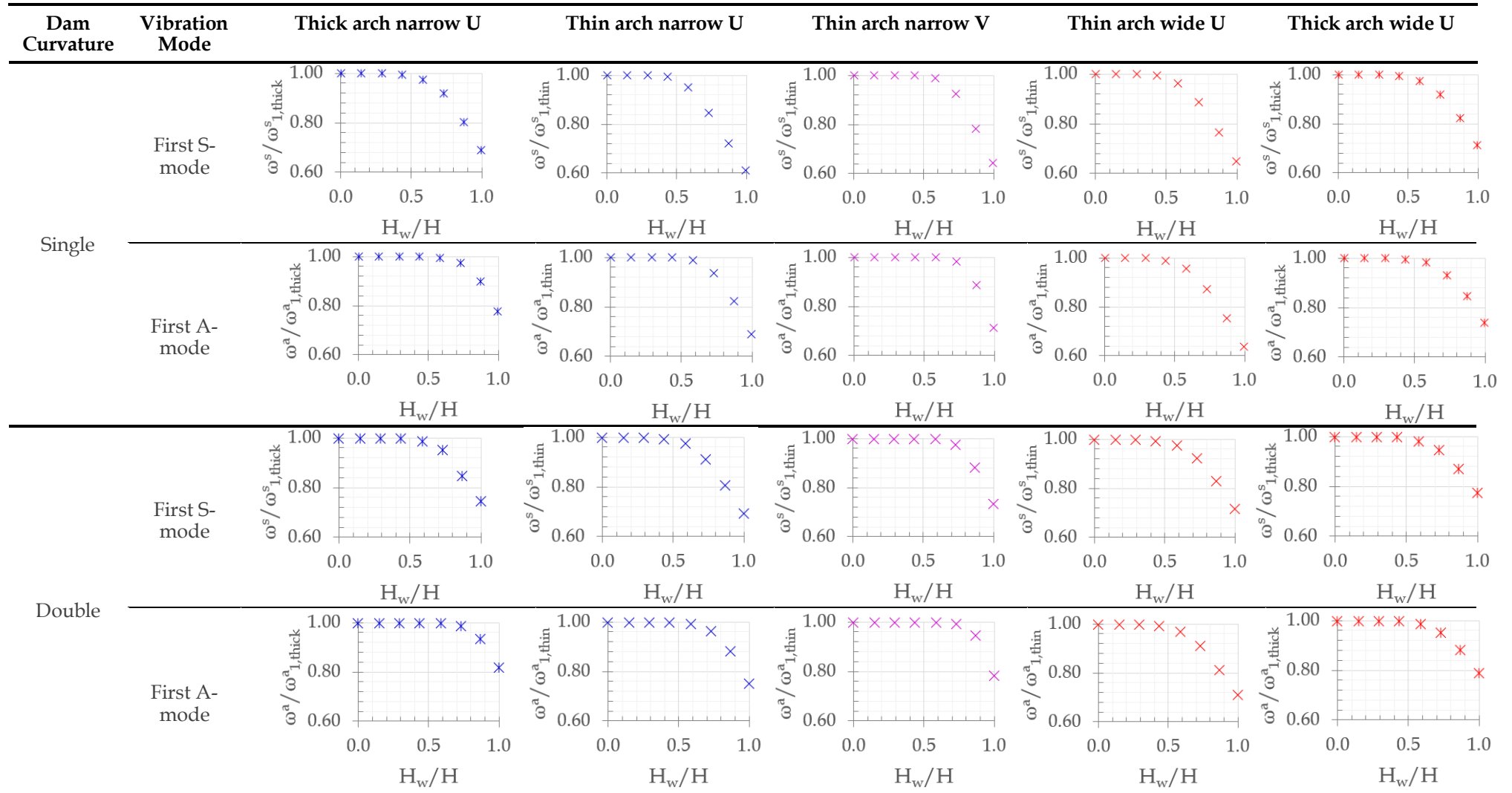
A – antisymmetric mode
C – complex mode
S – symmetric mode

Table 6.10: Thick wide-U shaped dam configuration(s) theoretical vibration modal parameters

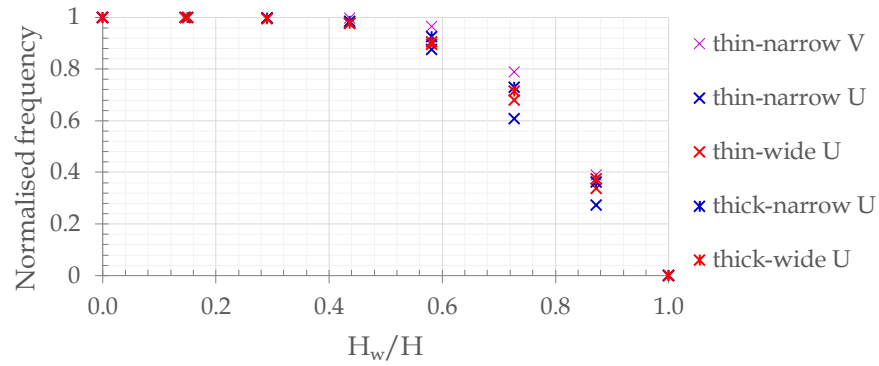
Dam Curvature	Conditions	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Single	Fixed boundary, empty dam	 S 4.48 Hz	 A 4.61 Hz	 S 5.26 Hz	 A 6.13 Hz	 S 7.14 Hz	 A 8.21 Hz	 S 9.36 Hz	 C 10.57 Hz	 A 10.57 Hz	 C 11.11 Hz
	$E_f/E_c=1$, empty dam	 S 4.07 Hz	 A 4.21 Hz	 S 4.96 Hz	 A 5.90 Hz	 S 6.95 Hz	 A 8.06 Hz	 C 9.21 Hz	 S 9.27 Hz	 C 9.53 Hz	 C 10.11 Hz
	$E_f/E_c=1$, full dam	 S 2.90 Hz	 A 3.11 Hz	 S 3.74 Hz	 C 4.41 Hz	 A 4.55 Hz	 C 5.12 Hz	 S 5.45 Hz	 C 5.48 Hz	 C 6.03 Hz	 C 6.41 Hz
Double	Fixed boundary, empty dam	 S 4.55 Hz	 A 4.56 Hz	 S 5.30 Hz	 A 6.19 Hz	 S 7.26 Hz	 A 8.37 Hz	 S 9.52 Hz	 C 9.85 Hz	 C 10.49 Hz	 A 10.75 Hz
	$E_f/E_c=1$, empty dam	 A 4.17 Hz	 S 4.19 Hz	 S 5.01 Hz	 A 5.94 Hz	 S 7.01 Hz	 A 8.11 Hz	 C 8.61 Hz	 S 9.28 Hz	 C 9.30 Hz	 C 10.34 Hz
	$E_f/E_c=1$, full dam	 S 3.23 Hz	 A 3.28 Hz	 S 3.95 Hz	 C 4.41 Hz	 A 4.80 Hz	 C 5.21 Hz	 C 5.47 Hz	 S 5.73 Hz	 C 6.09 Hz	 C 6.61 Hz

A - antisymmetric mode
C - complex mode
S - symmetric mode

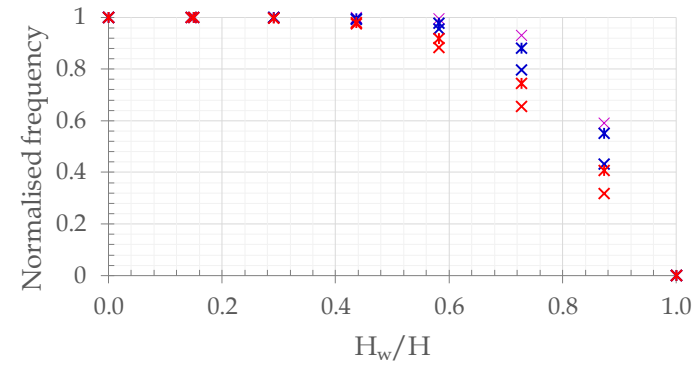
Table 6.11: Effect of water level (added mass) on the dynamic behaviour of selected arch dam configurations



A - antisymmetric mode
S - symmetric mode

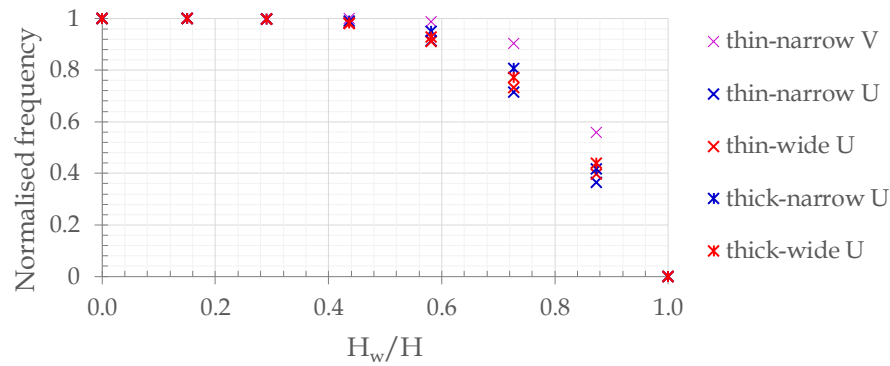


a) First symmetric mode

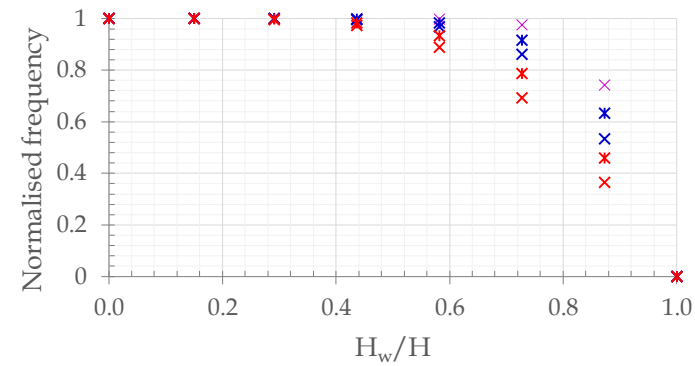


b) First antisymmetric mode

Figure 6.16: Normalised natural frequencies versus water level (added mass) for the selected single curvature arch configurations



a) First symmetric mode



b) First antisymmetric mode

Figure 6.17: Normalised natural frequencies versus water level (added mass) for the selected double curvature arch configurations

6.3.8 Variation of natural frequencies with water level

The relationship(s) between the water level with the first symmetric and antisymmetric modes for the selected arch dam configurations are presented in Table 6.11. Similarly to the hydrostatic load effect, the rate of change in the natural frequencies of the first symmetric and antisymmetric modes decrease as the water level decreases until there is no change in natural frequency below a certain reservoir level depending on the arch dam configuration. The point at which the natural frequency stops changing with decreasing water level for the different profiles due to the added mass effect is influenced by the valley type, vibration mode, arch thickness, and curvature class. Similarly, the total decrease in natural frequencies due to the water level varies depending on the vibration mode (refer to the Appendix for further details), valley type, arch thickness, and arch dam curvature classifications. The rate of change in natural frequency (for both symmetric and antisymmetric modes) generally increases from U- to V- shaped valley types. Whilst the rate of change in natural frequency increases from narrow- classified valleys to wide- classified valleys for the symmetric mode and vice versa (i.e., from wide- classified valley to narrow- classified valley) for the antisymmetric mode for all selected arch dam configurations (Figure 6.16 and Figure 6.17). The difference in the rate of change (i.e., sensitivity) in natural frequency generally decreases as the thickness class increase for the symmetric mode, whilst for the antisymmetric mode the difference is generally unaffected (i.e., remains relatively constant) as the thickness class of the arch increases (Figure 6.16 and Figure 6.17).

6.4 Structural Response Sensitivity to Variation in Material Properties

The sensitivity of the selected responses to variations in E_f/E_c are discussed first followed by the sensitivity to variations in v_f/v_c , ρ_c and α_c . The sensitivity of the selected dam configurations' first two parameters was conducted for a system that included the dam wall and foundation whilst for the last two parameters, a fixed boundary condition at the dam wall- foundation interface was assumed. An empty dam condition assumed throughout for the results presented in this section.

6.4.1 Variation of natural frequencies with system stiffness (E_f/E_c)

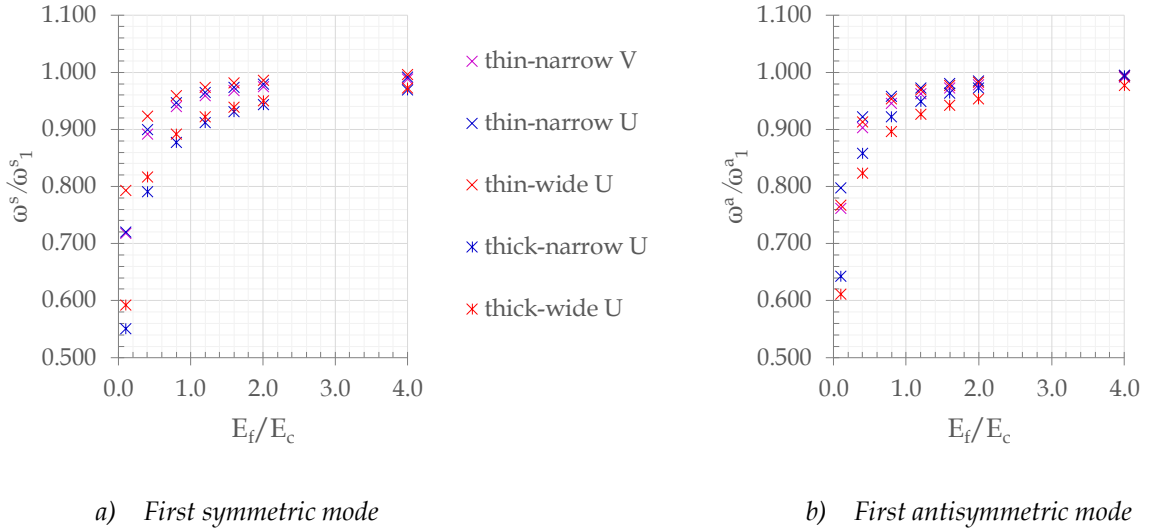


Figure 6.18: ω/ω_1 versus E_f/E_c for the selected single curvature arch configurations

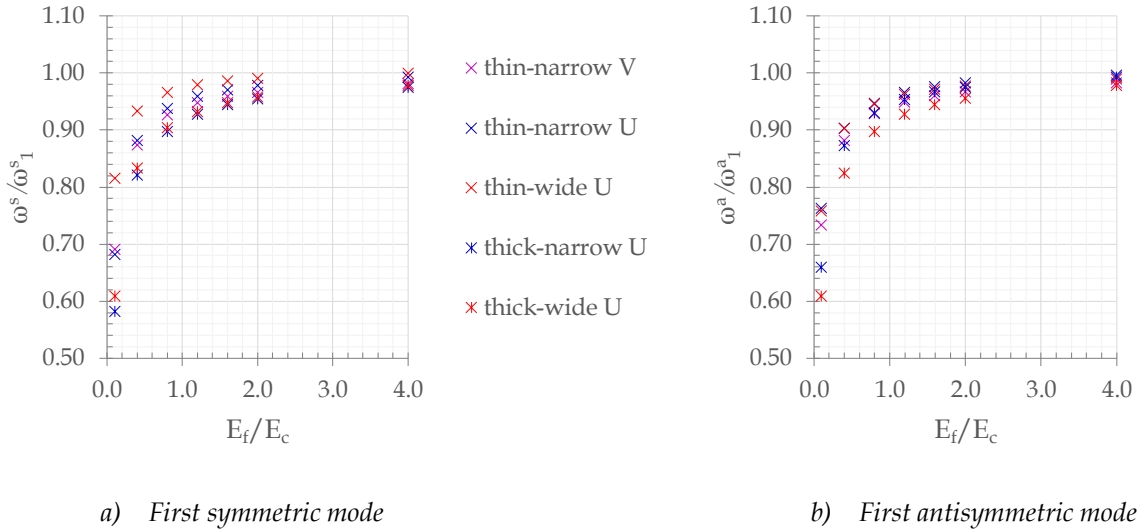


Figure 6.19: ω/ω_1 versus E_f/E_c for the selected double curvature arch configurations

Figure 6.18 and Figure 6.19 show the variation of the natural frequency (expressed as a ratio of the fundamental frequencies) with the system stiffness for the first symmetric and first antisymmetric modes of the selected ten arch configurations. The sensitivity of the natural frequencies to the system stiffness varies depending on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. For all the selected arch dam configurations, the sensitivity of the natural frequencies (both symmetric and antisymmetric modes) to system stiffness increases as stiffness modulus ratio decreases. As the stiffness modulus ratio increases the natural frequency also increase (at a decreasing rate) approaching a constant frequency

value (that corresponds to the rigid foundation case). In the general, as the thickness class increases the overall change in natural frequency increases. Additionally, as the valley class changes from wide to narrow the overall change in natural frequency seems to increase for the first symmetric mode whilst it seems to decrease for the antisymmetric mode.

6.4.2 Variation of displacements with system stiffness (E_f/E_c)

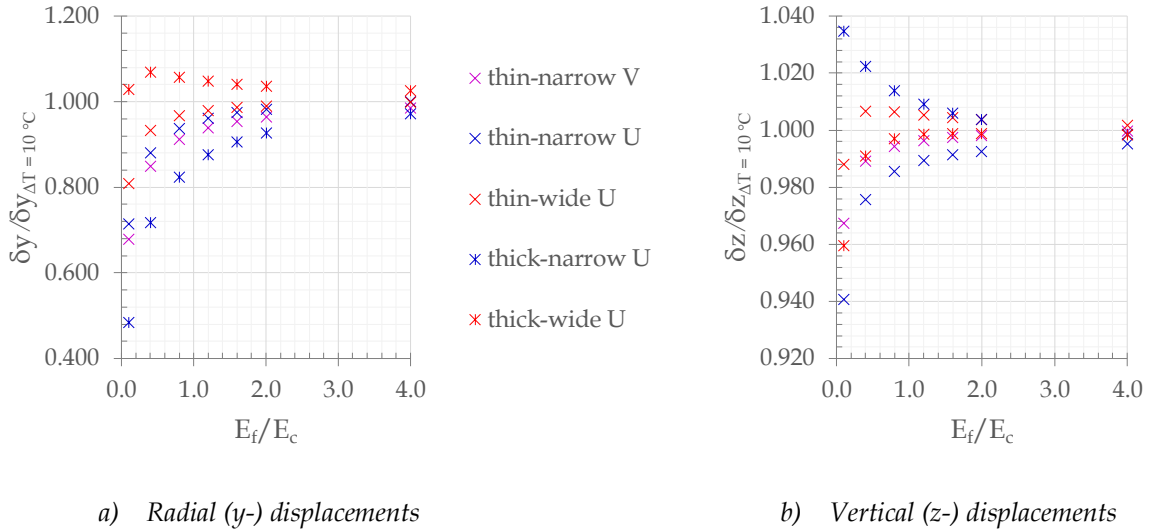


Figure 6.20: $\delta/\delta_{\Delta T=10^\circ\text{C}}$ versus E_f/E_c for selected single curvature arch configurations

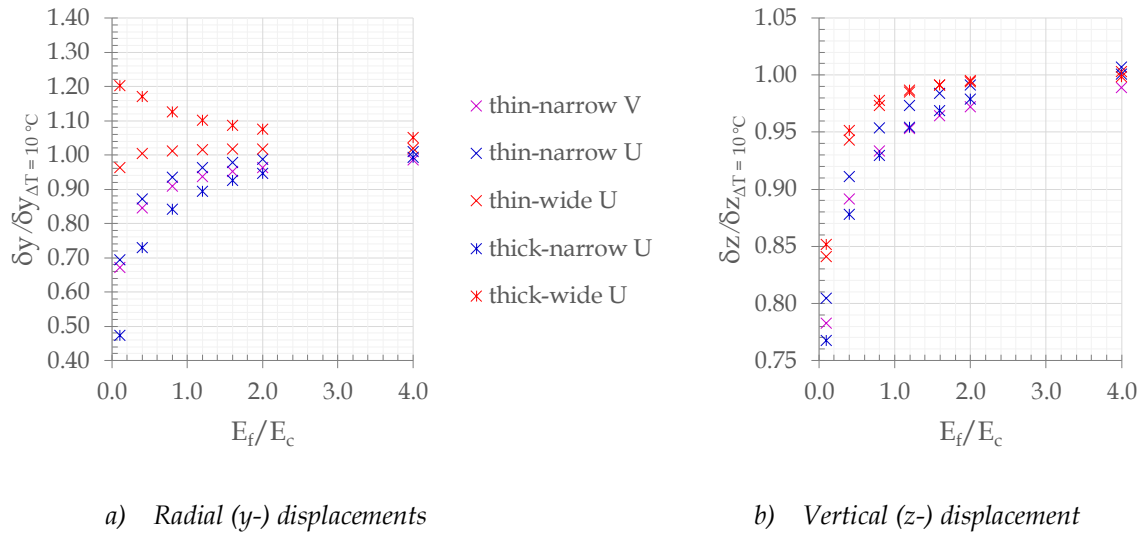


Figure 6.21: $\delta/\delta_{\Delta T=10^\circ\text{C}}$ versus E_f/E_c for selected double curvature arch configurations

Figure 6.20 and Figure 6.21 show the variation of the displacements (both the y- (radial) and z- (vertical) displacements) (expressed as a ratio as defined in section 6.2.4) with the system stiffness for the selected ten arch configurations (for an applied thermal load, $\Delta T = 10^\circ\text{C}$). For all the selected arch dam configurations, as the stiffness ratio increases, the displacement

values (both radial and vertical) approach a constant value. Similarly, to the dynamic response, the sensitivity of the displacements (both radial and vertical displacement ratios) to variation in E_f/E_c increases as the modulus ratio decreases. Additionally, the sensitivity of the displacements to the system stiffness varies depending on the displacement type, valley type, arch thickness, and arch dam curvature classifications. As the modulus ratio increases the displacements increase/decrease (depending on the displacement type, arch thickness class and valley class) approaching a constant value.

6.4.3 Variation of natural frequencies with system v_f/v_c

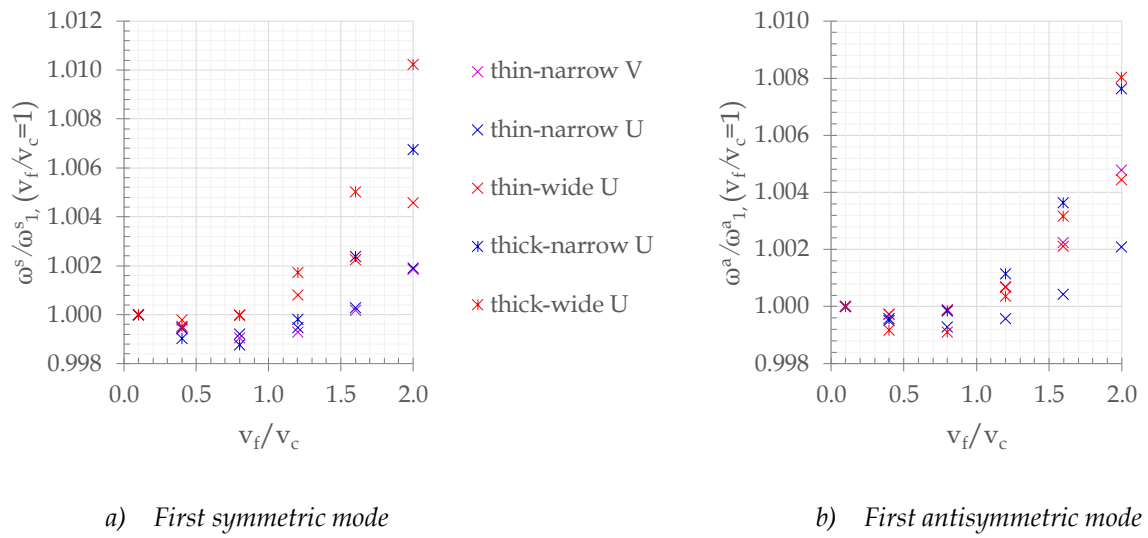


Figure 6.22: ω/ω_1 versus v_f/v_c for selected single curvature arch configurations

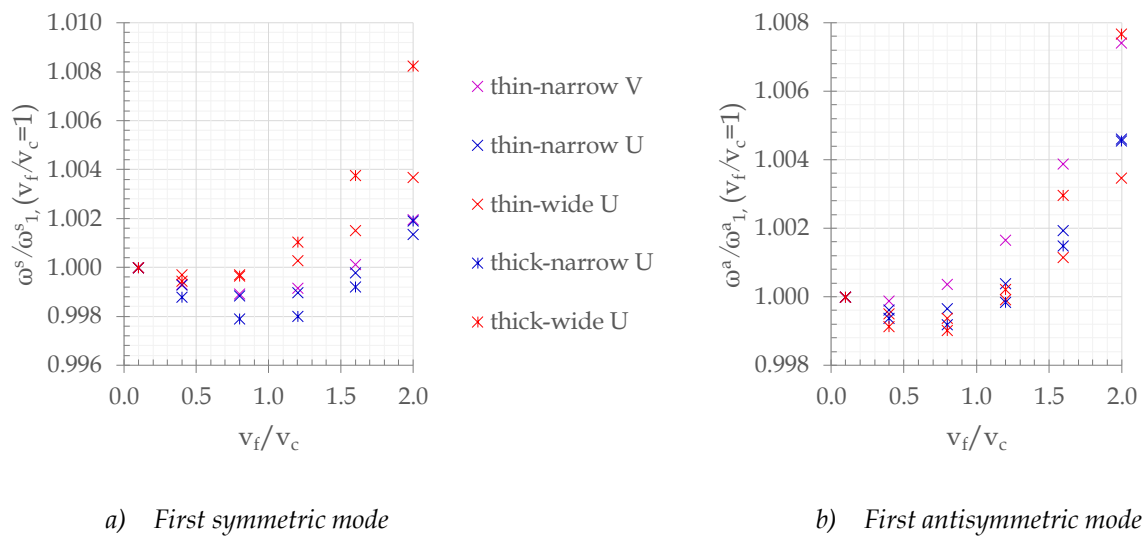
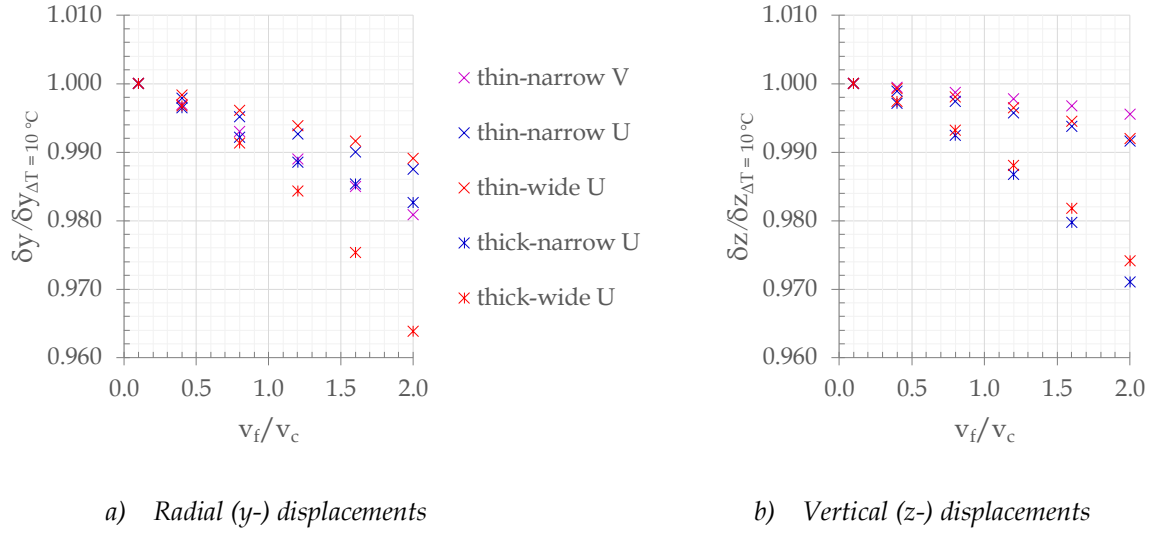
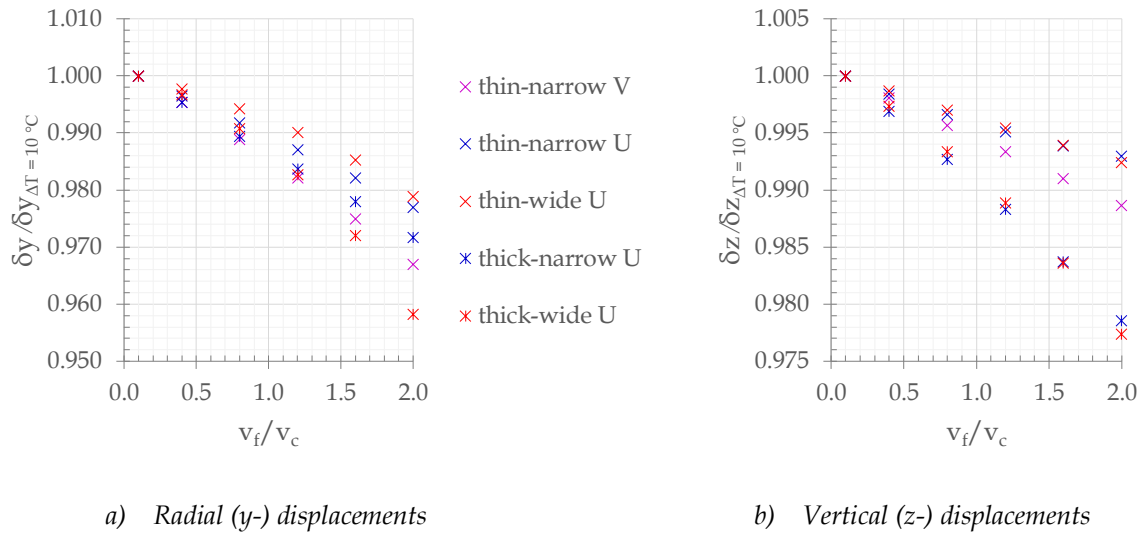


Figure 6.23: ω/ω_1 versus v_f/v_c for selected double curvature arch configurations

Figure 6.22 and Figure 6.23 show the variation of the natural frequencies (expresses as a ratio as detailed in section 6.2.4) with the system's v_f/v_c for the first symmetric and first antisymmetric modes of the selected ten arch configurations. The sensitivity of the natural frequencies to the system's v_f/v_c varies depending on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. Additionally, it was observed that the best-fit line to the frequency ratio vs variation in v_f/v_c data for the selected arch configurations was a parabolic profile (Figure 6.22 and Figure 6.23). Consequently, the rate of change (i.e., sensitivity) in natural frequency initially decreases with the increasing system's v_f/v_c , changing direction between the system's v_f/v_c of 0.5 to 1.0 (depending on the vibration mode, valley type, arch thickness, and arch dam curvature classifications) and increases with increasing v_f/v_c from then on.

6.4.4 Variation of displacements with system v_f/v_c

Figure 6.24 and Figure 6.25 show the variation of the displacements (both the y- (radial) and z- (vertical) displacements) (expresses as a ratio as detailed in section 6.2.4) with the system's v_f/v_c for the selected ten arch configurations (for an applied thermal load, $\Delta T = 10^\circ\text{C}$). The displacements generally decrease as the system's v_f/v_c increases. However, the sensitivity of the displacements to the system's v_f/v_c varies depending on the displacement type, valley type, arch thickness, and arch dam curvature classifications. Additionally, the rate of change of the displacements is not constant but varies slightly for each arch dam configuration. The thick wide U configuration consistently has the largest overall decrease (across the investigated v_f/v_c range) in displacements across all the investigated configurations and displacement types. Whilst the rest of the other configurations vary depending on the arch configuration.


 Figure 6.24: $\delta/\delta_{\Delta T=10^\circ C}$ versus v_f/v_c for selected single curvature arch configurations

 Figure 6.25: $\delta/\delta_{\Delta T=10^\circ C}$ versus v_f/v_c for selected double curvature arch configurations

6.4.5 Variation of natural frequencies with concrete density ρ_c

Figure 6.26 shows the typical variation of the natural frequency (presented as a ratio relative to the fundamental frequencies determined at $\rho_c = 2400 \text{ kg/m}^3$) with concrete density (ρ_c) which was the same (with a sensitivity ($[\text{Hz}/(\text{kg/m}^3)] \%$) of approximately 0.02% on average) for all the investigated configurations irrespective of the vibration mode and the geometric characteristics of the arch structure (valley type, arch thickness and arch curvature). The rate of change (i.e., sensitivity) in natural frequencies with increasing density decreases slightly.

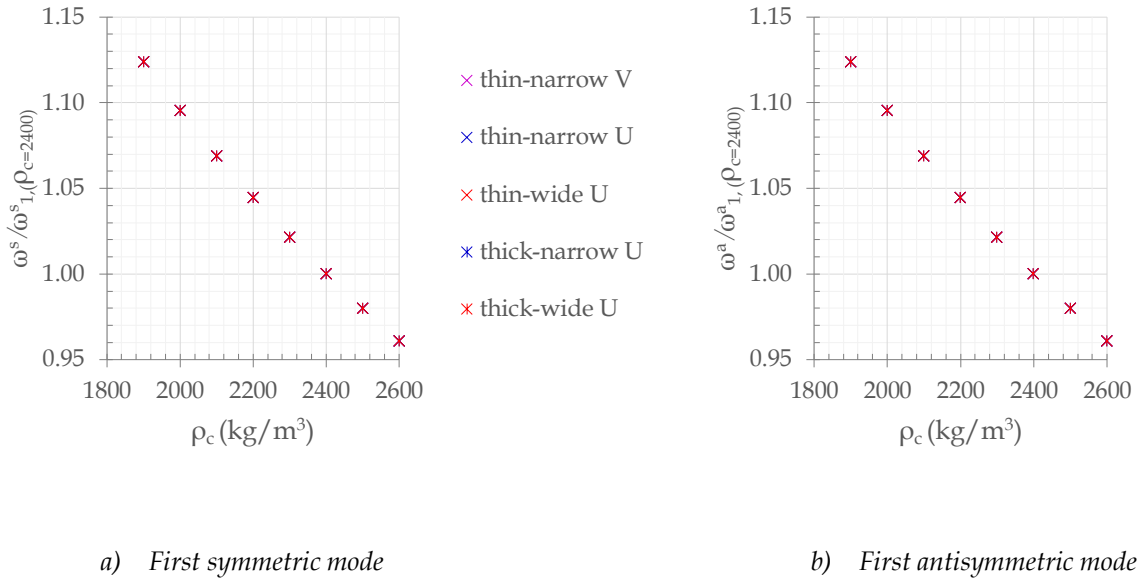


Figure 6.26: Typical ω/ω_1 versus ρ_c for the selected arch dam configurations

6.4.6 Variation of displacements with concrete density ρ_c

Figure 6.26 shows the typical variation of displacements (both the radial and vertical) with concrete density (ρ_c). There is no correlation between displacements and the variation in concrete density for all arch dam configurations.

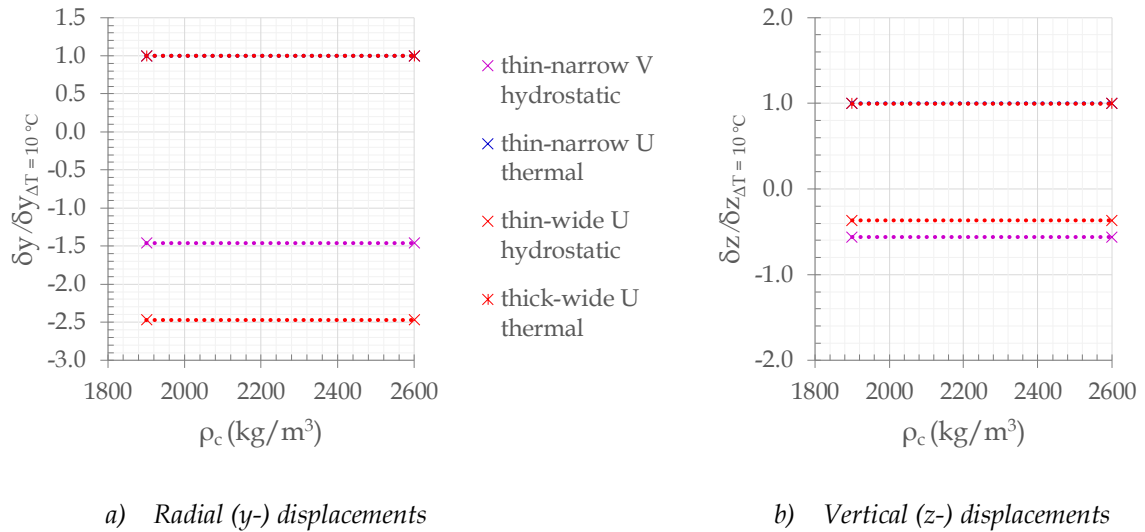


Figure 6.27: Typical $\delta/\delta_{\Delta T=10^\circ\text{C}}$ versus ρ_c for the selected arch dam configurations

6.4.7 Variation of natural frequencies with the coefficient of thermal expansion (α_c)

Figure 6.28 and Figure 6.29 show the variation of the natural frequencies with the coefficient of thermal expansion (α_c) for the first symmetric and first antisymmetric modes of the selected

ten arch configurations. A thermal load, $\Delta T = 10^\circ\text{C}$, was applied in the analysis of the results presented in this section and the natural frequencies are presented as a ratio relative to the fundamental frequencies corresponding to $\alpha_c = 10 \times 10^{-6} \text{ }^\circ\text{C}^{-1}$. The sensitivity of the natural frequencies to the coefficient of thermal expansion (α_c) varies depending on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. Similarly to the variation in natural frequencies to thermal loading, the natural frequencies are only sensitive to the 4 decimal place with variation the coefficient of thermal expansion. The individual sensitivities are summarised in section 6.5.

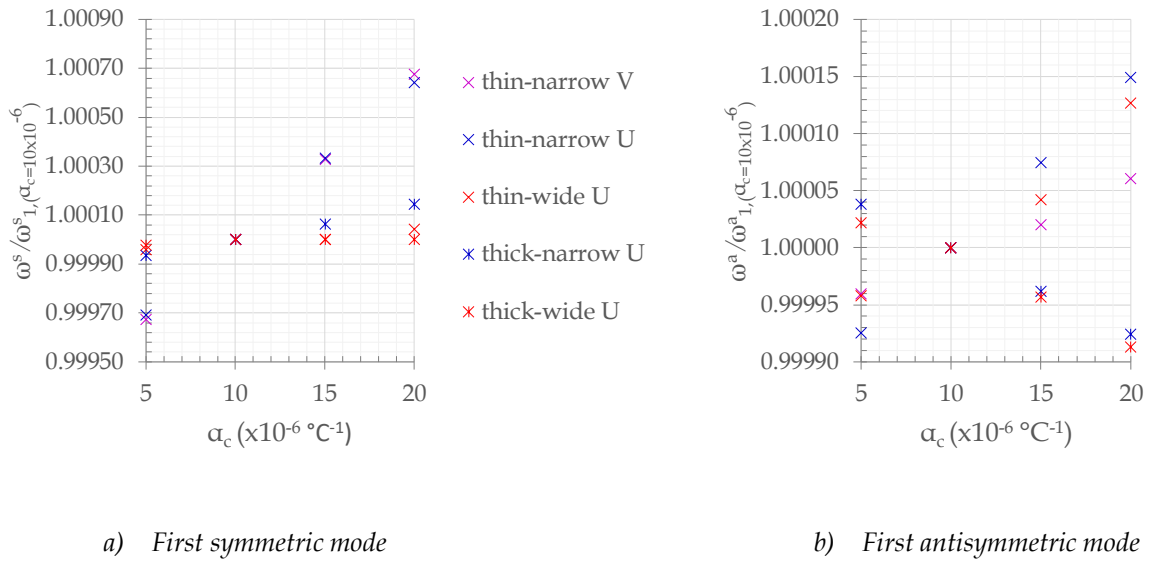


Figure 6.28: ω/ω_1 versus α_c for selected single curvature arch configurations

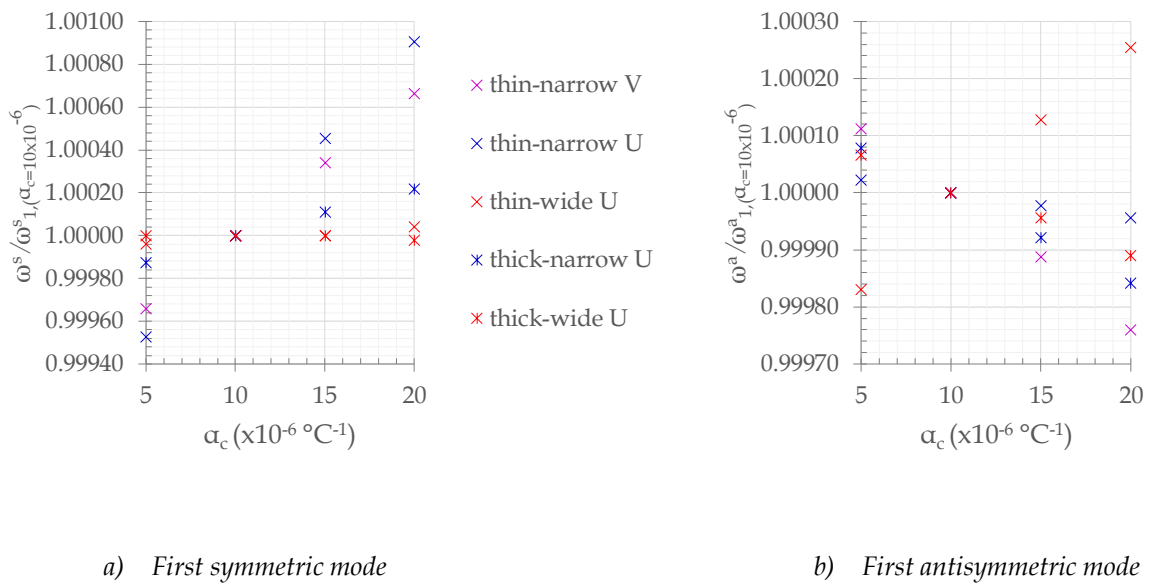


Figure 6.29: ω/ω_1 versus α_c for selected double curvature arch configurations

6.4.8 Variation of displacements with the coefficient of thermal expansion (α_c)

Figure 6.30 shows the typical variation of the displacements (both radial and vertical displacements presented as ratios relative to the displacements determined using $\alpha_c = 10 \times 10^{-6} \text{ }^\circ\text{C}^{-1}$ for an applied thermal load, $\Delta T = 10 \text{ }^\circ\text{C}$,) with the coefficient of thermal expansion (α_c) which was the same (with a sensitivity ($[\text{mm}/(\alpha_c \times 10^{-6} \text{ }^\circ\text{C}^{-1})]\%$) of approximately 10%) for all the investigated configurations irrespective of the displacement type and the arch dam configuration characteristics of the arch structure.

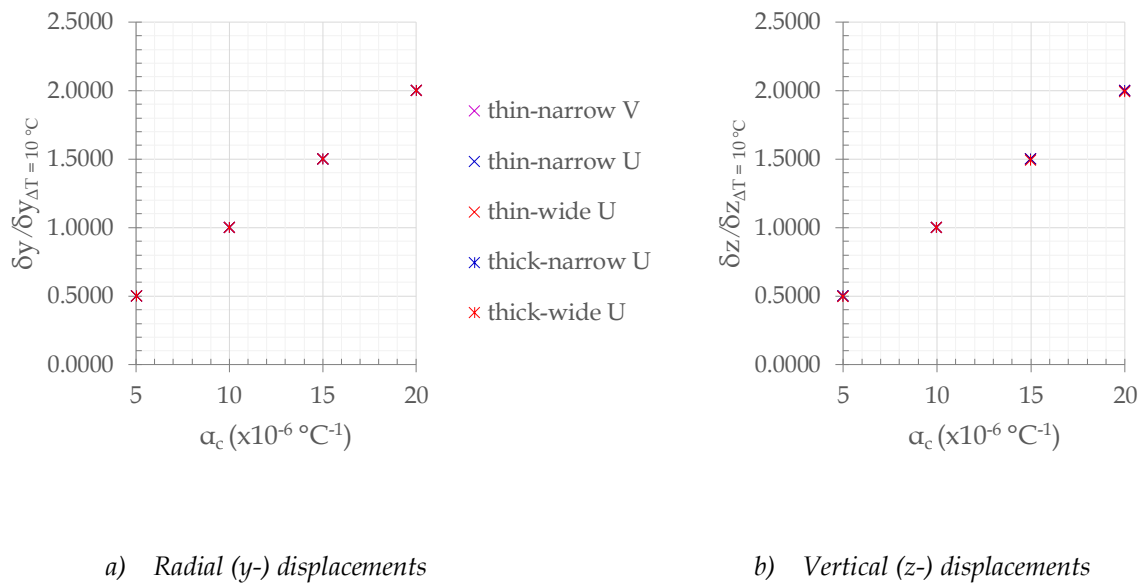


Figure 6.30: Typical $\delta/\delta_{\Delta T=10^\circ\text{C}}$ versus α_c for the selected arch configurations

6.5 Chapter Summary

This chapter presented the sensitivity study of the structural behaviour of various arch dam configurations to load effects and variation in material properties with the aim of arriving at a selection of parameters to update with respect to the structural behaviour of operational concrete arch dams. Table 6.12 Table 6.13 provide a summary of the dynamic response sensitivities in which Hz is used to indicate frequency although the natural frequency results were presented as ratios in this Chapter, whilst Table 6.14 Table 6.15 provide a summary of the static response sensitivities in which mm is used to indicate displacements although the displacement results were presented as ratios in this Chapter. The results in Table 6.12-6.15 are presented as sensitivity values (as a percentage) for the variation due to the thermal load, ASR, concrete density, and the coefficient of thermal expansion. Whilst for the variation of the structural behaviour due to the hydrostatic load, water level, system stiffness and Poisson's

ratio, the total change (as a percentage) over the investigated ranges is presented, since the sensitivities vary (i.e. are not constant) over the range for these variables.

For the dynamic behaviour response irrespective of the mode, arch dam thickness classification, curvature or site type, the added water and system stiffness have the highest influence on the arch dam's response. Of particular interest is the sensitivity due to the concrete density. There is a strong correlation between the dynamic properties of concrete arch dams and the density of concrete. However, the sensitivity value at face value seems low, however, considering that density of concrete is normally reported and/or measured typically to nearest 50 kg/m³, multiplying the sensitivity with typical density variation may result in significant changes to the modal parameters of a structure. ASR also does result in significant variation of the dynamic properties of concrete arch dams. Evidently, the dynamic behaviour of the investigated arch dams is not as sensitive to stress stiffening due to ASR related expansion as was initially assumed. There is no strong influence of the rest of the investigated parameters on the dynamic response of the investigated arch dams.

On the other hand, the static behaviour of the investigated arch dam configurations is influenced by virtually all the investigated parameters except for concrete density. With respect to load effects, the sensitivity of the static behaviour is highest for ASR related effects, followed by ambient temperature and hydrostatic load effect. Additionally, the system stiffness, coefficient of expansion and Poisson's ratio, influence the static behaviour of arch dams.

Consequently, the parameters to update with respect to the static behaviour may include the coefficient of thermal expansion, system stiffness ratio and Poisson's system ratio without neglecting the influence of temperature, ASR, and hydrostatic load effects. Whilst the parameters to update with respect to the dynamic behaviour of concrete arch dams may include the system stiffness ratio and concrete density. Based on this observation the updating parameters for the proposed finite element model updating approach were selected and implemented in the following Chapter.

Table 6.12: Summary of sensitivities of the 1st symmetric mode natural frequency to variations in the investigated variables

Arch curvature	Investigated variable	Presented as sensitivity or range	Thick Arch Narrow U 	Thin Arch Narrow U 	Thin Arch Narrow V 	Thin Arch Wide U 	Thick Arch Wide U 
Single	Thermal load	[Hz/°C] %	0.001	0.006	0.007	0.0004	0.0001
	ASR load	[Hz/(ϵ_v %)] %	0.61	2.48	2.70	0.13	0.04
	Hydrostatic load	[$\Delta\omega_{Total}$] %	-0.03	-0.35	-0.27	-0.64	-0.01
	Water load	[$\Delta\omega_{Total}$] %	-31	-39	-36	-35	-29
	System Stiffness	[$\Delta\omega_{Total}$] %	41.9	27.2	27.0	20.3	38.0
	System Poisson's Ratio	[$\Delta\omega_{Total}$] %	0.80	0.27	0.28	0.48	1.07
	Wall Density	[Hz/(kg/m ³)] %	0.02	0.02	0.02	0.02	0.02
	Wall Coefficient of thermal expansion	[Hz/($\alpha_c (\times 10^{-6}) ^\circ\text{C}^{-1}$)] %	0.001	0.006	0.007	0.0005	0.0001
Double	Thermal load	[Hz/°C] %	0.002	0.009	0.007	0.0005	-0.0002
	ASR load	[Hz/(ϵ_v %)] %	1.04	3.59	2.92	0.19	-0.10
	Hydrostatic load	[$\Delta\omega_{Total}$] %	-0.03	-0.20	-0.14	-0.46	-0.02
	Water load	[$\Delta\omega_{Total}$] %	-26	-31	-27	-28	-23
	System Stiffness	[$\Delta\omega_{Total}$] %	39.3	31.1	29.0	18.4	36.9
	System Poisson's Ratio	[$\Delta\omega_{Total}$] %	0.40	0.25	0.30	0.40	0.88
	Wall Density	[Hz/(kg/m ³)] %	0.02	0.02	0.02	0.02	0.02
	Wall Coefficient of thermal expansion	[Hz/($\alpha_c (\times 10^{-6}) ^\circ\text{C}^{-1}$)] %	0.002	0.009	0.007	0.0005	-0.0001

Table 6.13: Summary of sensitivities of the 1st antisymmetric mode natural frequency to variations in the investigated variables

Arch curvature	Investigated variable	Presented as sensitivity or range	Thick Arch Narrow U 	Thin Arch Narrow U 	Thin Arch Narrow V 	Thin Arch Wide U 	Thick Arch Wide U 
Single	Thermal load	[Hz/°C] %	-0.0008	0.001	0.0006	0.001	-0.0008
	ASR load	[Hz/(ϵ_v %)] %	-0.35	0.55	0.23	0.39	-0.31
	Hydrostatic load	[$\Delta\omega_{Total}$] %	-0.02	-0.28	-0.18	-0.62	-0.03
	Water load	[$\Delta\omega_{Total}$] %	-22	-31	-28	-36	-26
	System Stiffness	[$\Delta\omega_{Total}$] %	35.1	19.8	22.9	22.6	36.5
	System Poisson's Ratio	[$\Delta\omega_{Total}$] %	0.80	0.28	0.51	0.47	0.89
	Wall Density	[Hz/(kg/m ³)] %	0.02	0.02	0.02	0.02	0.02
	Wall Coefficient of thermal expansion	[Hz/($\alpha_c (\times 10^{-6}) ^\circ\text{C}^{-1}$)] %	-0.0008	0.001	0.0006	0.001	-0.0007
Double	Thermal load	[Hz/°C] %	-0.002	-0.0004	-0.002	0.003	-0.001
	ASR load	[Hz/(ϵ_v %)] %	-0.64	-0.17	-0.94	1.07	-0.45
	Hydrostatic load	[$\Delta\omega_{Total}$] %	-0.02	-0.16	-0.08	-0.42	-0.03
	Water load	[$\Delta\omega_{Total}$] %	-18	-25	-22	-29	-21
	System Stiffness	[$\Delta\omega_{Total}$] %	33.3	23.4	24.9	23.1	36.9
	System Poisson's Ratio	[$\Delta\omega_{Total}$] %	0.54	0.50	0.75	0.41	0.87
	Wall Density	[Hz/(kg/m ³)] %	0.02	0.02	0.02	0.02	0.02
	Wall Coefficient of thermal expansion	[Hz/($\alpha_c (\times 10^{-6}) ^\circ\text{C}^{-1}$)] %	-0.002	-0.0004	-0.002	0.003	-0.001

Table 6.14: Summary of sensitivities of the radial (y-) displacements to variations in the investigated variables

Arch curvature	Investigated variable	Presented as sensitivity or range	Thick Arch Narrow U 	Thin Arch Narrow U 	Thin Arch Narrow V 	Thin Arch Wide U 	Thick Arch Wide U 
Single	Thermal load	[mm/°C] %	10	10	10	10	10
	ASR load	[mm/(ϵ_v %)] %	4422.8	3971.1	4044.5	3866.8	4176.1
	Hydrostatic load	[$\Delta\delta_{Total}$] %	-107	-228	-205	-251	-140
	Water load	[$\Delta\delta_{Total}$] %	n/a	n/a	n/a	n/a	n/a
	System Stiffness	[$\Delta\delta_{Total}$] %	49	29	31	19	-3.2
	System Poisson's Ratio	[$\Delta\delta_{Total}$] %	-1.7	-1.3	-1.9	-1.1	-3.6
	Wall Density	[mm/(kg/m ³)] %	0	0	0	0	0
	Wall Coefficient of thermal expansion	[mm/(α_c (x10 ⁻⁶)°C ⁻¹)] %	10	10	10	10	10
Double	Thermal load	[mm/°C] %	10	10	10	10	10
	ASR load	[mm/(ϵ_v %)] %	4380.2	3876.4	4212.6	3764.9	4131.8
	Hydrostatic load	[$\Delta\delta_{Total}$] %	-103	-161	-146	-247	-147
	Water load	[$\Delta\delta_{Total}$] %	n/a	n/a	n/a	n/a	n/a
	System Stiffness	[$\Delta\delta_{Total}$] %	52	31	31	5.6	-15
	System Poisson's Ratio	[$\Delta\delta_{Total}$] %	-2.8	-2.3	-3.3	-2.1	-4.2
	Wall Density	[mm/(kg/m ³)] %	0	0	0	0	0
	Wall Coefficient of thermal expansion	[mm/(α_c (x10 ⁻⁶)°C ⁻¹)] %	10	10	10	10	10

Table 6.15: Summary of sensitivities of the vertical (z-) displacements to variations in the investigated variables

Arch curvature	Investigated variable	Presented as sensitivity or range	Thick Arch Narrow U 	Thin Arch Narrow U 	Thin Arch Narrow V 	Thin Arch Wide U 	Thick Arch Wide U 
Single	Thermal	[mm/°C] %	10	10	10	10	10
	ASR	[mm/(ϵ_v %)] %	4399.2	3991.6	4048.7	3830.7	4138.9
	Hydrostatic	[$\Delta\delta_{\text{Total}}$] %	40	66	61	85	62
	Water load	[$\Delta\delta_{\text{Total}}$] %	n/a	n/a	n/a	n/a	n/a
	System Stiffness	[$\Delta\delta_{\text{Total}}$] %	-3.6	5.5	3.2	-0.5	3.9
	System Poisson's Ratio	[$\Delta\delta_{\text{Total}}$] %	-2.9	-0.8	-0.4	-0.8	-2.6
	Wall Density	[mm/(kg/m ³)] %	0	0	0	0	0
	Wall Coefficient of thermal expansion	[mm/(α_c (x10 ⁻⁶)°C ⁻¹)] %	10	10	10	10	10
Double	Thermal	[mm/°C] %	10	10	10	10	10
	ASR	[mm/(ϵ_v %)] %	4350.9	3866.2	4172.1	3742.8	4087.7
	Hydrostatic	[$\Delta\delta_{\text{Total}}$] %	-24	-44	-56	-54	-36
	Water load	[$\Delta\delta_{\text{Total}}$] %	n/a	n/a	n/a	n/a	n/a
	System Stiffness	[$\Delta\delta_{\text{Total}}$] %	23	20	21	16	15
	System Poisson's Ratio	[$\Delta\delta_{\text{Total}}$] %	-2.1	-0.7	-1.1	-0.8	-2.3
	Wall Density	[mm/(kg/m ³)] %	0	0	0	0	0
	Wall Coefficient of thermal expansion	[mm/(α_c (x10 ⁻⁶)°C ⁻¹)] %	10	10	10	10	10

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Chapter 7

MULTI-STEP FINITE ELEMENT MODEL UPDATING PROCEDURE FOR CONCRETE ARCH DAMS

7.1 Introduction

The structural behaviour of arch dams is complex and is influenced by a number of dam-reservoir-foundation system properties (including material properties, geometric properties, boundary conditions and loading conditions) requiring FE models to evaluate their structural response (refer to Chapter 5 and Chapter 6)). In their evaluation, there are always numerous unknown or uncertain system properties due to a lack of up-to-date information and/or other limitations. Model updating (also referred to as model calibration or parameter identification/estimation in generic terms), whose implementation in FE models has been discussed in Chapter 3, is used to identify unknown and/or minimise the uncertainty of these system properties (which appear as parameters in FE models) to achieve reliable and accurate FE models using monitoring data (Simoen et.al., 2014). It goes without saying that up-to-date measurement data of these system properties (where possible) eliminates any uncertainty associated with them, which, effectively reduces the number of unknown and/or uncertain parameters to be considered in the model updating process.

To meet the objectives of this study (particularly objective (iv) of this study (refer to section 1.3), a multi-step model updating approach for the assessment of concrete arch dams, including those affected by ASR is proposed and then implemented on a case study dam affected by ASR. The aim is to provide engineers and practitioners with an updating framework that can be easily implemented, and that incorporates ambient vibration measurements to achieve accurate and reliable FE models for the assessment of these structures. Fundamental to the successful implementation of the proposed updating approach is the availability of monitoring data relevant to each model-updating step. The chapter begins by discussing the proposed model updating approach and its implementation in FE models, followed by a discussion on the implementation of the approach on the case study arch dam.

7.2 Vibro-Thermo-Deformational Finite Element Model Updating Procedure

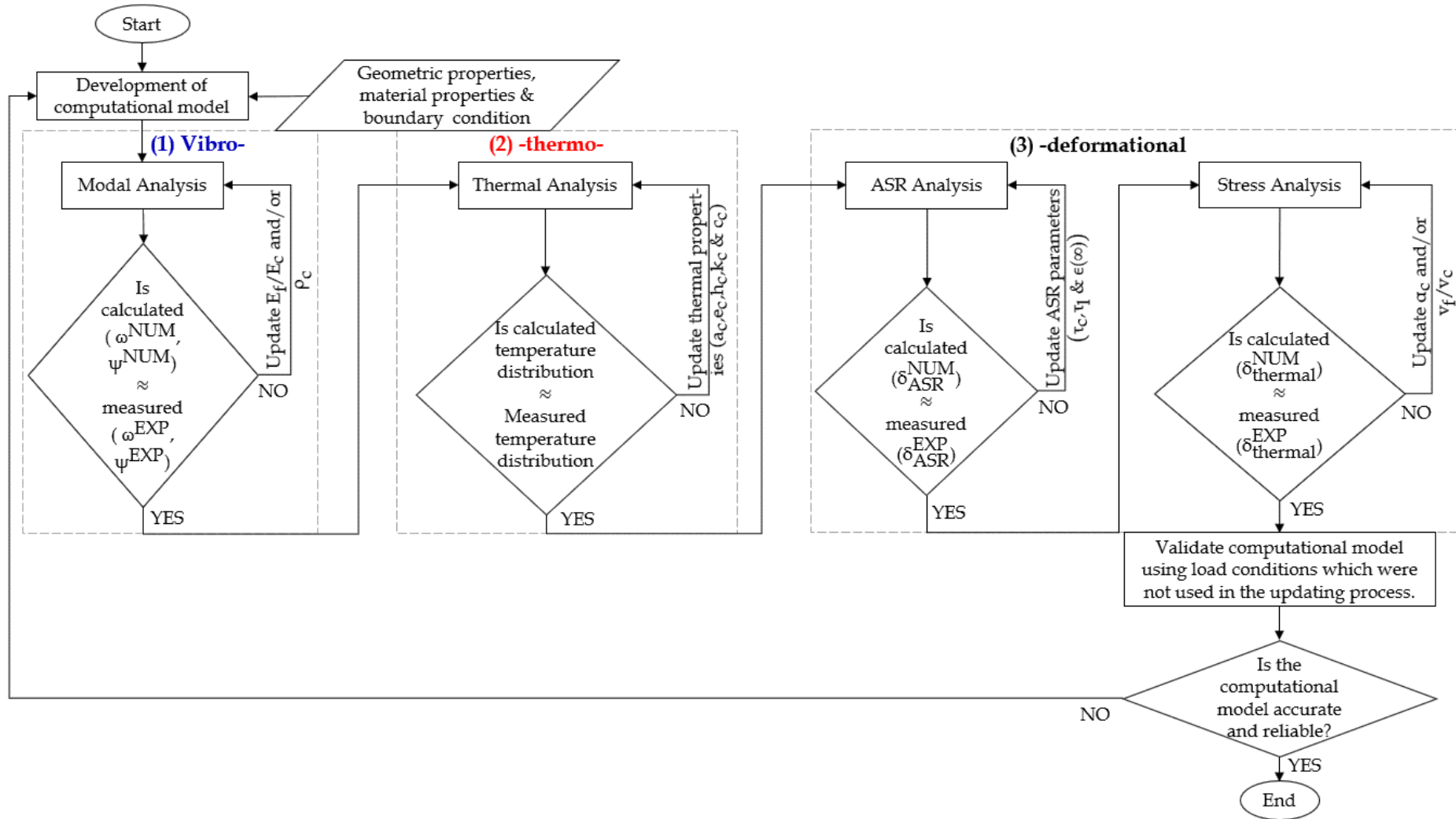


Figure 7.1: Multi-step model updating approach of concrete arch dams including those affected by ASR

Figure 7.1 summarises the proposed model updating procedure that begins with modal updating followed by thermal parameter updating and then deformation updating which has two components, namely: ASR related parameter updating and thermal stress deformation updating. Each of these steps will be discussed in detail in the following sections:

7.2.1 Vibration based (Vibro-) model updating

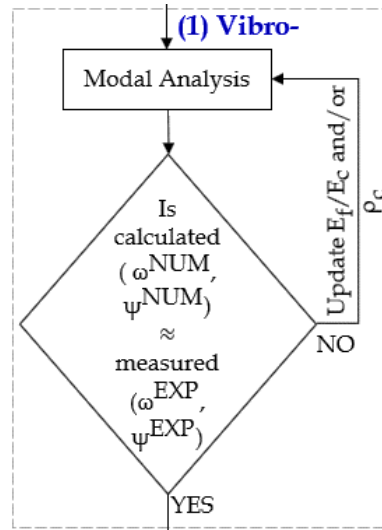


Figure 7.2: First step in the multi-step model updating approach

The first step in the proposed model updating procedure is the vibration-based model updating step. It is well understood that modal parameters (natural frequencies, mode shapes and damping ratios) can be related to structural parameters (structural stiffness, mass, and damping) of a structural system (Haidarpour & Tee, 2020). Measured modal parameters are therefore used to update the system mass and stiffness. Additionally, the mode shapes provide useful insight on the system stiffness distribution, which is useful in the updating of boundary conditions in cases where there is variation in the boundary stiffness values.

Why modal updating first

The dynamic response of concrete arch dams is sensitive to geometric properties, system stiffness and system mass (i.e., added mass due to water and concrete density) (Refer to Chapter 6). If any of these parameters are incorrect, the solution to the updating problem is unlikely to be satisfactory achieved unless some parameters are changed beyond the point which, the parameters lose physical meaning. Modal parameter updating, if done correctly, ensures primarily that the model system is a correct representation of the physical real system. Additionally, the dynamic response of the investigated arch dams is not as sensitive to any

other load conditions investigated in Chapter 6 (which constitutes the load conditions experienced by most arch dams affected by ASR in regions with high seasonal temperature variation besides earthquake loading). Thus, no other loading besides the water level (where relevant) needs to be applied to obtain accurate modal parameters from an FE model.

Vibration based model-updating methodology

This step involves the comparison of the measured modal parameters with the computed FE model modal parameters. The objective is to minimise the difference between the natural frequency values and the variation in the mode shapes, a category of model updating approach referred to as the modal property difference formulation. Under this formulation, the eigenvector difference approach (Otsuki et al., 2022) using the multi objective approach (discussed in section 3.2.2) was implemented in this study as follows:

$$\text{minimise } \sum_{i=1}^{n_{\text{modes}}} \left(\frac{\omega_i^{\text{EXP}} - \omega_i^{\text{NUM}}(\theta)}{\omega_i^{\text{EXP}}} \cdot w_{\omega_i} \right)^2 \quad (7.1)$$

$$\text{minimise } \sum_{i=1}^{n_{\text{modes}}} \left\| \left\{ \psi_{-q_i i}^{\text{EXP}, M} - \psi_{-q_i i}^{\text{NUM}, M}(\theta) \right\} \cdot w_{\psi_i} \right\|_2^2 \quad (7.2)$$

where θ is a column vector of p updating parameters, ω_i and ψ_i are the i^{th} eigen-frequency and eigenvector respectively, w_{ω_i} and w_{ψ_i} represent the weighting factor of the i^{th} eigenvalue difference and eigenvector difference respectively, superscript ^{EXP} and ^{NUM} denote experimentally obtained and numerically obtained respectively, whilst the superscript ^M denotes vector entries corresponding to the measured degree-of-freedom (DOF), whereas the subscript _{- q_i} denotes that the q_i^{th} entry is removed (Otsuki et al., 2022).

As can be alluded from equation (7.1) and equation (7.2), the model outputs are functions of p updating parameters. Each updating parameter is subject to a lower and upper bound of the variable (i.e., the minimum and maximum value the parameter can have without losing physical meaning). The measurement derived eigenvector at the measured DOFs, $\psi_i^{\text{EXP}, M}$, is normalised such that its largest entry, denoted as the q_i^{th} entry, equals to 1. Similarly, the FE model computed eigenvector at the measured DOFs, $\psi_i^{\text{NUM}, M}$, is normalised so that the q_i^{th} entry equals to 1. $\psi_{-q_i i}^{\text{EXP}, M}$ and $\psi_{-q_i i}^{\text{NUM}, M}$ represent the experimentally obtained and numerically determined eigenvectors respectively at the measured DOFs with the q_i^{th} entry removed (Otsuki et al., 2022).

Updating parameters

Assuming that the geometrical properties of the dam have been captured accurately, the parameters to update are the system stiffness and mass parameters. Normally for dam structures, modal parameters are specified at a particular water level as the added mass effect of the water level is well understood. Thus, the water level in most cases is not an unknown. As a result, the parameters to update are E_f/E_c and ρ_c .

In Chapter 6 it was observed that the dynamic behaviour of concrete arch dams is sensitive to ρ_c whereas its static response is not affected by any change in ρ_c . Whilst both the static and dynamic response of the arch dam system is sensitive to E_f/E_c . However, the static response is also sensitive to other load and material properties. Thus updating E_f/E_c using the dynamic response ensures the likely E_f/E_c is obtained as the influence of other parameters does not significantly influence the modal parameters of the system.

7.2.2 Temperature distribution based (-thermo-) model updating

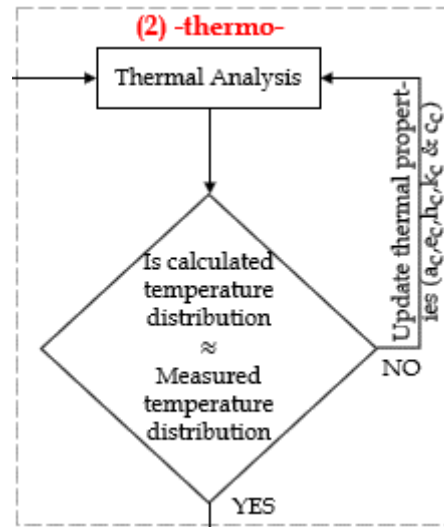


Figure 7.3: Second step in the multi-step model updating approach

In regions with high seasonal temperature variations, thermal stresses dominate the seasonal stress variation in concrete arch dams (Moyo et al., 2019). In addition, the concrete temperature distribution influences other mechanisms such as creep and the reaction kinetics for dams affected by ASR (section 4.5). Heat analysis (whose output is the temperature distribution) is performed via three main temperature mechanisms namely: (i) conduction, (ii) convection, and (iii) radiation (discussed in detail in section 4.5), which are dependent on the thermal properties of concrete such as specific heat (c_c), and geometrical properties such as the dam

thickness (Meri, 2011). However, by the end of the vibration based updating step (section 7.2.1), the geometric properties are confirmed and do not need further updating. In this step, the difference between the computed temperature distribution (as a function of time) and the measured temperature distribution (as a function of time) of the concrete dam are minimised by updating the thermal material properties (assuming the temperature load is applied correctly). For a detailed discussion on heat transfer analysis of concrete arch dams the reader is directed to Nzuza (2013), Agullo et al. (1996) and, Bafong (1997) among others, as heat transfer mechanisms and the resulting concrete temperature distribution did not fall part of the scope of this research and were thus, not investigated in this study. However, the assumption in this study is that a proper temperature analysis of the dam is conducted before the applications of the ASR constitutive model and the thermal stress analysis as part of the comprehensive assessment of concrete arch dams (Chapter 4). This section has been included for completeness of the discussion on the proposed multi-step approach, as this step is a vital component of the proposed approach since deformation based updating steps are dependent and follow on from this step.

7.2.3 Deformation based (-deformational) model updating

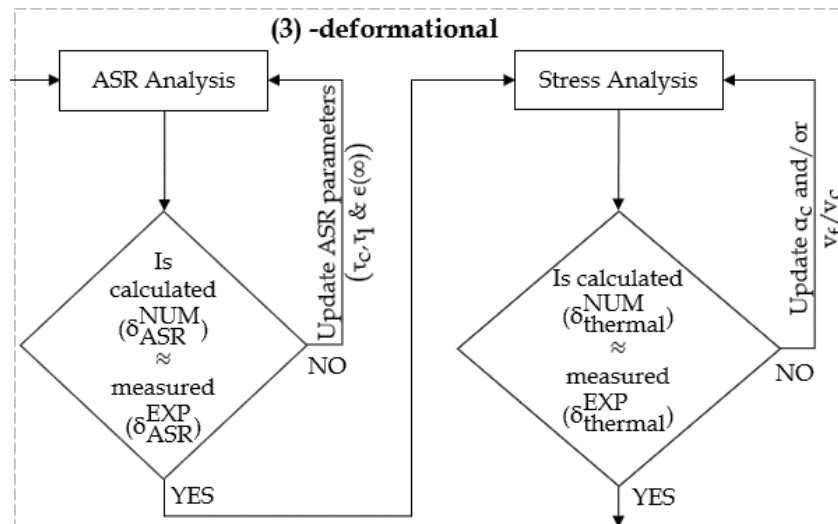


Figure 7.4: Third step in the multi-step model updating approach

The third step in the proposed multi-step approach is the deformation based updating step that consists of two components (i) updating of ASR related parameters (for dam structures affected by ASR), and (ii) updating of the thermal coefficient of expansion of concrete. If a dam structure has no ASR, then the deformation based updating step consist of only (ii). The defining characteristic of this step is that deformations are used in the updating process. The

order in which the two deformation based model updating components is carried out is of non-consequence as the deformation characteristics and/or trends used in updating each component are different. ASR causes a permanent deformation trend observable over time whereas for thermal related displacements, the displacement values and range (i.e., difference between the highest displacement and lowest displacement or summer and winter displacements) are important due to the seasonal nature of thermal loading. The objective in this step is to reduce the difference between the measured displacements and the computed displacements, implemented (in this study) using equation (7.3) which predicts the fitness of the FE model displacements with the measured displacements.

$$\text{Fitness} = \left(1 - \frac{\|\delta_L^{\text{EXP}} - \delta_L^{\text{NUM}}(\theta)\|}{\|\delta_L^{\text{EXP}} - \delta_L^{\text{NUM}}(\theta)\|} \right) \times 100\% \quad (7.3)$$

where the subscript L stands for the load, which is either ASR or Thermal (Figure 7.4), $\delta = \{\delta_1 \dots \delta_n\}$, and $\| \quad \|$ is the 2-norm. The aim is to maximise the fitness percentage value (Kim et al., 2016). Similarly to equation (7.1) and equation (7.2), the model outputs are functions of p updating parameters with each updating parameter subject to a lower and upper bound (i.e., the minimum and maximum value the parameter can have without losing physical meaning).

ASR related updating parameters

The uncertain parameters to be updated in this step are the key parameters essential in defining the reaction kinetics for ASR namely: (i) latency time (τ_l), (ii) characteristic time (τ_c), and (iii) ASR ultimate strain ($\epsilon(\infty)$). Normally, these parameters are determined using laboratory tests including accelerated tests. In this step, the aim is to get a good estimate of these parameters (which will have to be confirmed through laboratory tests) using deformation measurements that capture or are as a result of ASR expansion. As was observed in Chapter 6, the load effect (stress stiffening) due to ASR does not significantly change the dynamic characteristics (natural frequencies) of the dam but may result in potentially observable small variations (both numerically and experimentally).

Thermal stress related updating parameters

Thermal displacements are dependent on the dam geometric parameters, stiffness modulus ratio (E_f/E_c), coefficient of thermal expansion (α_c) and to some extent although not significantly (depending on the arch dam configuration) the system's Poisson's ratio as discussed in Chapter 6. With the dam geometry confirmed and the system stiffness (E_f/E_c)

updated in the vibration based updating step (Step 1), the only parameters that remain uncertain with respect to thermal displacements (unless these parameters are measured) is the coefficient of thermal expansion (α_c) and Poisson's ratio.

7.2.4 Validation of updated model

It is good practice to validate an updated model to ensure its reliability. The proposal with the proposed model updating approach is that the validation be carried out in two steps to validate the overall behaviour (both dynamic behaviour and static behaviour). Firstly, a dynamic behaviour analysis of the updated model is carried out for a specified load case (i.e., water level including all relevant loads) not used in the updating process and the resulting modal parameters validated against the corresponding measured modal parameters. Secondly, a static behaviour analysis of the updated model incorporating all applicable static loads (i.e. for a specified load case, load combination or scenario for which measured static observations not used in the updating process are available) is run and the resulting static behaviour validated against the corresponding observed static behaviour. If satisfactory results are obtained, the updated model can be deemed a reliable and accurate representation of the subject structure, that can be used to infer the structural health state of the structure, perform reliability and risk assessment, predict the structure's remaining useful life, etc. (Chiachío et al., 2022). If not, then the assumptions going into the model development might have to be reconsidered.

In the following section, the implementation of the proposed model updating approach is discussed.

7.3 Case Study: Implementation of Proposed Model Updating Approach for Kouga Dam

7.3.1 Introduction

Kouga Dam (Figure 7.5) is a double curvature concrete arch dam with an approximate height of 72m from the lowest foundation level (LFL), a crest length of 317m, a centrally located uncontrolled ogee spillway with a chute spillway on the left flank controlled by two radial gates, and an outlet works that releases into a concrete channel situated on the right flank. The dam is situated on the Kouga River, about 27 km west of Patensie, in the Eastern Cape Province of South Africa. The dam is an integral constituent of the Gamtoos River Government Water

Scheme (GRGWS), mainly providing irrigating water to the Gamtoos River Valley and adds to the water supply of the Nelson Mandela Metropolitan. The dam was designed and constructed by the formerly known as Department of Water Affairs (DWA), now Department of Water and Sanitation (DWS) ((DSO, 2011) (Prins, 2017).



Figure 7.5: Location of Kouga Dam in South Africa ((Google Earth, 2024) (Google Maps, 2024) (<https://www.news24.com> & <https://www.instagram.com> , 26/02/2024))

7.3.2 Brief History of Kouga Dam

The dam has been in service in 1969 when it was completed. The potential of raising the dam wall by up to 12 m to increase the dam's capacity was accounted for in its design. Additionally, the dam was built on a pulvino pad, a thicker layer of concrete, incorporated to aid with the distribution of loads from the dam wall to the foundation (Egles et al., 1995). Expansion of the dam wall from some chemical reaction was initially noticed in 1972, which was afterward verified as ASR in 1981 through comprehensive laboratory tests including petrographic examination on concrete cores samples obtained from the structure (Prins, 2017). Additionally, the dam's static behaviour was observed to deviate from the expected behaviour due to inelastic movement on the right flank foundation a few years after its completion (Egles et al., 1995).



Figure 7.6: Kouga Dam (<https://snowreport.co.za> , 25/01/2024))

7.3.3 Kouga Dam Characteristics

Table 7.1: Kouga dam characteristics

Dam characteristics	
Wall type	Double curvature arch
Crest length (l_c)	317m
Wall height (h) (LFL to NOC)	72m
Wall thickness (FSL)	6.93m
Wall thickness (NOC)	6.43m
Wall thickness (b_f) (LFL)	10.08m
Level of NOC	RL 156.972m
Level of FSL	RL 149.352m
Level LFL	RL 85.344
Storage capacity	$128.49 \times 10^6 \text{ m}^3$
FSL area	554.5 ha

Table 7.1 provides a summary of characteristics of Kouga Dam (DSO, 2011). The Lowest Foundation Level (LFL), Non-Overspill Crest (NOC), and Full Supply Level (FSL) reduced levels are as given in Table 7.1. The dam has a gross capacity of 128.49 million m³. Based on the characteristics provided above, Kouga dam is a thin arch dam with a $b_t: h$ ratio of 0.14, situated on a valley that is shaped somewhere in between a V-shaped and U-shaped valley configuration that is classified in between a narrow and wide valley with a crest length-to-height ratio ($l_c: h$) of 4.4 (based on the arch dam classification in section 4.2) .

7.3.4 Surveillance of the dam

The surveillance scheme on Kouga Dam, consisting mainly of static monitoring, began in the early 1970s and was subsequently expanded (i.e., the extent and range of instrumentation) after the identification and diagnosis of ASR (Egles et al., 1995). Dynamic monitoring on the dam was only recently implemented (Prins, 2017). The monitoring components pertinent to this study are briefly discussed next. (For a detailed discussion on the monitoring systems installed on Kouga Dam, the reader is directed to Prins, 2017).

Water level

The water level monitoring system comprises of a gauge plate that is recorded daily, and a digital recording system that logs data at least twice daily and every fifteen minutes during a flood.

Temperature

Four thermometers, two on each flank (Figure 7.7 shows the approximate locations of the thermometers using red dots), were placed on the upstream face of the dam wall in 2011. The height of installation of the thermometers is assumed to be at the gallery level. Temperature measurements are logged every 6 hours (i.e., 4 readings per day).

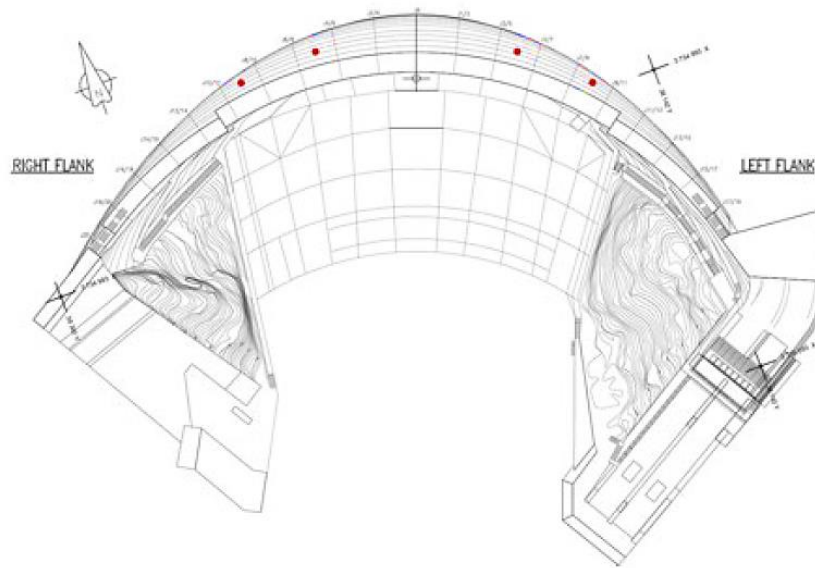


Figure 7.7: Location of thermometers on the upstream face (adopted from Prins, 2017)

Deformation measurements

Three systems have been installed on the dam to measure its deformation behaviour: namely: (i) Geodetic measurements, (ii) Global Navigation Satellite System (GNSS)/Global Positioning System (GPS), and (iii) Trivec measurements. The geodetic system utilises a triangulation network of beacons and targets placed on the downstream face of the dam wall (Figure 7.8), supplemented by precise traverse observations within the gallery, has been operational since 1972. The result of which are three-dimensional displacements of monitoring positions with an accuracy of approximately 0.3-0.5 mm in x- and y- directions and an accuracy of 0.1-0.2 mm in the z- (vertical) (Pretorius et al., 2001). The geodetic measurements are done twice a year, during summer (February-March) and in winter (August-September).

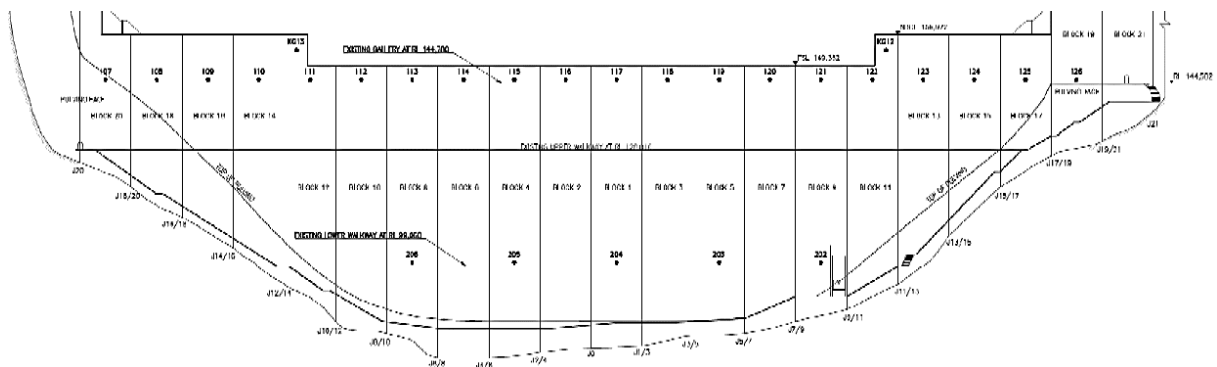


Figure 7.8: Downstream elevation showing geodetic monitoring points (adopted from Prins, 2017)

A GNSS/GPS has provided continuous deformation monitoring, at a frequency of one reading every month, in x-, y- and z-, since 2011. The installation layout (Figure 7.9) consists of two principal base sensors (used as reference stations) in the immediate area of the dam structure, two secondary sensors adjoining the overspill section on the dam wall structure's NOC and a server placed in the outlet house. The trivec system is used to measure bi-annually (once during summer and during winter), the three-dimensional comparative displacements (mainly at foundation level) at 1 m intervals along a vertical borehole.



Figure 7.9: Kouga Dam GNSS/GPS Network (adopted from Pretorius & du Toit, 2014)

Ambient Vibration Monitoring

An AVM system consisting of 24 AC-2x tri-axial accelerometers, a 24bit digitizer and an industrial PC installed at the gallery level of Kouga dam has been in operation since. The resulting natural frequencies can be determined with an estimated error of between $\pm 0.01 - 0.02$ Hz.



Figure 7.10: Approximate location of accelerometers within the gallery (adopted from Prins, 2017)

7.3.5 Observations from the dam monitoring systems at Kouga Dam

Only the observations pertinent to this study and the updating process will be discussed, these include temperature data, water level, geodetic measurements, GNSS/GPS measurements, and AVM measurements.

Temperature

Kouga dam lies in between a valley along Kouga river such that its upstream face is North facing. Consequently, the un-wetted part of the dam wall's upstream side is continuously subjected to direct sunlight whilst the downstream side is in continuous shade all through the day. Additionally, the right flank experiences higher temperatures of about 2-3 °C on average (Figure 7.11) more than the left flank due to the sun's trajectory and terrain at the dam's location (Prins, 2017). The observed temperatures normally reach the highest during summer (February-March) and the lowest in winter (August-September).

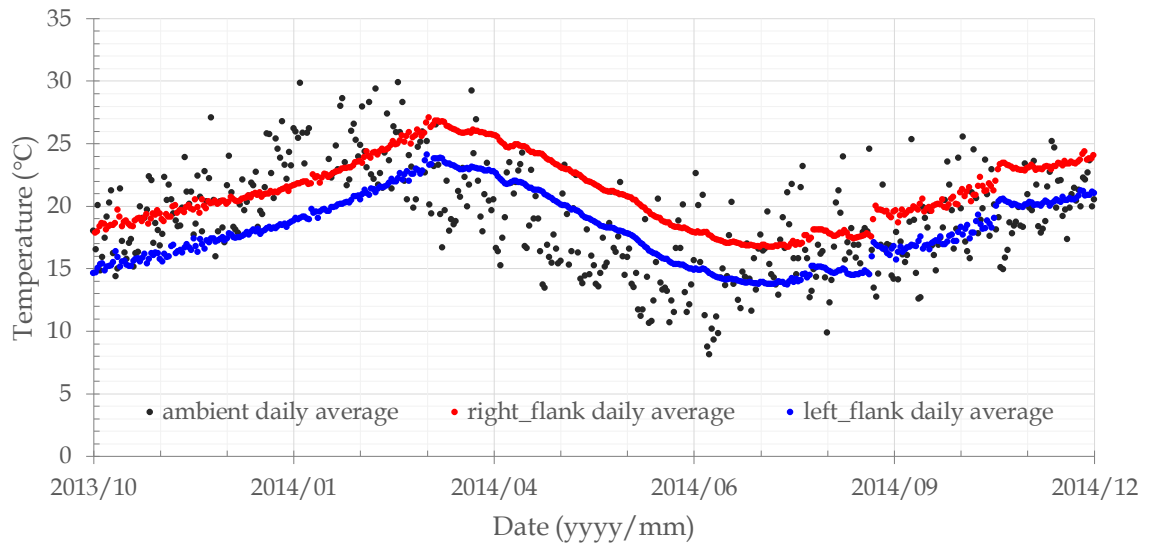


Figure 7.11: Average temperature readings at Kouga dam for a typical one full year cycle

Water level

Figure 7.12 shows the water level record from 1968-2019. The dam generally experiences cold wet winters and dry warm summers. The red dots show the points at which geodetic measurements were taken.

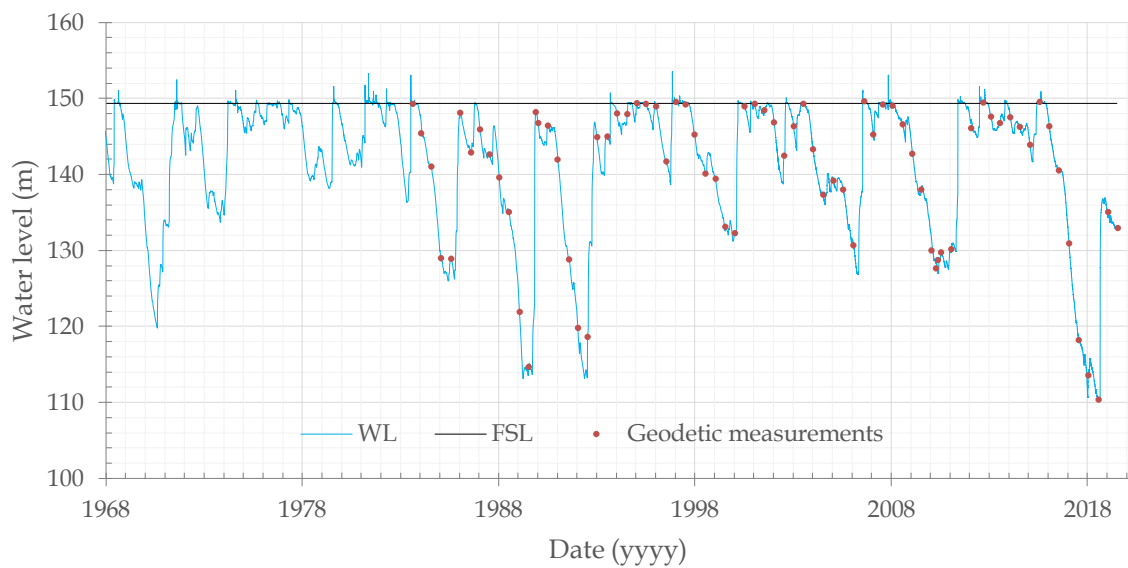


Figure 7.12: Kouga dam water level record

Geodetic measurements

Although geodetic measurements result in three-dimensional displacements at the monitored points, only the x- (radial) and z- (vertical) displacements for the selected monitoring points (corresponding with the points for which model results will be presented in sections 7.3.7-7.3.8) are presented in this section. Figure 7.13 shows the radial (x-) displacements at monitoring points (on blocks) 111, 117 and 122, whilst Figure 7.14 shows the vertical (z-) displacements at monitoring point (block) 117. For the x-displacements, a positive deflection value is towards the downstream and a negative displacement towards the upstream (Figure 7.13).

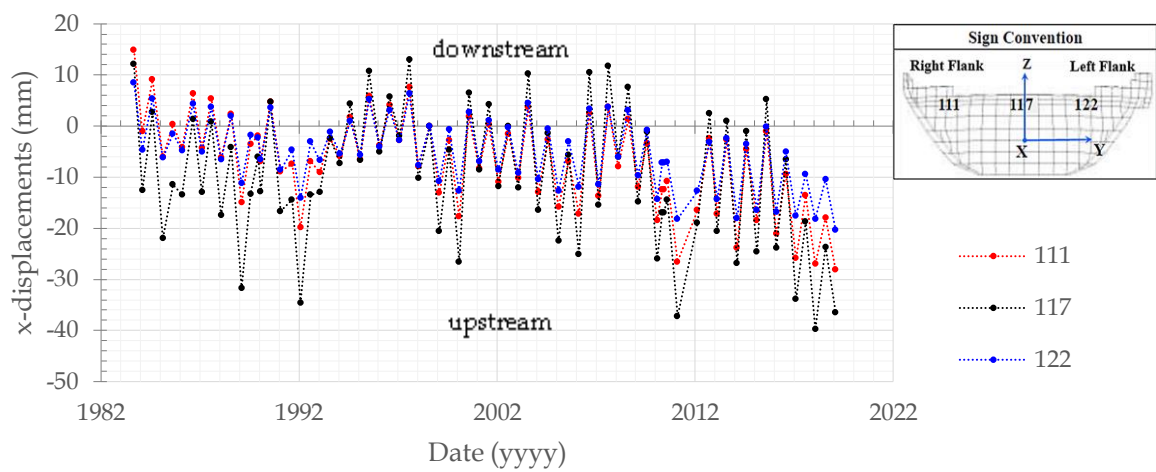


Figure 7.13: Geodetic radial displacement measurements

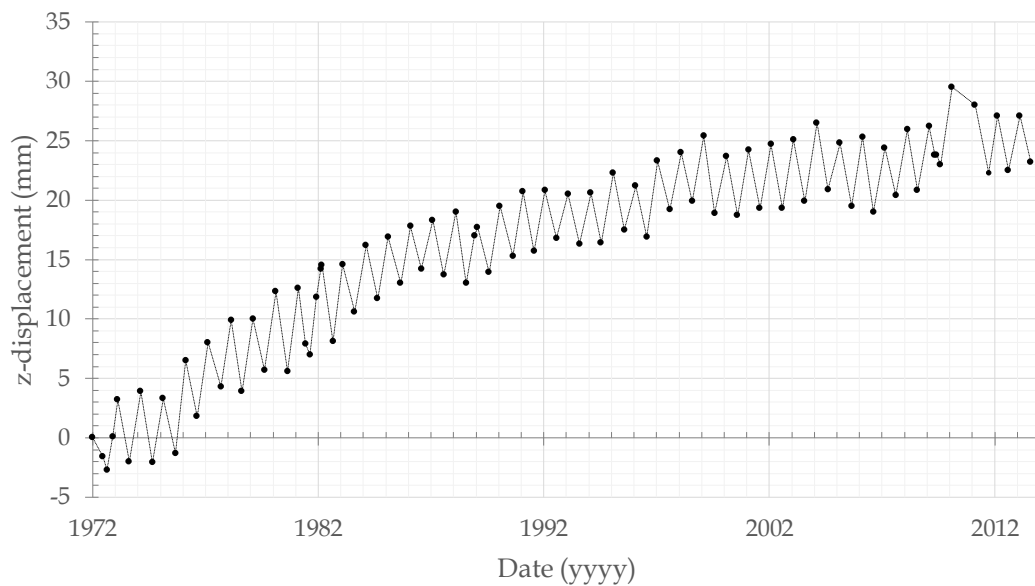


Figure 7.14: Geodetic vertical displacement measurements (at monitoring point 117)

The vertical displacements varied seasonally between 1972 and 1976 driven by temperature mainly. The vertical displacements have been permanently increasing since about 1977 (Figure 7.14). As a result, the whole dam structure has permanently displaced upstream (Figure 7.15), with the right flank experiencing noticeably larger relative displacements compared to the left flank.

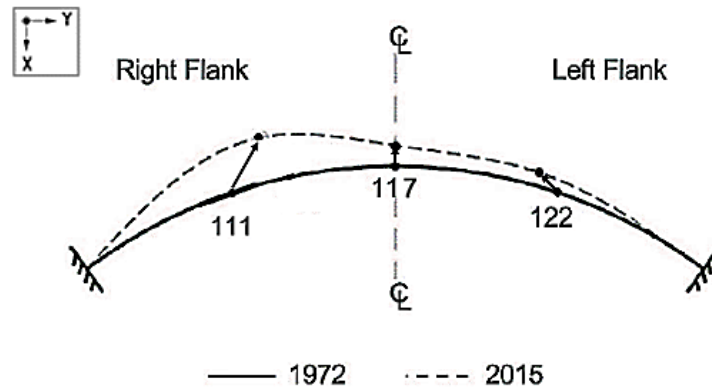


Figure 7.15: Schematic of permanent x-y displacement of dam wall (adopted from Prins, 2017)

GNSS/GPS data

The GNSS/GPS allows for the continuous observation of the behaviour of the structure throughout the year with displacements recorded monthly, as opposed to geodetic measurements which are only carried out twice a year. Figure 7.16 shows the left and right flank x- (radial) displacements for the period 2013-2014.

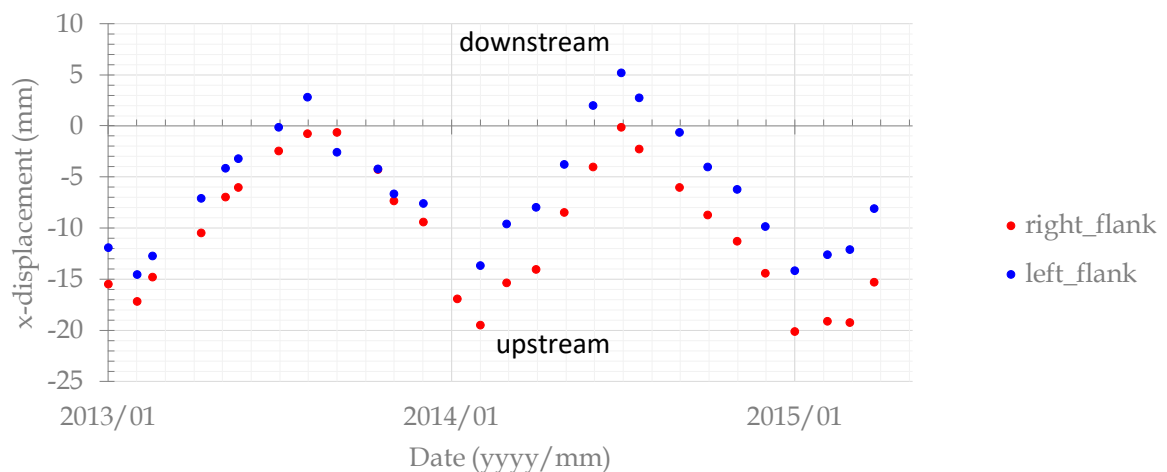


Figure 7.16: GNSS/GPS radial displacements on both the flanks of Kouga dam

Ambient vibration measurements

Table 7.2 provides a summary of the AVM results relevant to this study. The mode shape sequence is consistent with both the theoretical and observed mode shape sequences reported in Chapter 5 for thin double curvature arch configurations in U-shaped valleys.

Table 7.2 Kouga dam measured natural frequencies and mode shapes

Mode number and description	Mode shape	Measured natural frequency (Hz)	
		Sept 2009 (WL - 137.2)	July 2013 (WL - 146.7)
1 - First antisymmetric		3.60	3.31
2 - First symmetric		4.01	3.63
3 - Second symmetric		4.70	4.63
4 - Second antisymmetric		6.32	6.07

Additional observations

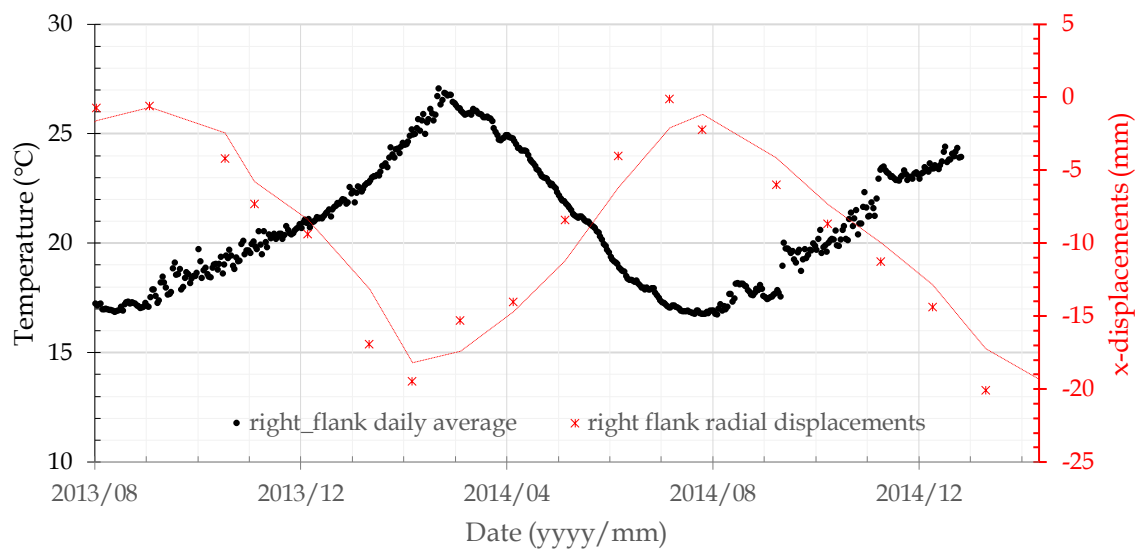


Figure 7.17: Temperature and displacement data for the right flank

illustrated in Figure 7.21. The dam wall was modelled as monolithic and isotropic based on the assumption that the interlocking mechanism between the blocks allows for the monolithic behaviour. The upstream water was modelled as a fluid part by means of acoustic elements. Whilst the foundation was modelled as a massless thermodynamically isolated deformable component divided into three parts (Figure 7.19) (based on the available information and monitoring data) to enable the adjustment of stiffness during the updating process. Table 7.3 provides a summary of the mechanical material properties initially assumed based on the available information for Kouga dam.

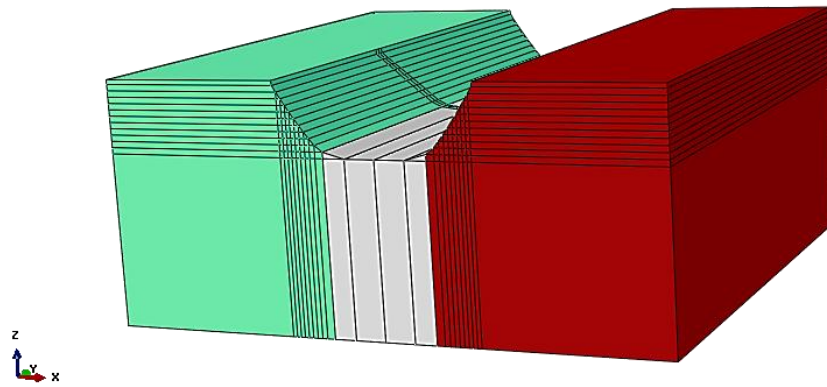


Figure 7.19: Developed foundation part partition illustration

Table 7.3: Kouga dam model mechanical material properties

Property and Symbol	SI Units	Concrete Wall	Rock Foundation		
			Left flank	Bedrock	Right flank
Static Elastic Modulus ($E_{c, static}$)	GPa	30	16	9	13
Dynamic Elastic Modulus ($E_{c, dynamic}$)	GPa	39	28	22	26
Poisson's ratio (ν_c)	-	0.2	0.22	0.22	0.22
Density (ρ_c)	kg/m ³	2440	n/a	n/a	n/a
Coefficient of expansion (α_c)	-	1.0×10^{-5}	n/a	n/a	n/a

In-line with the proposed model updating approach, dynamic modelling, thermal modelling, ASR modelling and stress modelling was carried out on the developed model. A linear perturbation procedure which performs Eigen property extraction using the Lanczos method was used (as implemented in ABAQUS) as this method accounts for surface-based tie constraints on the structural-acoustic coupled surface (Vezi, 2014). The temperature model was presumed to be controlled by the laws of heat transfer through radiation, irradiation, and convection, which need data concerning the current climate such as air temperature, wind speeds, orientation of the dam and water temperature (refer to section 4.5). Solar radiation loads are affected by the dam location (latitude & longitude), its orientation, month, time of the day and cloud cover. As mentioned in section 7.3.5, the downstream side of the dam is in the shadow from sunrise to sunset (illustrated in Figure 7.20). Table 7.4 provides a summary of thermal properties, used in this study.

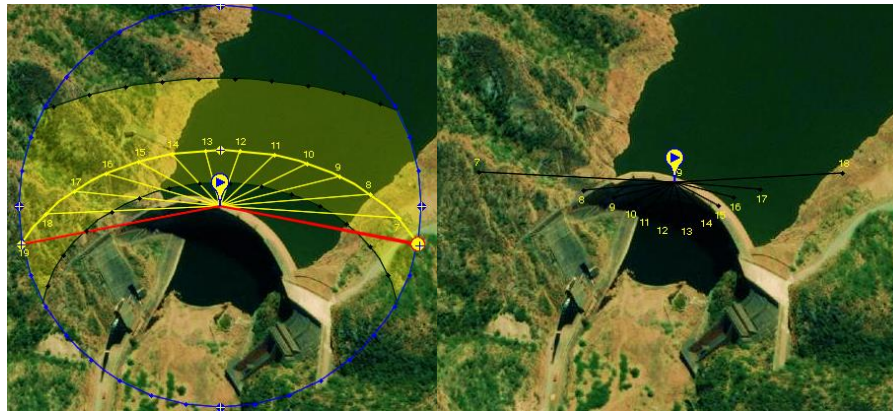


Figure 7.20: Sun's trajectory and shadow at Kouga dam (<https://www.sunearthtools.com/tools> , 26/02/2024)

Table 7.4: Concrete dam wall thermal material properties

Property and Symbol	SI Units	Concrete Wall
Specific heat (c_c)	J/(kg.K)	900
Thermal conductivity (k_c)	W/ (m K)	2.0
Convection coefficient (h_c)	W/ (m ² K)	19
Emissivity (e_c)	-	0.88
Solar absorptivity(a_c)	-	0.65

The ASR modelling approach implemented in this study has been detailed in section 4.6. Furthermore, the hydrostatic load for the stress and/or displacement analysis was modelled

using the usual linearly varying function for static behaviour analysis. Figure 7.21 shows the meshed Kouga dam-foundation-reservoir model.

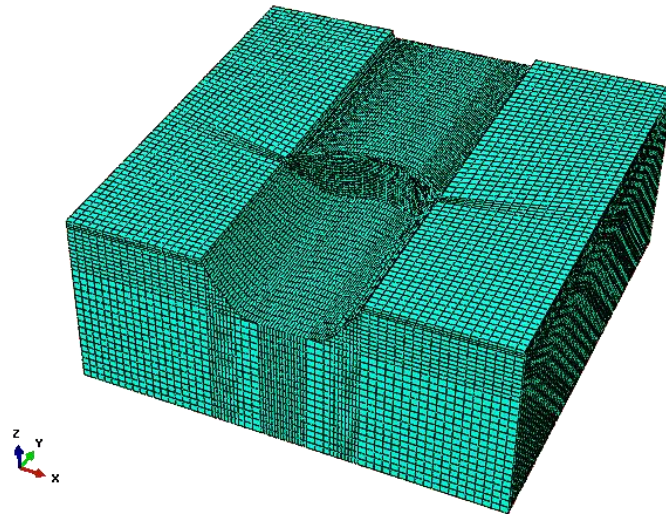


Figure 7.21: Kouga Dam FE model dam-foundation-reservoir system

7.3.7 Kouga dam FE model sensitivity results

In this section the developed Kouga dam model's sensitivity to different parameters is presented as a means of identifying the dam's behaviour in the context of the already presented results and to a lesser extent to validate the theoretical analysis results. To this end, the same material properties and load conditions used for the theoretical study of the different arch configurations are implemented in the analysis of the developed Kouga Dam model.

Dynamic behaviour characterisation

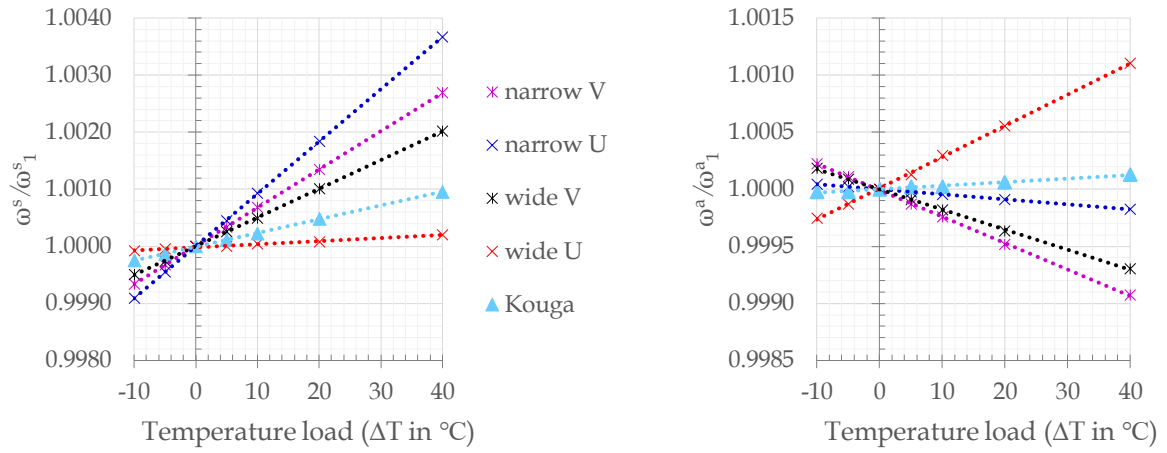
Table 7.5 shows the first ten theoretical modal parameters obtained from the developed Kouga dam model in comparison with the thin wide-U double curvature arch configuration modal properties for the different boundary conditions and the dam-foundation-reservoir system interaction. The mode shape sequence from the developed model starts off similarly to the thin wide U configuration with the antisymmetric mode as the first mode shape followed by the first and second symmetric modes. The sequence of the first three mode shapes of Kouga dam model are not affected by changes in the dam-foundation-reservoir system whilst the rest of the mode shape sequence does not remain unchanged. Additionally, it can be observed that the sequence of the mode shapes for the Kouga dam model is not identical to any of the investigated arch dam configuration mode shape sequences presented in Chapter 5 (refer to section 5.5).

Table 7.5: Kouga Dam vs Thin wide-U shaped dam configuration(s) theoretical vibration modal parameters

Dam Curvature	Conditions	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10
Kouga Dam (Double)	Fixed boundary, empty dam	A 	S 	S 	A 	S 	C 	A 	C 	S 	C 
		4.73 Hz	5.22 Hz	6.76 Hz	8.07 Hz	9.95 Hz	11.00 Hz	12.44 Hz	13.09 Hz	15.72 Hz	16.24 Hz
	$E_f/E_c=1$, empty dam	A 	S 	S 	A 	S 	C 	C 	A 	C 	C 
		4.35 Hz	4.92 Hz	6.38 Hz	7.68 Hz	9.51 Hz	10.28 Hz	11.66 Hz	12.44 Hz	14.24 Hz	14.62 Hz
	$E_f/E_c=1$, full dam	A 	S 	S 	C 	C 	A 	C 	C 	C 	S 
		3.57 Hz	3.90 Hz	5.33 Hz	5.88 Hz	6.51 Hz	6.92 Hz	7.56 Hz	8.12 Hz	8.50 Hz	8.60 Hz
Double	Fixed boundary, empty dam	A 	S 	S 	A 	S 	A 	C 	S 	C 	A 
		2.36 Hz	2.46 Hz	3.35 Hz	3.46 Hz	4.26 Hz	5.03 Hz	5.76 Hz	5.90 Hz	6.28 Hz	6.76 Hz
	$E_f/E_c=1$, empty dam	A 	S 	S 	A 	S 	A 	C 	S 	C 	A 
		2.25 Hz	2.39 Hz	3.28 Hz	3.40 Hz	4.20 Hz	4.97 Hz	5.60 Hz	5.83 Hz	6.11 Hz	6.72 Hz
	$E_f/E_c=1$, full dam	A 	S 	S 	A 	S 	A 	C 	C 	C 	S 
		1.59 Hz	1.72 Hz	2.24 Hz	2.50 Hz	3.15 Hz	3.82 Hz	3.93 Hz	4.44 Hz	4.49 Hz	4.54 Hz

A – antisymmetric mode
C – complex mode
S – symmetric mode

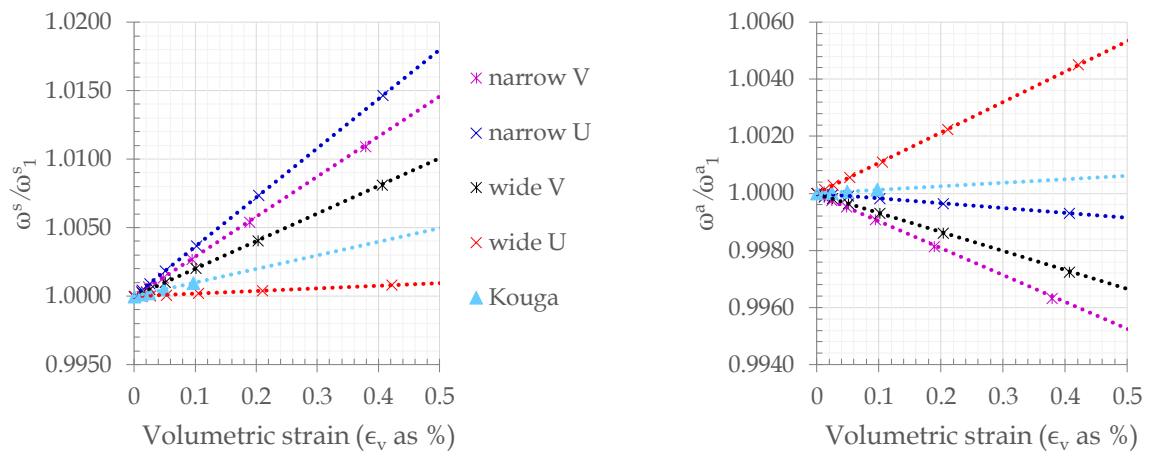
Figure 7.22 and Figure 7.23 show the variation of natural frequencies with thermally induced and volumetric expansion (ASR) related stress-stiffening respectively (consistent with the findings presented in Chapter 6). The dynamic behaviour of Kouga dam lies in-between the wide-V and wide-U classified arch configurations for the first symmetric, whilst for the antisymmetric modes, it lies in-between the wide-V and narrow-U classified arch configurations.



a) Thin arch first symmetric mode

b) Thin arch first antisymmetric mode

Figure 7.22: ω/ω_1 versus applied thermal load graphs for double curvature arch configurations

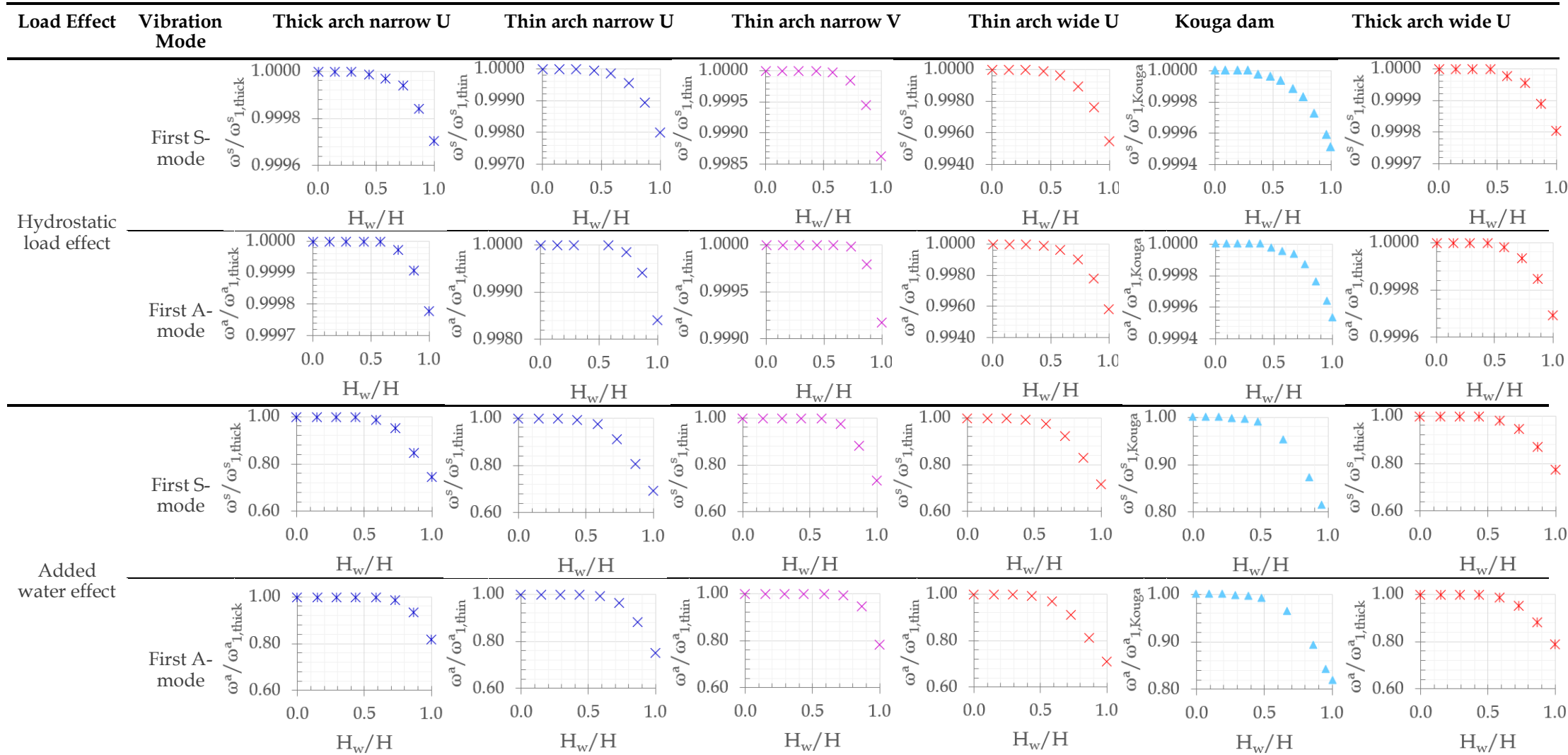


a) Thin arch first symmetric mode

b) Thin arch first antisymmetric mode

Figure 7.23: ω/ω_1 versus ϵ_v graphs for double curvature arch configurations

Table 7.6: Effect of the water load on the dynamic behaviour of selected arch dam configurations



A - antisymmetric mode

S - symmetric mode

Table 7.6 shows the effect of the water load on the dynamic properties of Kouga dam in the context of the selected arch dam configurations. The sensitivity of the frequency ratio to the water level decreases as the water level decreases. The point below which there is no significant change in the frequency ratio is below $H_w/H=0.5$ for both the symmetric and antisymmetric modes as well as for the different load cases. The added mass effect of the water level has a greater effect than the stress-stiffening effect on the dynamic behaviour for Kouga dam similarly to the investigated configurations.

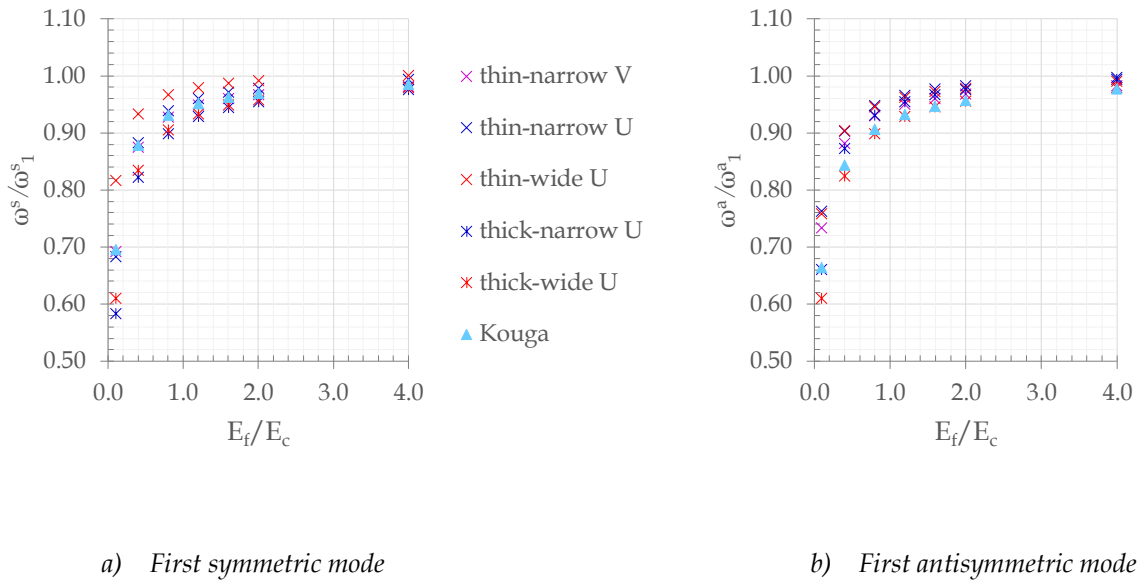


Figure 7.24: ω/ω_1 versus E_f/E_c for selected double curvature arch configurations

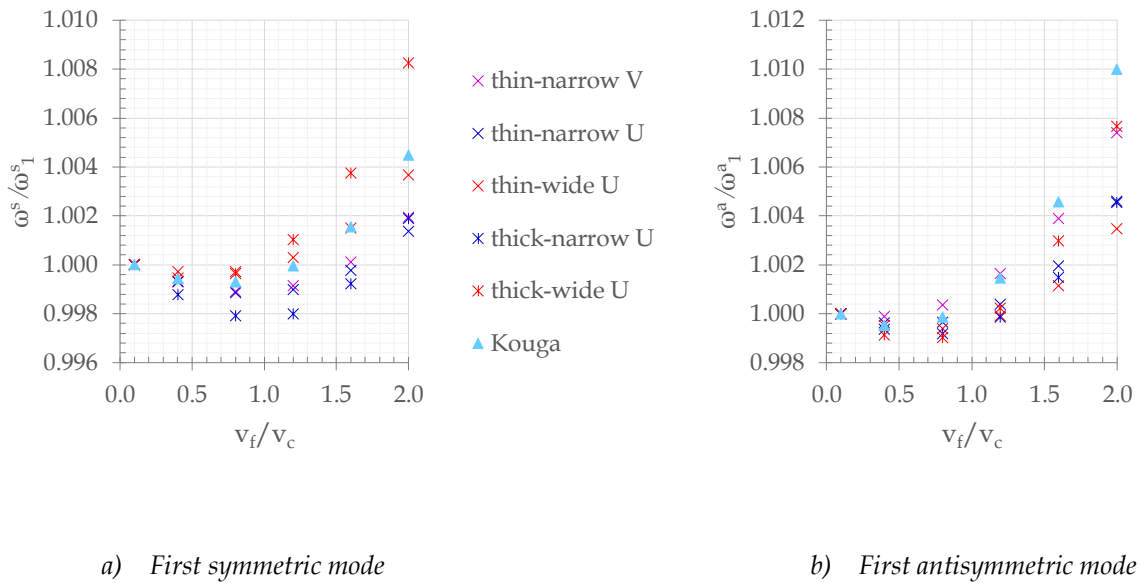


Figure 7.25: ω/ω_1 versus v_f/v_c for selected double curvature arch configurations

Figure 7.24 to Figure 7.27 show the variation of the natural frequency with material properties including Kouga dam model's sensitivity to variation with material properties. The resulting Kouga dam sensitivities do not deviate from the obtained sensitivities as discussed in Chapter 6.

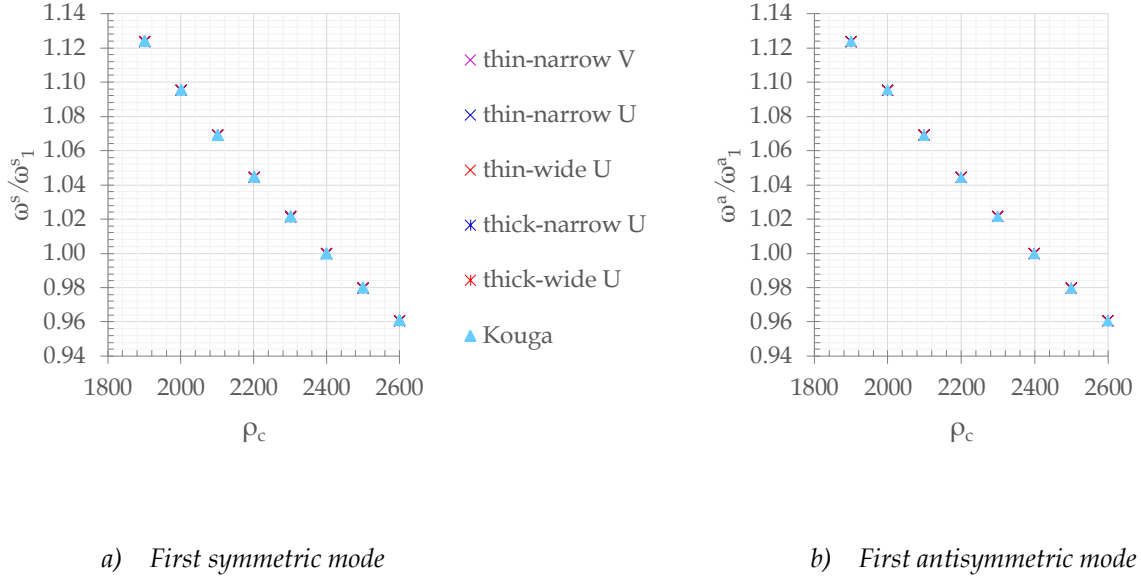


Figure 7.26: ω/ω_1 versus ρ_c for selected double curvature arch configurations

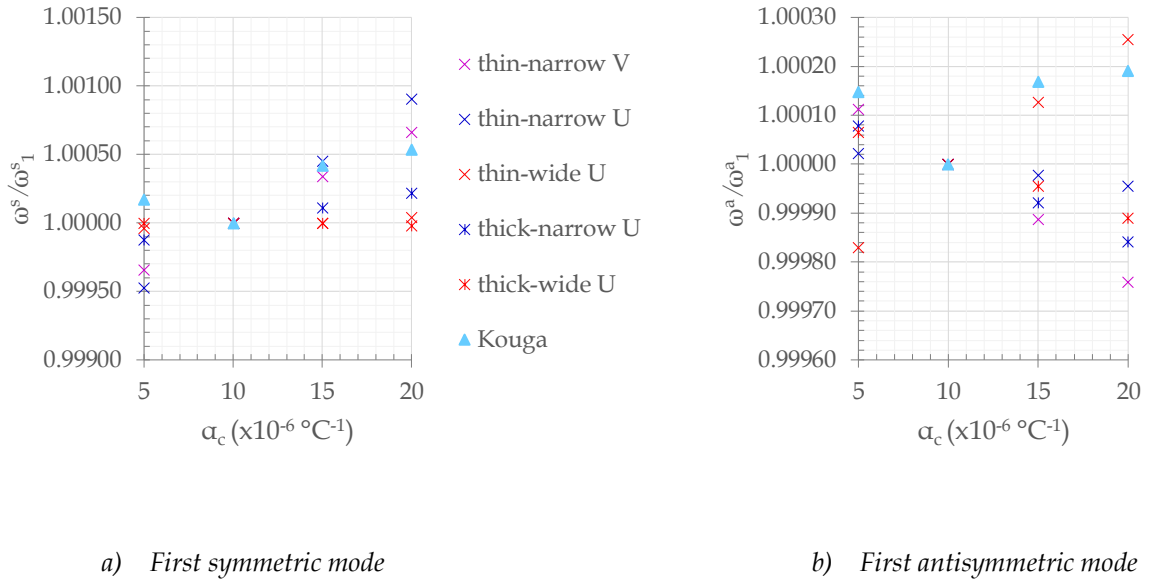


Figure 7.27: ω/ω_1 versus α_c for selected double curvature arch configurations

Static behaviour characterisation

Table 7.7 provides a summary of the static theoretical behaviour of Kouga dam in comparison with the thin wide U double curvature arch configuration static displacements. Kouga dam's classification is in between a narrow to wide and U- to V- shaped valley (section 7.33). Additionally, the crest thickness for Kouga dam is greater than that of the investigated arch dam configurations.

Table 7.7: Summary of theoretical static behaviour characteristics for Kouga dam

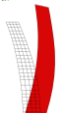
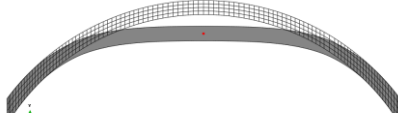
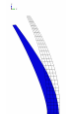
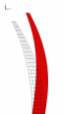
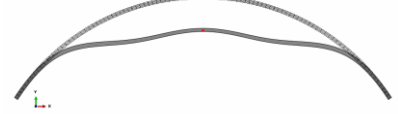
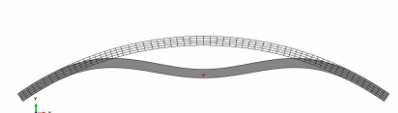
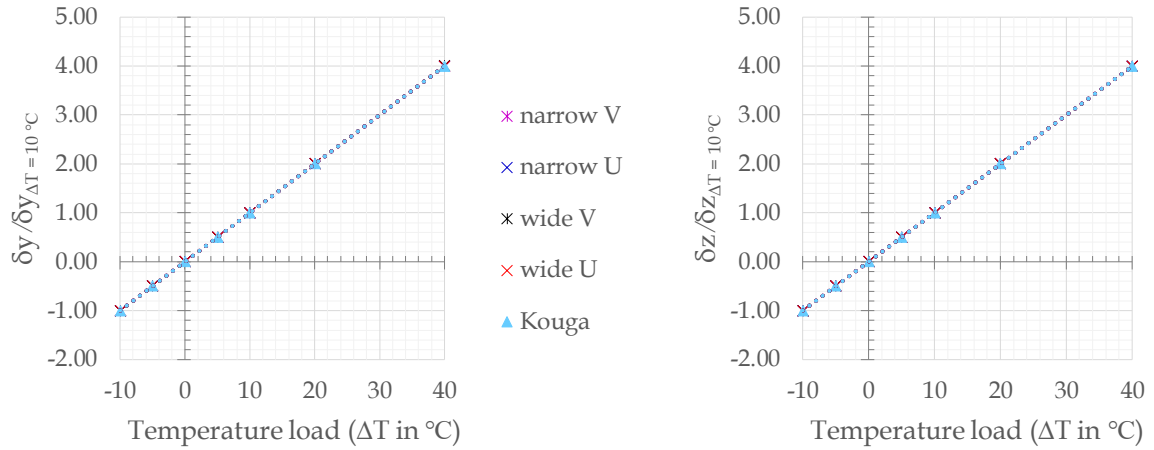
Configuration	Double Curvature			
	Low temp (mm)	High temp (mm)	Hydrostatic (mm)	Crest level displacement profile due to hydrostatic load
Kouga				
δ_y	-17	17	-13	
δ_z	-9	9	-0.5	
Thin wide U				
δ_y	-24	24	-58	
δ_z	-14	14	-5	
Thin narrow V				
δ_y	-15	15	-23	
δ_z	-12	12	-7	

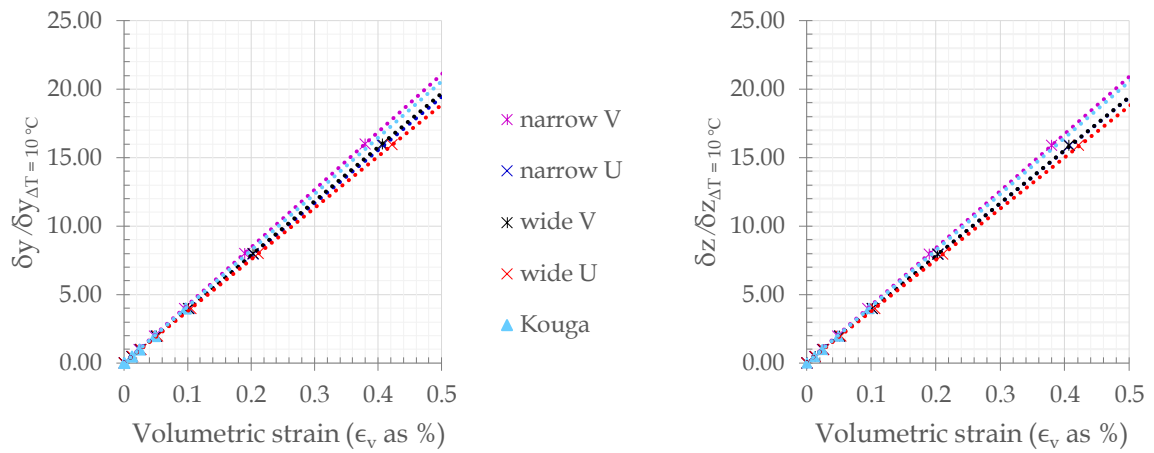
Figure 7.28 and Figure 7.29 show the variation of the displacements with thermally induced and volumetric expansion (ASR) related stress-stiffening respectively (consistent with the findings presented in Chapter 6). The static behaviour of Kouga dam does not deviate from the observed behaviour as discussed in Chapter 6.



a) Thin arch radial displacements

b) Thin arch vertical displacements

Figure 7.28: $\delta/\delta_{\Delta T=10^{\circ}\text{C}}$ versus applied thermal load graphs for double curvature arch configurations



a) Thin arch radial displacements

b) Thin arch vertical displacements

Figure 7.29: $\delta/\delta_{\Delta T=10^{\circ}\text{C}}$ versus ϵ_v graphs for double curvature arch configurations

Table 7.8: Effect of the water load on the static behaviour of selected arch dam configurations

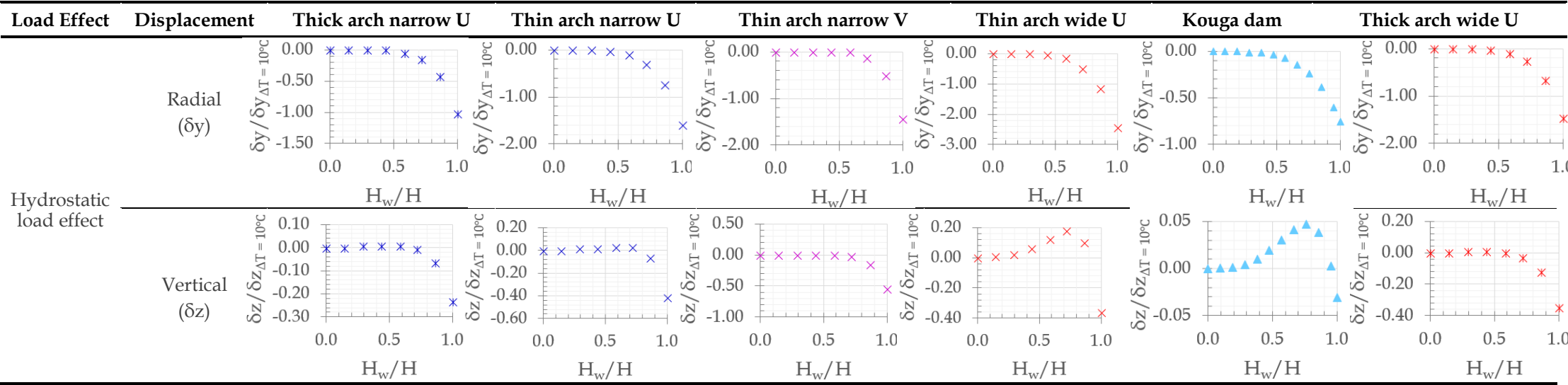


Table 7.8 shows the variation in displacements due to the hydrostatic load with changing water levels for Kouga dam in the context of the selected arch dam configurations. The radial displacements follow a similar profile as that obtained for the variation in the dynamic behaviour (as the other profiles for the selected arch dam configurations). Whilst the vertical displacement profile with variation in water level for Kouga dam is similar to that of the thin wide-U arch configuration, which are both distinctly different from the rest of the vertical displacement profiles (Table 7.8).

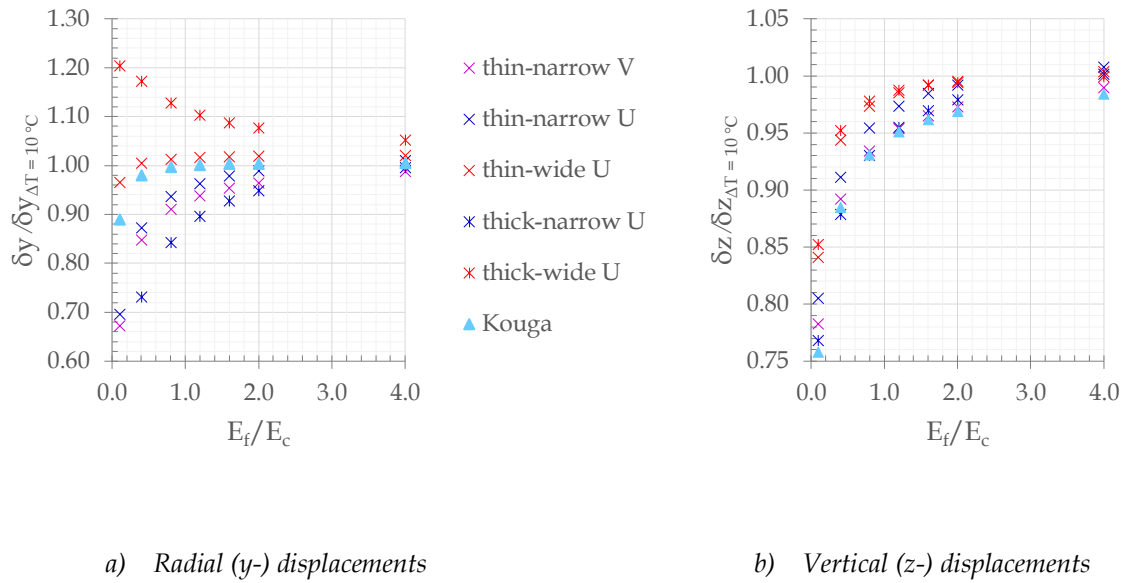


Figure 7.30: $\delta/\delta_{\Delta T=10^\circ\text{C}}$ versus E_f/E_c for selected double curvature arch configurations

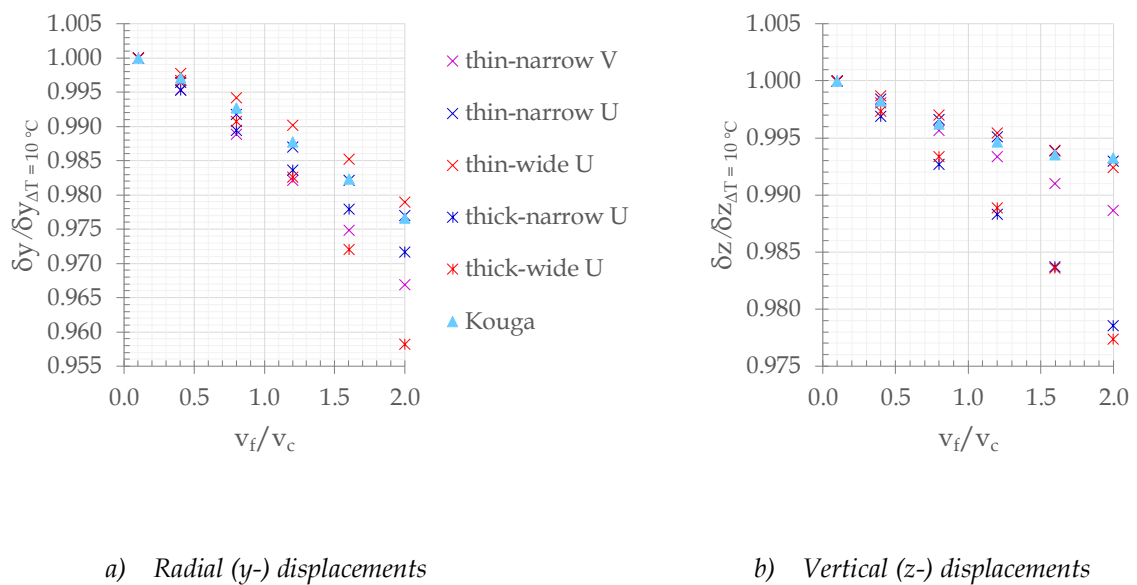


Figure 7.31: $\delta/\delta_{\Delta T=10^\circ\text{C}}$ versus v_f/v_c for selected double curvature arch configurations

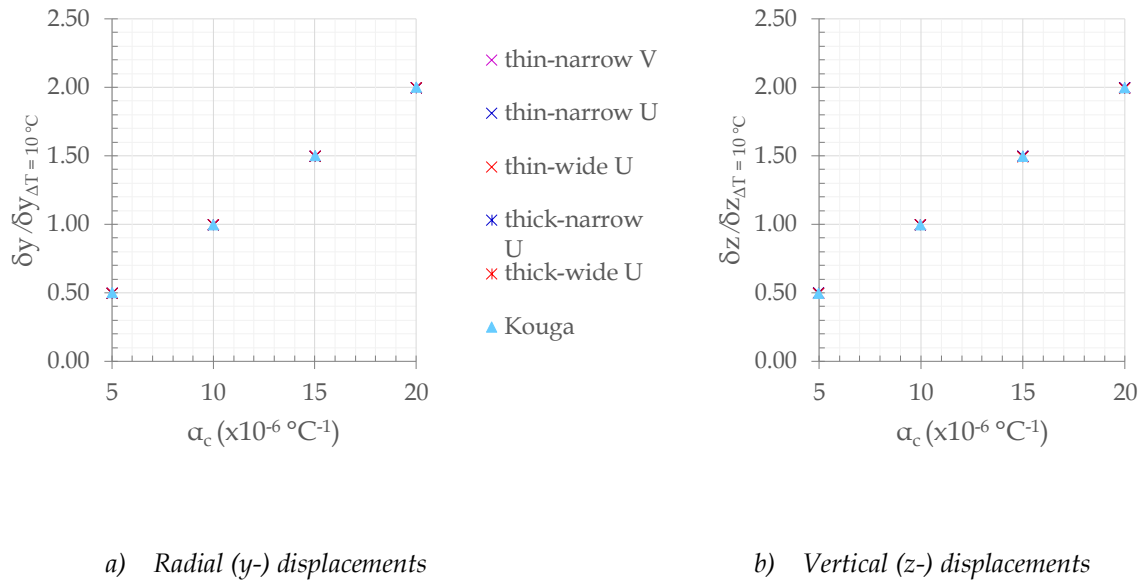


Figure 7.32: $\delta/\delta_{\Delta T=10^\circ\text{C}}$ versus α_c for selected double curvature arch configurations

Figure 7.30 to Figure 7.32 show the displacement profiles with variation in material properties for Kouga dam relative to the selected arch dam configuration profiles. In all cases Kouga dam's displacement profiles (both radial and vertical) show similar relation and/or sensitivities (as was observed for the various configurations) to variation in material properties.

7.3.8 Updating of Kouga dam's FE model

As already mentioned in section 7.2.2, the analysis of the heat transfer mechanisms and the resulting temperature distribution did not fall part of the scope of this project. Consequently, the updating of the thermal properties of the system in relation to heat transfer mechanisms is not presented in this study. The accuracy of the temperature distribution used in the updating of the thermal displacements will be inferred or assumed to be confirmed in the updating of the thermal displacements step.

Vibration based (vibro-) model updating

Table 7.9 shows the stiffness system and density updating (as described in section 7.2.1) using the first and second symmetric and antisymmetric modes measured from the dam in July 2013 at a water level RL reading of 146.7 (Table 7.2). The ratio of the concrete material stiffness to the left flank, right flank and bedrock foundation stiffness was kept constant throughout the

analysis. The objective is to minimise equation (7.1) which resulted in an $E_{c, \text{dynamic}} = 34.6 \text{ GPa}$ and $\rho_c = 2440 \text{ kg/m}^3$ giving the best model results (Table 7.9).

Table 7.9: Kouga dam model system stiffness and density updating

$E_{c, \text{dynamic}}$	39	35	34	34.6	34.6	34.6	w_{ω_i}
ρ_c	2440	2440	2440	2440	2400	2300	
Mode 1	3.52	3.32	3.27	3.30	3.32	3.37	6.74×10^{-2}
Mode 2	3.87	3.65	3.60	3.63	3.65	3.70	9.29×10^{-1}
Mode 3	5.32	5.02	4.95	4.99	5.02	5.09	2.66×10^{-3}
Mode 4	6.86	6.47	6.38	6.43	6.48	6.59	7.88×10^{-4}
$\sum_{i=1}^{n_{\text{modes}}} \left(\frac{\omega_i^{\text{EXP}} - \omega_i^{\text{NUM}}(\theta)}{\omega_i^{\text{EXP}}} \cdot w_{\omega_i} \right)^2$	2.08×10^{-3}	1.47×10^{-5}	4.59×10^{-5}	3.91×10^{-6}	1.32×10^{-5}	1.64×10^{-4}	$\Sigma = 1.0$

Another important aspect in the vibration-based model-updating step is the mode shape updating, the aim of which, is to account for or capture any variations in the system stiffness. Here the analyst may choose to analyse the mode shape variation with the objective to minimise equation (7.1) and equation (7.2) iteratively in sequential steps or simultaneously (section 7.2.1). In this study, the natural frequency was calibrated first with the model partitioned to account for the variations in the foundation stiffness (based on the information provided by DWS). In general, the applied stiffness variations resulted in good correlation between the measured mode shapes and the corresponding FEM based mode shape (Figure 7.33 and Table 7.10). Thus, the foundation stiffness values do not need to be updated further. This highlights the importance of up-to-date information for the different unknown and/or uncertain parameters (were possible), as this is very beneficial for the updating process (here illustrated) by reducing the number of unknowns the analyst/or engineer needs to update.

Figure 7.35 illustrates the measured and calculated first four arch mode shapes (as shown in Table 7.2), indicating the fitness between the two sets of data. It can be observed that the left flank foundation stiffness is still softer than assumed in the model. Consequently, the left flank foundation stiffness still needs to be refined further. However, for the purposes of this discussion, it is assumed that the mode shapes are still sufficiently accurate.

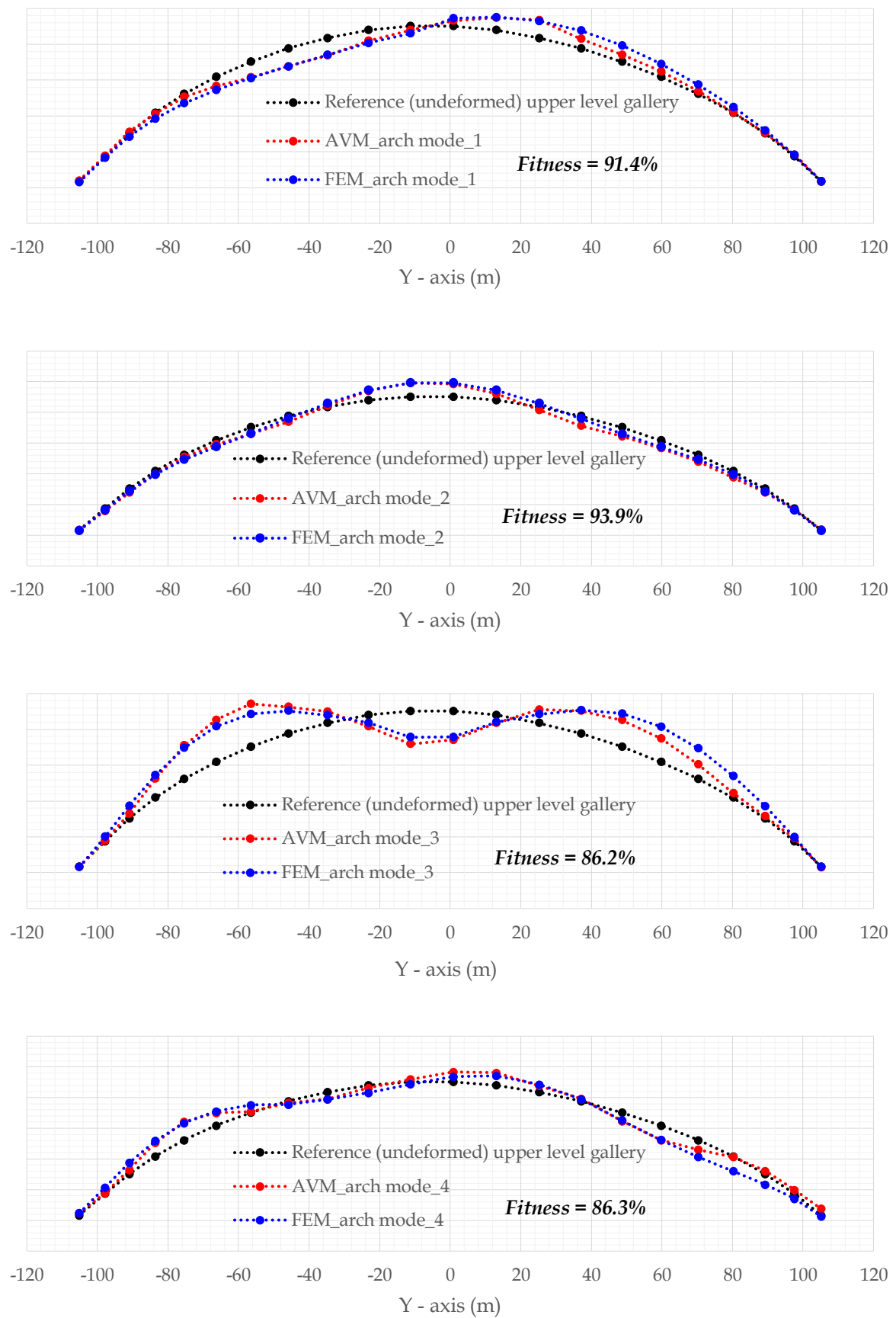


Figure 7.33: First four arch mode shape comparison between the measured and FEM- based mode shapes

Table 7.10 shows the results determined using equation (7.2). Since, there was no need to adjust the stiffness values, equation (7.2) was then used to calculate the cross relation between the measured and FEM-based mode shapes, to illustrate that the equation does result in minimisation of the function to achieve updating. Since there was strong correlation between the corresponding measured and analysed mode shapes, a weighting factor of 1 (i.e., $w_{\psi_i}=1$) was implemented for all the mode shapes.

Table 7.10: Objective function calculation results with respect to updating of mode shapes

	FEM_arch mode_1	FEM_arch mode_2	FEM_arch mode_3	FEM_arch mode_4
AVM_arch mode_1	3.55	9.11	7.05	5.39
AVM_arch mode_2	9.09	1.62	9.77	8.52
AVM_arch mode_3	9.45	6.57	5.97	8.77
AVM_arch mode_4	4.69	7.63	7.20	4.56

ASR related deformation based (-deformational) model updating

Critical to the modelling of ASR are the assumptions on the spatial distribution of temperature, humidity, and extent of ASR. As has already been noted (section 4.6), the ASR chemical reaction and its progression is dependent on temperature. Thus, the spatial temperature profile is critical in ensuring that ASR is modelled correctly (Saouma, 2014). Additionally, the assumption of where and how much of the dam is assumed to be expanding due to ASR is critical in determining the extent of application of the ASR constitutive model. For dams, the humidity is assumed to be 1 unless there is evidence indicating contrary, in which case, the spatial distribution of humidity will have to be carefully considered and/or determined. Assuming that the correct temperature profile and an accurate extent of reaction has been assumed, the deformations due to ASR may be used to update the parameters used to define the assumed ASR reaction kinetics through an iterative process as described in section 7.2.3. Table 7.11 shows the updating results (graphically shown in Figure 7.34) of the characteristic parameters that define the assumed ASR reaction kinetics that resulted in the best displacement profile related to ASR. The parameters that resulted in the best displacement fit are as given in Table 7.11. (Note that these values would need to be confirmed or compared with laboratory test observations to ensure their reliability).

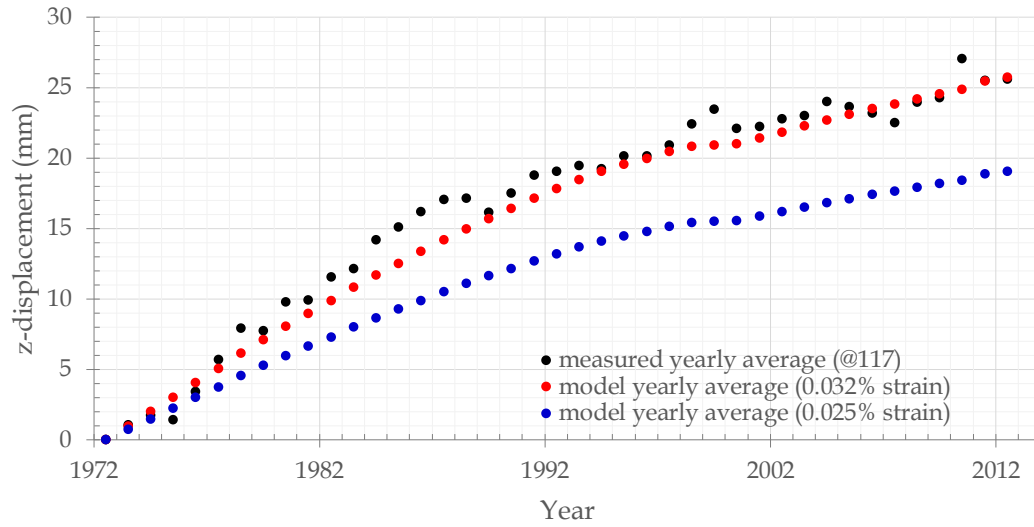


Figure 7.34: Yearly average vertical displacements

The yearly average displacements were determined from the geodetic measurements (illustrated by the black plot in Figure 7.34). The two FEM based displacement profiles used in the updating process are also plotted in Figure 7.34. As can be seen in Table 7.11, the best-fit plot has a fitness of about 82.2%.

Table 7.11: Kouga dam model ASR reaction kinetics parameter updating

τ_c	8000	5100
τ_l	1000	5500
$\epsilon(\infty)$	0.025%	0.032%
<i>Fitness =</i>	37.2%	82.2%

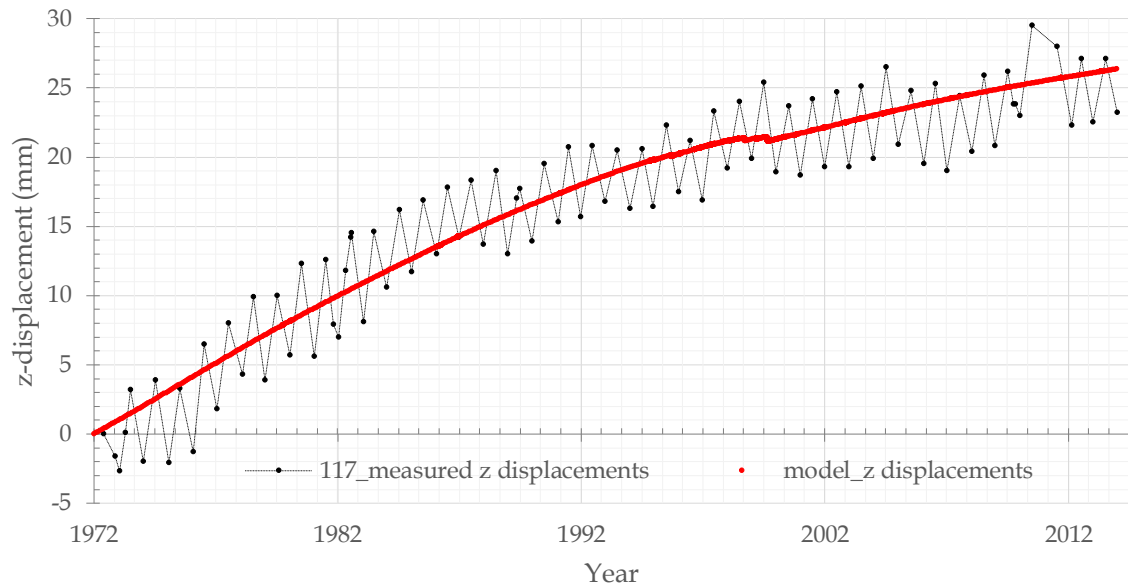


Figure 7.35: ASR related displacement trend

The displacement trend obtained from the ASR model produces a good approximation of the measured displacement trend (Figure 7.35). Figure 7.36 shows the corresponding calibrated expansion curve.

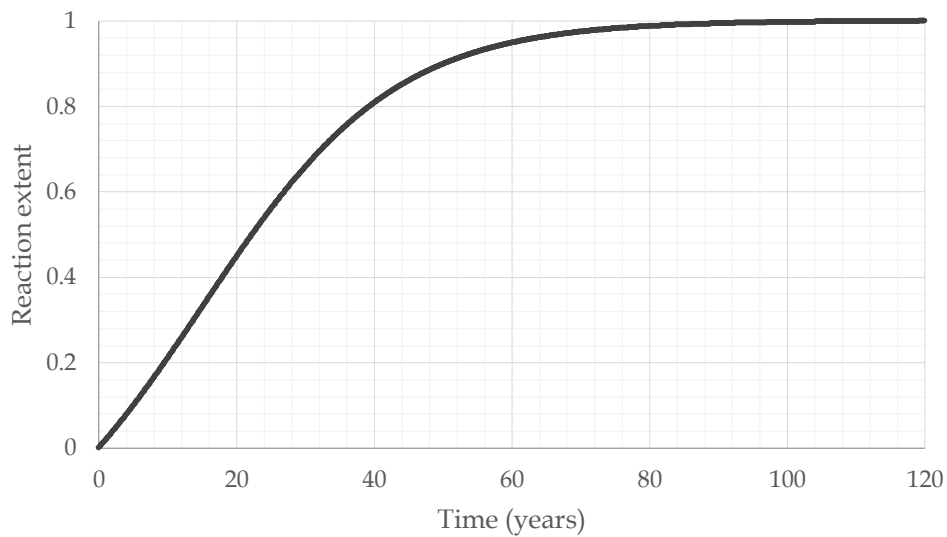


Figure 7.36: Calibrated expansion curve

Thermal stress related deformation based (-deformational) model updating

In this section the displacements used in the updating process include both hydrostatic and thermal loads for the selected seasonal period. Table 7.12 shows the updating of the coefficient of thermal expansion (as described in section 7.2.3) (in this instance the system Poisson's ratio (ν_f/ν_c) was assumed to be as estimated based on the information provided by DWS). The best fitness as a function of the estimated coefficient of thermal expansion based on the GNSS/GPS readings between January 2013 to December 2014 varied for the left flank, right flank, and overall seasonal displacement (Table 7.12). An $\alpha_c=7.7\mu$ resulted in a the best fitness value of 53% for the left flank measurements, whilst the best fitness value of 45.4% was obtained for the right flank corresponding to an estimated $\alpha_c=8.8\mu$ and the best seasonal temperature difference was obtained for an estimated $\alpha_c=9.9\mu$. Additionally, it is worth noting that a best fit (smooth) model of the measured temperature profiles was assumed (for the left flank, right flank and ambient air temperature) for the model, which, explains the observed difference in displacements (Figure 7.37). It is postulated, that if the actual temperature readings were used in the thermal modelling the model displacement would have followed much closely the measured displacement (as the measured displacement are not perfectly sinusoidal (Figure 7.37)). Hence, the maximum fitness that could be obtained for the period analysed was 53% for the left flank and 45% for the right flank. Figure 7.37 shows the best-fit plots of the seasonal radial displacement variation for the left and right flanks. Whilst Figure 7.38 illustrates the best-fit plot of the summer displacement profile using geodetic measurements. A best fitness value of 52.8% was obtained. Considering the left flank, right flank, seasonal displacement and assumptions made in the modelling, the $\alpha_c=9.9\mu$ was selected as the best estimate for the analysis.

Table 7.12: Kouga dam model updating of the coefficient of thermal expansion

$\alpha_c (\times 10^{-6}/^{\circ}\text{C})$	12	11	9.9	8.8	7.7	6.6
Left flank GNSS/GPS seasonal displacement (mm)	19.7	19.7	19.7	19.7	19.7	19.7
Model left flank seasonal displacement (mm)	23.0	21.2	19.0	16.9	14.8	12.7
Fitness left flank	25.6%	35.2%	45.1%	51.7%	53.0%	49.1%
Right flank GNSS/GPS seasonal displacement (mm)	19.3	19.3	19.3	19.3	19.3	19.3
Model right flank seasonal displacement (mm)	23.0	21.1	19.0	16.9	14.8	12.7
Fitness right flank	30.5%	39.3%	45.3%	45.4%	40.7%	33.5%

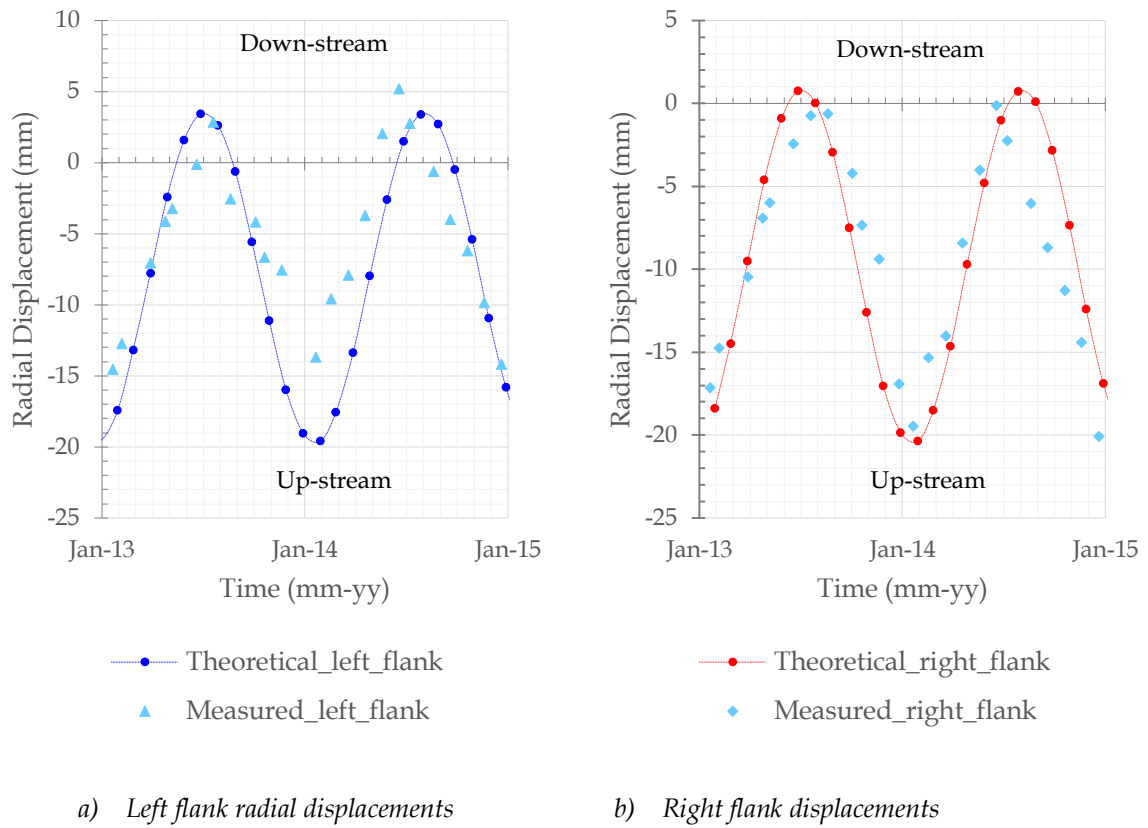


Figure 7.37: Illustration of the best fit plots of the measured and modelled seasonal radial displacement variation

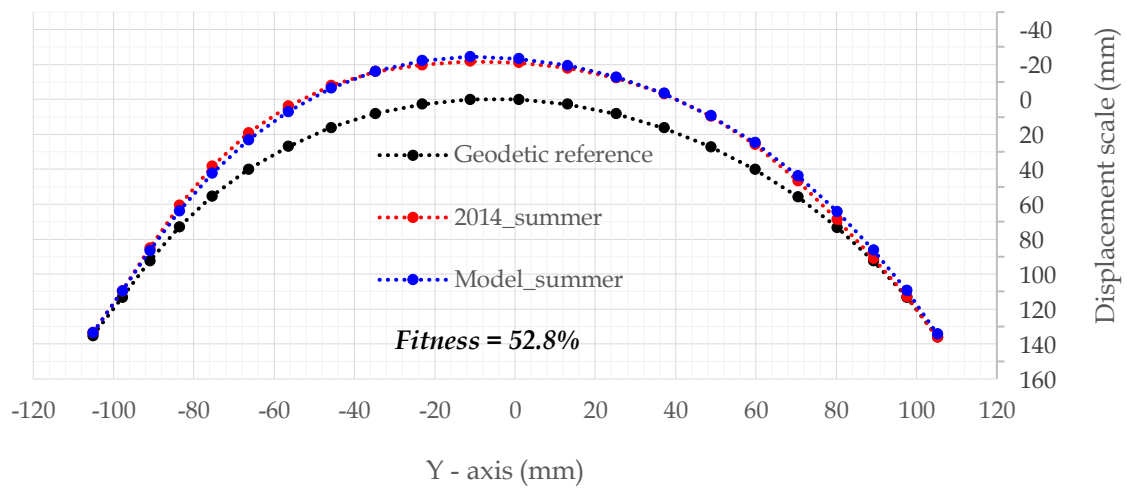


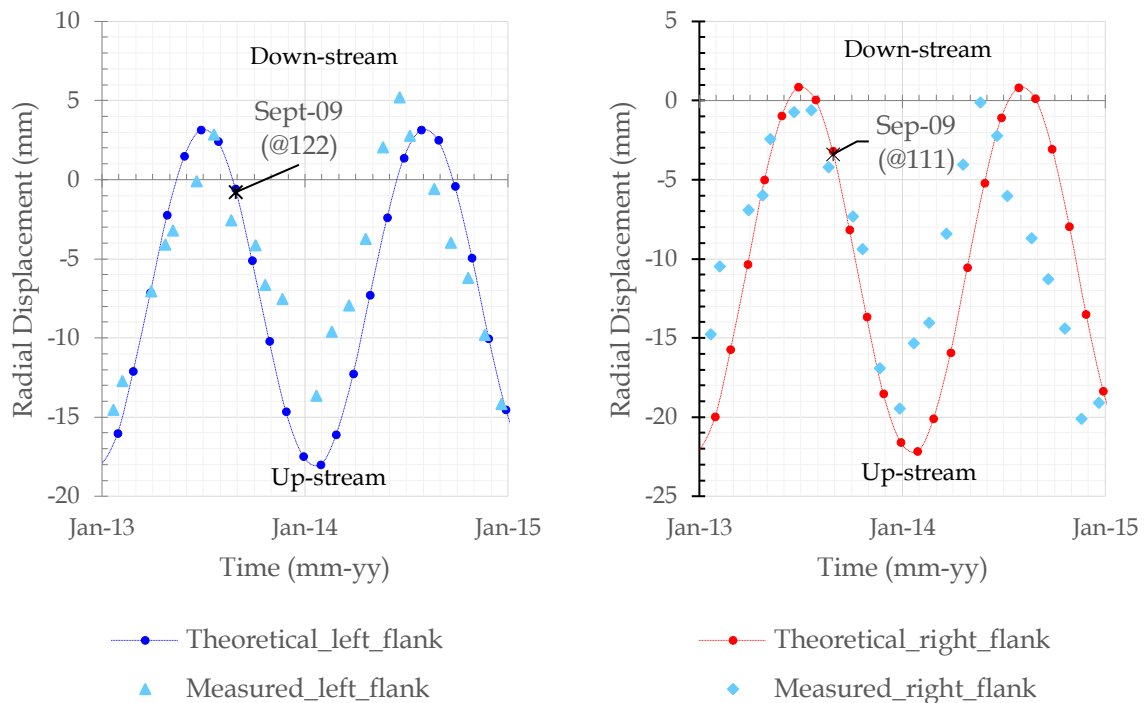
Figure 7.38: Illustration of the best fit plots of the measured and modelled radial displacements

Model Validation

The model validation was conducted using September 2009 measured data (both dynamic measurements (Table 7.2) and static displacements (Figure 7.13)). Table 7.13 shows the dynamic model validation results. There is good agreement between the measured natural frequencies and the FEM determined frequencies, especially for the first two modes.

Table 7.13: Kouga dam model system stiffness and density validation

E_c , dynamic	34.6	w_{ω_i}	Measured natural frequency (Hz)	
ρ_c	2440		Sept 2009 (WL - 137.2)	%difference
Mode 1	3.62	6.74×10^{-2}	3.60	0.54
Mode 2	4.02	9.29×10^{-1}	4.01	0.35
Mode 3	5.50	2.66×10^{-3}	4.70	16.9
Mode 4	6.78	7.88×10^{-4}	6.32	7.28
$\sum_{i=1}^{n_{\text{modes}}} \left(\frac{\omega_i^{\text{EXP}} - \omega_i^{\text{NUM}}(\theta)}{\omega_i^{\text{EXP}}} \cdot w_{\omega_i} \right)^2$		1.84×10^{-5}	$\Sigma = 1.0$	



a) Radial displacement measurement at block 122 b) Radial displacement measurement at block 111

Figure 7.39: Radial geodetic measurements on the left and right flank blocks in September 2009

There is good agreement between the measured and theoretical displacements, confirming that the calibrated model represents the dam under varying environmental and hydrostatic loads. (Figure 7.39). Consequently, the developed and updated Kouga dam model resulted in an accurate and reliable representation of the current state of behaviour of the dam.

7.4 Chapter Summary

In this chapter, a proposed vibro-thermo-deformational FEMU procedure for operational concrete arch dam structures affected by ASR has been presented and successfully implemented in developing an accurate and reliable model for a case study dam structure affected by ASR. The important aspects to highlight include the importance of up-to-date information concerning unknown and/or uncertain parameters that affect the reliability of numerical models and the successful incorporation of AVM data in achieving reliable models in the proposed updating procedure. Additionally, the sensitivity study conclusions in Chapter 6 were validated in this Chapter.

Some key consideration and observations in the implementation of the proposed model updating approach are here summarised:

- i. Updating of the foundation stiffness, wall stiffness and concrete density using ambient vibration measurements. The main objective of this step is to minimise the difference between the measured natural frequencies and, mode shapes and theoretical values (at a given water level). At least, one of the system stiffness parameters (either the wall stiffness, foundation stiffness or their ratio) must be known. Thus, measuring the elastic modulus and concrete density of the dam wall from concrete cores is recommended. At the end of this process, the elastic moduli of the foundation and the wall and concrete density must be identified.
- ii. Updating the thermal behaviour of the dam wall using temperature monitoring data. The main objective of this step is to minimise the difference between measured and theoretical temperatures (both magnitude and phase). The thermal behaviour of the dam is crucial for evaluating seasonal displacements.
- iii. Updating the dam's displacement. This step consists of two aspects, firstly, updating of ASR related parameters (τ_c , τ_1 & $\epsilon(\infty)$) and updating the coefficient of thermal expansion using displacement measurements at a given at a given water level and time of year. The main objective of this step is to minimise the difference

between measured and theoretical displacements. Any tests towards the estimation of the ASR reaction kinetics and/or coefficient of thermal expansion are recommended. It is important to note that in this step, the stiffness of the dam should not be adjusted as much as possible. Any adjustments to the stiffness should not change the dynamic behaviour of the dam.

- iv. Validation of the calibrated finite element model using long-term monitoring data (both static and dynamic). The FE model is used to predict the dynamic properties and displacements of at least two historical measurements.

Evidently, the finite element model should include as much information about the dam-foundation-reservoir system and its interaction with its surrounding as much as possible to minimise any differences between the as-built structure and the theoretical model.

7.5 References

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Chapter 8

CONCLUNSIONS AND RECOMMENDATIONS

8.1 Introduction

The work in this thesis focuses on the structural health monitoring assessment of concrete arch dams including those affected by ASR using finite element (FE) models. The challenge in finite element analysis (FEA) is generating an FE model that mimics the current behaviour and condition of the dam to be trusted and used for further study. This challenge is further complicated by the fact that the operational behaviour of concrete arch dams is complex due to the form of arch dam structures (i.e., geometric characteristics) and the complex interaction between the dam's system components with each other (i.e., dam-foundation-fluid interaction) and with their surrounding through physical or chemical processes (which account for the external actions on dam structures). Finite element model updating (defined as the systematic tuning of model assumptions and parameters till there is agreement between the FE model and measured results) of well-conditioned problems is the process through which representative, reliable and accurate FE models are achieved using dam surveillance data (structural health and vibration monitoring data). Additionally, Structural Health Monitoring and Vibration testing offer insight into the behaviour of concrete arch dams useful in the development of improved FE models, by monitoring the time varying loads and the response of the dam.

The aim of this study was therefore to develop a technically sound, practical, holistic and scientifically based finite element model updating procedure for concrete arch dams affected by ASR using both static and ambient vibration (dynamic) monitoring data, that accounts for the complex interactions and influences of the aforementioned variables resulting in representative, accurate and reliable numerical models. This aim was achieved by addressing the following restated objectives:

- i. To investigate the influence of geometrical properties of arch dams on their dynamic behaviour under ambient conditions.
- ii. To investigate the influence of stress-induced stiffness on the dynamic behaviour of concrete arch dams.
- iii. To identify the most relevant parameters to update to achieve reliable and accurate models for the assessment of the current state of behaviour of concrete arch dams.
- iv. To develop an integrative FE model updating approach for concrete arch dams.

and research questions:

- i. What is the state of the art regarding the structural assessment of ASR affected concrete arch dams?
- ii. How is model updating carried out to account for changes in the internal stress state of a structure?
- iii. What is the current state of practice regarding the incorporation of AVM data in the management and safety evaluation of concrete arch dams?
- iv. Which parameters are the best to adjust when updating FE models of concrete arch dams?
- v. What is the best model updating approach of concrete arch dams including those that are subject to ASR, using ambient vibration measurements?

To achieve the aim and objectives of this study, a deeper understanding of the sensitivity of the variables that affect the behaviour of arch dams is essential. To this end an extensive sensitivity study, based on the theoretical background information derived from literature (Chapter 2 and Chapter 3 which also provided a response to questions (i) and (ii) respectively), to investigate the influence of geometric properties (including the valley type and arch dam classification) (subject of Chapter 5), loads (including ASR and their effects), variation in material properties and boundary conditions (subject of Chapter 6) on the structural behaviour of concrete arch dams was undertaken to facilitate the selection of the best parameters to tune in the model updating process. Based on the findings from this study, a FEMU procedure for concrete arch dam structures affected by ASR was proposed and implemented in updating an FEM model developed for an existing dam affected by ASR (subject of Chapter 7). The modelling approach and development implemented in this study was described in Chapter 4.

Additionally, the current state of practice regarding the incorporation of AVM data in the management and safety evaluation of concrete arch dams (research question (iii)) was addressed in Chapter 4.

The findings, general discussion of the research findings and possibility for further work emanating from this research follows:

8.2 Research Findings

The conclusions presented in this chapter are divided according to the presentation of the results in this thesis as follows:

8.2.1 Characterisation of the Structural Behaviour of Concrete Arch Dams

The observations from the characterisation of the various configurations of concrete arch dams (Chapter 5) are concluded as follows:

Conclusions based on the dynamic behaviour characterisation of arch dams

- i. The full vibration mode shape sequence (including symmetric, antisymmetric, and complex modes) is influenced by the valley type (narrow, wide, U-, V- shaped), arch thickness class (thin, medium-thick & thick) and arch dam curvature type (single or double) indicating the uniqueness of each arch dam configuration's dynamic characteristics.
- ii. The arch dominant vibration mode shape sequence of the various configurations investigated in this study were validated against measured dominant arch vibration mode shape sequences from vibration tests of various arch dams around the world achieving good agreement.
- iii. In a general sense, using the general characteristics traditionally used to classify arch dams, the first five dominant arch vibration mode sequence of an arch dam structure can be reasonably estimated using the observations of this study if no other analytical or measured results are available.
- iv. The arch dominant vibration mode sequence for all V-shaped arch configurations (i.e., SASAS) does not vary based on the valley type and/or arch classification (both the thickness classification and curvature classification) whereas the vibration

mode sequence for the U- shaped arch configurations varies based on the same characteristics.

- v. Keeping all other factors constant, as the crest length (l_c) increases, the natural frequency decreases.
- vi. Keeping all other factors constant, as the base length (l_b) decreases, the natural frequency increases.
- vii. As the crest length (l_c) and/or base length (l_b) decreases or the arch thickness classification ($b_t: h$) increases, the total number of extracted theoretical mode shapes decreases.
- viii. The natural frequency is generally directly proportional to the arch thickness class (i.e., increases with increasing arch thickness class) whilst the rate of change of the natural frequency is a function of the curvature type (i.e., the sensitivity is constant for each single curvature arch configuration with increasing $b_t: h$ whilst the sensitivity increases as $b_t: h$ increases for double curvature arch configurations).
- ix. For the thin classified arch dams whose first vibration mode is the fundamental antisymmetric mode and the second vibration mode the fundamental symmetric mode, as $b_t: h$ increases, the first two mode shapes tend to swap (i.e., the fundamental symmetric mode becomes the first vibration mode with the fundamental antisymmetric mode as the second mode of vibration).

Conclusions based on the static behaviour characterisation of arch dams

- i. The static behaviour of concrete arch dams is highly variable compared to the dynamic behaviour. Thus, it is difficult to estimate the static behaviour response of concrete arch dams based solely on the geometric characteristics of concrete arch dams without measurements or performing numeral analysis to get a sense of a structure's static response.
- ii. The deflected shape of an arch dam along the crest length due to the hydrostatic load extends further towards the foundation more in single curvature dam configurations as compared to double curvature configurations and also for U-shaped valley configurations as compared V-shaped valley configurations.
- iii. Keeping all other factors constant, as the crest length (l_c) increases, both radial and vertical displacements due to both thermal and hydrostatic loading increase. The

increase in the displacements due to the hydrostatic loading is about four times the increase due to thermal loading (on average) as the crest length increases.

- iv. The rate of change in displacements with increasing b_t : h varies depending on the displacement type, valley characteristics (both the shape (U or V and width (narrow or wide) of the valley) and arch classification (both the thickness classification and curvature classification) for all the investigated valley types.
- v. Keeping all other factors constant, as the crest length (l_c) increases, the change in displacements varies based on displacement type, valley characteristics (both the shape (U or V and width (narrow or wide) of the valley) and arch classification (both the thickness classification and curvature classification) for all the investigated valley types.

8.2.2 Sensitivity Analysis of the Structural Behaviour of Concrete Arch Dams

The observations from the sensitivity analysis of the structural behaviour of the selected arch dam configurations (Chapter 6) are concluded as follows:

Conclusions drawn from the structural response sensitivity to variation in selected service loads

- i. The rate of change (i.e., sensitivity) of the natural frequencies to the applied thermal loading is constant for each arch configuration but varies across different configurations based on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. However, the magnitude of the sensitivity of the natural frequencies to thermal loading is small but could result in observable variations in natural frequency though not significant.
- ii. The rate of change (i.e., sensitivity) of the displacements to the applied thermal loading is constant and equal to about 10% for all arch dam arch configurations considered in this study irrespective of type of displacement, valley type, arch thickness, and arch dam curvature classifications. The magnitude of the sensitivity of the displacements to thermal loading is sufficiently significant to affect the static response of concrete arch dams.
- iii. The rate of change (i.e., sensitivity) of the natural frequencies to volumetric expansion is constant for each arch configuration but varies across different configurations based on the vibration mode, valley type, arch thickness, and arch

dam curvature classifications. However, the magnitude of the sensitivity of the natural frequencies to volumetric expansion is small but could result in observable variations in natural frequency though not significant.

- iv. The rate of change (i.e., sensitivity) of the displacements to volumetric expansion is constant for each arch configuration but varies across different configurations based on the type of displacements, valley type, arch thickness, and arch dam curvature classifications. Additionally, the magnitude of the sensitivity of the displacements to volumetric expansion is sufficiently significant to affect the static response of concrete arch dams.
- v. The sensitivity of natural frequency to hydrostatic loading decreases as the water level decreases with the natural frequency approaching a constant value (the natural frequency of the dam empty). The total change and the rate of change (i.e., sensitivity) of the natural frequencies to hydrostatic loading varies across different configurations based on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. The reservoir level at which no further change in frequency is observed varies between an H_w/H value of 0.15 & 0.7 depending on the arch dam configuration and mode shape. However, the magnitude of the sensitivity and/or total change of the natural frequencies due to hydrostatic loading is small but could result in observable variations in natural frequency though not significant.
- vi. The sensitivity of displacements to hydrostatic loading decreases as the water level decreases with the displacement approaching a constant value (the displacement of the dam empty). The total change and the rate of change (i.e., sensitivity) of displacements to hydrostatic loading varies across different configurations based on the type of displacement (radial or vertical), valley type, arch thickness, and arch dam curvature classifications. The reservoir level at which no further change in displacements is observed varies depending on the arch dam configuration (including the valley type, arch dam thickness and curvature classifications) and type of displacement. Additionally, the magnitude of the sensitivity and/or total change in displacements due to hydrostatic loading is sufficiently significant to affect the static response of concrete arch dams.
- vii. The sensitivity of natural frequency to water level (added water effect) decreases as the water level decreases with the natural frequency approaching a constant

value (the natural frequency of the dam empty). The total change and the rate of change (i.e., sensitivity) of the natural frequencies to the water level varies across different configurations based on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. The reservoir level at which no further change in frequency is observed varies depending on the arch dam configuration and mode shape. The magnitude of the sensitivity and/or total change of the natural frequencies due to the water loading is sufficiently significant to affect the dynamic response of concrete arch dams.

- viii. The modal parameters of concrete arch dams are also affected by the boundary condition and water level evidenced by the differences in the mode shape sequences and natural frequency values for different boundary conditions and/or water level.

Conclusions drawn from structural response sensitivity to variation in material properties

- i. The sensitivity of natural frequency to the system stiffness (E_f/E_c) decreases with the natural frequency approaching a constant value (the natural frequency assuming a fixed boundary condition) as the stiffness modulus ratio increases. The total change and the rate of change (i.e., sensitivity) of the natural frequencies to the system stiffness varies across different configurations based on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. Additionally, the magnitude of the sensitivity and/or total change of the natural frequencies due to the system stiffness is sufficiently significant to affect the dynamic response of concrete arch dams.
- ii. The sensitivity of displacements to the system stiffness decreases with the displacement approaching a constant value (corresponding to the reference displacements assuming a fixed boundary condition) as the stiffness modulus ratio increases. The total change and the rate of change (i.e., sensitivity) of displacements to the system stiffness varies across different configurations based on the displacement type (radial or vertical), valley type, arch thickness, and arch dam curvature classifications. Additionally, the magnitude of the sensitivity and/or total change of the displacements due to the system stiffness is sufficiently significant to affect the static response of concrete arch dams.

- iii. The sensitivity of natural frequency to the system Poisson's ratio (ν_f/ν_c) varies depending on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. Additionally, the total change in natural frequencies due to the system Poisson's ratio (ν_f/ν_c) varies across different configurations based on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. However, the magnitude of the sensitivity and/or total change of the natural frequencies due to the system Poisson's ratio is small but could result in observable variations in natural frequency though not significant.
- iv. The displacements generally decrease as the system's ν_f/ν_c increases. The sensitivity of the displacements to the system's ν_f/ν_c varies depending on the displacement type, valley type, arch thickness, and arch dam curvature classifications. Additionally, the total change in displacements due to the system Poisson's ratio (ν_f/ν_c) varies across different configurations based on the displacement type, valley type, arch thickness, and arch dam curvature classifications. The magnitude of the sensitivity and/or total change of the displacement maybe sufficiently significant to affect the static behaviour of an arch dam depending on the displacement type, valley type, arch thickness, and arch dam curvature classifications.
- v. The rate of change (i.e., sensitivity) of the natural frequencies with increasing concrete wall density varies slightly as the density increases (with a sensitivity of about 0.02% assuming a linear relationship) for all arch dam arch configuration considered in this study irrespective of the mode shape, valley type, arch thickness, and arch dam curvature classifications. The magnitude of the sensitivity and/or total change in natural frequencies are sufficiently significant to affect the dynamic behaviour of concrete arch.
- vi. Density does not affect the static response of concrete arch dams.
- vii. The rate of change (i.e., sensitivity) of the natural frequencies to variation in the coefficient of thermal expansion (α_c) is constant for each arch configuration but varies across different configurations based on the vibration mode, valley type, arch thickness, and arch dam curvature classifications. However, the magnitude of the sensitivity of the natural frequencies to variation in the coefficient of expansion is small but could result in observable variations in natural frequency though not significant.

- viii. The rate of change (i.e., sensitivity) of the displacements to variation in the coefficient of thermal expansion (α_c) is constant and equal to about 10% for all arch dam arch configuration considered in this study irrespective of type of displacement, valley type, arch thickness, and arch dam curvature classifications. The magnitude of the sensitivity of the displacements to thermal loading is sufficiently significant to affect the static response of concrete arch dams.
- ix. Based on these observations the best parameters to update when considering the dynamic behaviour of concrete arch dams using ambient vibration monitoring data is the system stiffness and concrete density whilst accounting for the fluid structure interaction due to the water load.
- x. The stress stiffness due to all the service loads considered in this study, particularly ASR volumetric expansion related stress stiffness does not significantly affect the dynamic properties of these structures. However, these service loads affect the static behaviour of concrete arch dams significantly.
- vi. The parameters that could be updated when considering the static behaviour of concrete arch dams include the system stiffness, the coefficient of thermal expansion and the system's Poisson's ratio whilst accounting for thermal loads, hydrostatic loading, and ASR load effects.

8.2.3 Multi-step model updating approach

The observations from the implementation of the proposed multi-step model updating approach (Chapter 7) are concluded as follows:

- i. The proposed multi-step finite element model updating approach proposed in this study was implemented in updating a numerical model of a case study dam affected by ASR achieving good agreement between the observed current behaviour and the theoretically determined behaviour of the structure. Consequently, the developed model was successfully updated to achieve a representative, reliable and accurate model of the existing structure that can be used for structural health monitoring of the structure.
- ii. The proposed finite element model updating approach consists of four steps, namely: (i) updating of the system stiffness and concrete density using ambient vibration measurements, (ii) updating of the thermal behaviour of the dam wall

using temperature monitoring data, (iii) updating of the dam's displacements by considering two aspects that affect the static behaviour of concrete arch dams, namely ASR (updating the reaction kinetics parameters) and thermal displacements (updating the coefficient of thermal expansion and the system's Poisson's ratio), (iv) validation of the calibrated FE model using long-term monitoring data for at least two historical measurements.

- iii. Additionally, the finite element model should include as much information about the dam-foundation-reservoir system and its interaction with its surrounding as much as possible to minimise any differences between the as-built and/or current-state of the real structure and the theoretical model. For example, the material properties that can be measured, should be measured to reduce the number of unknowns in modelling and updating process.
- iv. Also, for the proposed integrative model updating approach, any change in any of the parameters should not significantly change the accuracy of any of the updating steps for parameters that are sufficiently significant to affect multiple steps such as the system stiffness.

8.3 General discussion of the findings

It is evident from the findings of this study that the structural behaviour (both static behaviour and dynamic behaviour to varying degrees) of concrete arch dams is subject to the dam system's characteristics (i.e., geometric characteristics and material properties of the dam system) and its interaction(s) with its surroundings (i.e., loading on the structure).

Although the dynamic behaviour of each concrete arch dam is unique to that structure based on its defining characteristics, the dynamic behaviour of concrete arch dams provides much clear trends and distinctions between different arch dam configurations as compared to the static behaviour of concrete arch dams. However, the characteristics which have traditionally been used to classify arch dam structures do not consider some geometric parameters such as the crest thickness of the structure, which affects the overall system mass and possibly the stiffness of the dam system. Consequently, it might be worthwhile to classify the dynamic dam behaviour of concrete arch dams to comprehensively account for all parameters that contribute to the overall dam system stiffness and mass.

Based on the findings above, the development of internal stress due to ASR does not significantly affect the dynamic behaviour of concrete arch dams. Consequently, the evaluation of the extent of ASR reaction by tracking the stress development in an affected arch dam structure using dynamic behaviour analysis is unlikely for now. Thus, the question of how best to evaluate the extent of ASR reaction and its effect on real structures (without losing the context within which the structure exists) still remains. The reaction kinetics for ASR estimated using the proposed FEMU procedure need to be validated using laboratory accelerated tests

Lastly, the best parameters to update when considering the dynamic behaviour of concrete arch dams using ambient vibration monitoring data is the system stiffness and concrete density whilst accounting for the fluid structure interaction due to the water load. Whilst the parameters that could be updated when considering the static behaviour of concrete arch dams include the system stiffness, the coefficient of thermal expansion and the system's Poisson's ratio whilst accounting for thermal loads, hydrostatic loading, and ASR load effects. The current state of practice is to consider the dynamic behaviour and static behaviour of these structures independently of each other. Considering both the dynamic and static response together as proposed in this study means that the interaction of the different behaviour and the different properties of the arch dam system need to be clearly understood.

8.4 Recommendations for further work

Based on the conclusions and discussion thereof, some recommendations for further study include:

- i. Some geometric parameters such as the crest thickness that affect the dynamic behaviour of concrete arch dams are not considered in the current classification of arch dams. To improve the estimation of the modal parameters of arch dam structures, the investigating of the influence of these parameters is required.
- ii. Another aspect, building on from the previous recommendation is the possibility of redefining the classification of concrete arch dams based on the dynamic behaviour of these structures as the current general classification of concrete arch dams seems to be based mainly on the static behaviour and or site classification, which might not be the best criteria in describing the dynamic behaviour aspects of concrete arch dams.

- iii. Additionally, the classification valley types: narrow and wide, do not account for valley types with a l_c/h between 2 and 5, a range in which some concrete arch dam structures have been built. Using the results from this study inferences can be made as to the dynamic behaviour of these structures. However, for a comprehensive, extensive, and reliable characterisation for these structures, further investigations for as many of the valley types could be useful in further providing a systematic characterisation of the dynamic behaviour of arch dam structures. Additionally, such an investigation will be crucial for the previously stated recommendation.
- iv. Another aspect on the characterisation of the behaviour of concrete arch dams that could be useful that requires further research is to investigate if there is a relationship between the dynamic behaviour and the static behaviour as a function of an arch dam's configuration characteristics.
- v. In line with investigating possible on-site methods of evaluating the extent of ASR reaction and degradation, the challenge in the successful implementation of dynamic testing to capture the degradation of ASR on the dynamic characteristics of concrete arch dams seems among other possibilities, to lie in the limitation of the extent of vibration testing for ASR affected concrete arch dam structures. Further study on a real structure, to investigate the effect of ASR on the modal parameters through extensive ambient vibration testing that is carried out at several levels and not just at the crest (subject to accessibility) may be necessary.
- vi. The reaction kinetics for ASR estimated using the proposed FEMU procedure needs to be validated using laboratory accelerated tests. A laboratory-based investigation to confirm these parameters is required.
- vii. The proposed model updating procedure can be adjusted by including other aspects not included in this study such as loading effects from silt, creep, or seismic analysis specific aspects. To this end, further sensitivity studies will be essential in the incorporation of any further aspects that affect the behaviour of some existing arch dam structures.
- viii. The next step from the overall findings of this study is to automate the model updating process using dam surveillance data with the aim of achieving real-time updating of the model where possible (in-line with the current digital trends of the management and assessment of structures). To achieve this, a number of investigations from both the modelling and dam surveillance end to make the auto-

matic interaction between the two aspects possible may need to be carried out.

8.5 List of Publications

- Moyo, P & Nyoni, B. 2020. *Climate Change Impacts on the Safety of Concrete Arch Dams in South Africa*. (WRC Project No. K5-2749). Gezina, South Africa: Water Research Commission.
- Moyo, P., Nyoni, B.R. & Kabani, M. 2019. Thermo-Mechanical Modelling of Arch Dams for Performance Assessment. *Proceedings of the Annual SANCOLD 2019 Conference on “Sustainable long-term dam infrastructure development and management”*. 6-8 November 2019. Benoni, Gauteng. 319-328.
- Nyoni, B.R. 2018. *Thermo-Chemo-Mechanical Modelling of Hydropower Structures Affected by Alkali Silica Reaction*. MSc. Eng. Thesis. University of Cape Town.
- Nyoni, B.R. 2014. *Dynamic Behaviour Assessment of Concrete Arch Dams using the Finite Element Modelling Technique*. BSc. Eng. Thesis. University of Cape Town.
- Moyo, P., Nyoni, B.R. & Lennard, C. 2018. Effects of Climate Change on Dam Safety in South Africa. *Proceedings of the Annual SANCOLD 2018 Conference on “Sustainable dam engineering in an ever-changing world”*. 7-9 November 2018. Somerset West, South Africa. 157-165.
- Nyoni, B.R. & Moyo, P. 2023. Dynamic behaviour characterisation of various configurations of concrete arch dams. *Proceedings of the 91st ICOLD Annual Meeting on Management for Safe Dams*. 13-14 June 2023. Gothenburg, Sweden. 1390-1398.

Appendix

ADDITIONAL RESULTS TABLES

This section provides the additional results referred to in the main thesis body.

Chapter 5 Result Tables

Table A.1 Relative difference in natural frequencies of single curvature vs double curvature configurations (as a percentage (%))

Valley Shape	Arch class	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9	Mode 10	Average	Standard deviation
 Narrow U	thin	13	10	16	14	16	16	12	9	11	8	13	3
	medium-thick	1	-5	-4	-4	-6	-6	-7	-7	-7		-5	2
	thick	2	-4	-4	-5	-5	-6	-6				-4	3
 Narrow V	thin	15	26	26	24	25	22	27	21	19		23	4
	medium-thick	3	2	2	1	1	0	1				1	1
	thick	5	3	3	1	2						3	1
 Wide V	thin	11	17	23	30	32	33	33	14	31	20	24	8
	medium-thick	0	-2	1	2	3	3	3	-6	3		1	3
	thick	2	1	2	3	3	3	3	-5	3		2	3
 Wide U	thin	4	0	20	14	26	31	33	32			20	12
	medium-thick	-5	-2	1	0	1	1	1	1			0	2
	Thick	2	-1	1	1	2	2	2	-7	2	-6	0	3

Chapter 6 Result Tables

Table A.2 Percentage difference in natural frequency ratios between the full dam load case and empty dam (%)

Type of Load	Arch class	Arch Mode	Thick arch narrow U	Thin arch narrow U	Thin arch narrow V	Thin arch wide U	Thick arch wide U
hydrostatic	single	1 st -S	-0.00045	-0.00388	-0.00332	-0.00401	-0.00013
		1 st -A	-0.00051	-0.00305	-0.00236	-0.00391	-0.00035
	double	1 st -S	-0.00051	-0.00249	-0.00193	-0.003	-0.00024
		1 st -A	-0.00045	-0.00195	-0.00139	-0.00262	-0.00037
hydrodynamic	single	1 st -S	-0.466	-0.415	-0.415	-0.214	-0.313
		1 st -A	-0.445	-0.325	-0.360	-0.221	-0.294
	double	1 st -S	-0.405	-0.361	-0.356	-0.181	-0.257
		1 st -A	-0.352	-0.294	-0.339	-0.177	-0.237

A - antisymmetric mode

S - symmetric mode