01/FT.01/SKRIP/05/2008

DYNAMIC PERFORMANCE OF DEMOUNTABLE GRANDSTAND STADIUM USING SANDWICH PLATE SYSTEM

FINAL ASSIGNMENT

By

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THIS FINAL ASSIGNMENT IS PROPOSED TO FULFILL THE REQUEREMENT TO BE A CIVIL ENGINEER

CIVIL ENGINEERING DEPARTMENT

ENGINEERING FACULTY UNIVERSTAS INDONESIA

DEPOK

2nd SEMESTER 2007/2008

FINAL ASSIGNMENT AUTHENTICITY STATEMENT

I declare that this final assignment entitled :

DYNAMIC PERFORMANCE OF DEMOUNTABLE GRANDSTAND STADIUM USING SANDWICH PLATE SYSTEM

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Is finished to fulfill the requirements to be a civil engineering from Civil Engineering Department-Engineering Faculty Universitas Indonesia. This final assignment has tested in final tes conference on May 26th, 2008 and declared fulfilling validity requirements as a final assignment in Civil Engineering Department-Engineering Faculty Universitas Indonesia.

Depok, May 28th 2008

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ABSTRACT

The need to build a demountable grandstand stadium for Olympic Games purposes has been a motivator to develop a grandstand using new materials and technology. The Sandwich Plate System (SPS) is a new technology in the civil engineering field. Due to its unique composite system which comprises two metal plates separated by an elastomer core and its light weight materials, the sandwich plate system is adopted in designing a demountable grandstand stadium. The idea is using the SPS with Fibre Reinforced Concrete (FRC) and Polystyrene instead of steel plates and Polyurethane because the use of concrete will not cause noise which the steel might generate when people jump and walk over it. The other reason is regarding to its costs; fibre reinforced concrete and Polystyrene have cheaper prices than steel and polyurethane. The dynamic performance of the grandstand is investigated as the stadium will be subjected to dynamic crowd actions such as when the stadium is used for sports events or music concerts. Therefore, the walking and the rhythmic criterion are investigated to find the appropriate materials and dimensions which will satisfy the dynamic performance criterion.

Keywords: Sandwich Plate System, demountable, grandstand stadium, stadium riser, dynamic, fibre reinforced concrete, polystyrene.

PREFACE

In general, the aim of this thesis is as a requirement to be a civil engineer at civil department, Universitas Indonesia. In particular, this project has given the author a very valuable experience and exposure in research work.

First of all the author would like to express gratitude to the God the infinite for His blessing and guidance. The author would like to express gratitude to her supervisors, Professor David Thambiratnam and Professor Nimal Perera for their patient guidance, continuous support and excellence advice during this thesis. The author would like also thank QUT researcher, Sandun de Silva and also her senior Riandy Bhaskara for their great teachings and guidance in the development of the software. Her friend Widi Baskoro for his support and also to her friends: Cindy Kaharmen, Dhierda Utiya, Ridho Sinuraya, Baratha Gunawan and Erwin Mulyono. Finally, the author would like to thank her partner James Hargreaves for his support and help.

The author realized that this thesis is not perfect; therefore, comments and criticisms are welcome to improve this thesis and to give benefits to all community in general and to structure world in particular.

Depok, May 2007

Jessie

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1 INTRODUCTION

1.1 Background

London as the host of Olympic Games 2012 has their main reason for taking on hosting the Games. However, the reasons of other cities' that hosted the Games in the past, it is to gain international prominence and thereby increase international investment and business opportunities and also expected is some economic benefits from the Games (Dann 2004). One of London's strategies is to build a demountable stadium to fulfil their long-term plan which is under the environmental and social issues. Their plan is to transform one big stadium for Olympics purposes to a few small stadiums once the Olympics is finished. The important part of the stadium that could be demountable is the grandstands. Therefore, the grandstand must be designed to fulfil the demountable criterion.

The new engineering technology called Sandwich Plate System is an alternative way to build the demountable grandstand stadium. Its performance which is light in weight is the important criterion when it is used as demountable material. Not just lighter in material, but outperforming concrete in dynamic and static performance test; this material will deliver substantial cost savings in substructure construction and total stadium erection times (D. Braun 2002). Although it is new to the civil engineering field, it has the added benefit of being tried, tested and regulated over the last decade, in some of the most unforgiving environmental conditions on earth, and by some of the strictest regulatory bodies (Turner 2003).

The Sandwich Plate System is a technology heralded by engineers and regulators as the first maritime and civil engineering material for more than 100 years. SPS is a composite material comprising two metal plates separated by an elastomer core, which transfers shear between each plate, eliminates the need for stiffeners and precludes local faceplate buckling. The thickness of the composite elements can be modified to meet the needs of each application. SPS is an excellent alternative to reinforced concrete and delivers high stiffness to weight ratio, simple fabrication, improved performance and increased safety (Turner 2003)

From the environmental point of view, the demountable grandstand stadium using a composite material could become an alternative choice in material selection. Fibre reinforced concrete combined with polystyrene is a smart technique to overcome the environmental issues as well as other issues such as strength and cost. Since the polystyrene is a type of plastic that can be reusable, it will not affect the environment.

1.2 Problem Definition

Long-term strategies have been planned by London as the hosting city of Olympic Games 2012. Thus, much of the planning was intended to maximize the lasting benefit to the city. London is planning to build a non-permanent Olympic stadium that would provide a valuable purpose after the Games. Their strategy was to transform a huge Olympic stadium into a few small stadiums after the Games.

Since the important part of the stadium that could be demountable is the grandstand, the grandstand should be designed to meet the demountable criterion. An alternative material for the grandstand stadium is required while the material should be light in weight for the demountable purpose. When concrete as the conventional material for grandstand stadium could not fulfil the lightweight criterion, Sandwich Plate System overcomes these difficulties and at the same time delivers numerous advantages over concrete. It has been proved that when formed into stadium risers; SPS is 73% lighter than traditional concrete (Turner 2003).

The composite materials for sandwich plate system that have been developed and used are steel plates and polyurethane elastomer core. The SPS polyurethane elastomer core is a natural damping component which provides an effective vibration-damping layer that is superior to conventional all-steel stiffened plate or all-concrete construction (IE). The use of polyurethane as a core material has provided the plates with high damping ratio and high natural frequency. On the other hand, the material cost of polyurethane is relatively high compared with other plastic materials. Therefore the use of polystyrene is being considered as the substitution of the elastomer core material. Furthermore, fibre reinforced concrete is used despite of steel plates.

1.3 Aims and Objectives

The aims of this project are to investigate the dynamic performance of demountable grandstand stadium for different cases of cross sections and constraint conditions and to develop the suitable dimensions and materials using sandwich plate system technology.

The objectives if this investigation were as follows:

1. To investigate the likelihood of each host city on hosting the Olympic Games and their strategies for hosting the Games.
2. To investigate the dynamic performance of demountable grandstand stadium using polystyrene and fibre reinforced concrete as the SPS materials.
3. To develop a suitable materials and dimensions of the grandstand stadium using Sandwich plate system.

Although the overall thesis is focused mainly on the literature research, a series of calculations and analysis are involved.

1.4 Methodology

To achieve the objectives, the following research method was undertaken:

1. General studies and problems.
2. Literature review of behind the Olympic Games, Sandwich Plate System, materials used for the grandstand, dynamic performance of grandstand structure and associated researches.
3. Manual calculations and computer analyses prior to predict the natural frequency.
4. Evaluation of the manual calculation results and suggestions for the best grandstand cross–section and materials.

1.5 Thesis Outline

This research is divided into four sections as follows:

1. Literature review

   This chapter is the foundation of this thesis. It explores the development, benefits, material properties of SPS, structural dynamics and any relevant information of dynamic performance of grandstand stadiums. Additionally the analysis methods will also be discussed.

2. Investigation

   This section will summarize the specific information from the literature review above for investigation purposes. Analysis will be performed and includes manual calculations and computer analysis of the design model.

3. Results and Analysis

   Following the investigations is the analysis of the results of grandstands manual calculations and computer analysis. This chapter presents the results and comparison between grandstand cross sections of its natural frequency and peak accelerations.

4. Conclusion and Recommendations

   The last chapter summarizes the key results from the investigations and obtains conclusions about the most effective grandstand's cross section. Some recommendations will also be provided for future research.

2 LITERATURE REVIEW

The basis of the following literature review is to present the background information of the reasons for taking on hosting the Olympic Games, the hosting cities long-term strategies, the sandwich plate system technology and applications that have been developed and the materials for new sandwich plate system which are fibre reinforced concrete and polystyrene. The dynamic performance of grandstand stadium is also reviewed in this part.

2.1 Behind the Olympics

Hosting the Olympic Games is a big opportunity for every city where the city becomes a representative of the whole country, which means that national pride is an important factor in supporting the Games. Although each city has different reasons for taking on hosting the Games, they have the same main reason, referring to Atlanta and Sydney, it was to gain international reputation and thereby increase international investment and business opportunities (Dann 2004). Although the international recognition is important, the real objective is to raise awareness of the city among businesses and investors and encourage them to think about the city in a new light, so they will bring more branch offices, conventions, and international investment to the city (Dann 2004).

The other reason for hosting the Games is to bring the community together. Some people believe in the power of the Olympic spirit to inspire and unite; where some want to overcome racial and ethnic tensions, and some just want to galvanize people around a common goal and get something done. Despite of those reasons, many seemed to believe that the experience of the Olympic Games would have a lingering impact on the cohesiveness of the citizenry.

Furthermore, both cities expected some economic benefit from hosting the Games and the opportunity to clean up a problematic area of the city (Dann 2004). Atlanta and Sydney have different strategies which was the degree to which they planned for the long term. They concentrated the new sporting venues for the Games in the areas with fewest resources, a small area of downtown Atlanta and the Western suburbs of Sydney.

2.1.1 Atlanta Olympics 2004

Atlanta's strategies for hosting the Games were to build permanent facilities only if there was an existing demand for them and planned those facilities from the beginning based on how they would be used after the Games were over (Dann 2004). The commitment to build nothing permanent that would not serve a valuable purpose after the Games less to a reasonably uneventful transition for the venues into their post-Games forms. Although some reconstruction was required, such as the conversion of the Olympic Stadium's into Turner field, it had been part of the original plan. Thus, nearly all the venues are still getting good use.

In Atlanta, the impending rush of visitors to the downtown gave urgency to revitalization of inner-city neighbourhoods. Those were chosen based on their preparedness and their proximity to venues of tourist sites, and public money was directed by the Corporation for Olympic Development in Atlanta to those neighbourhoods. Their efforts served as a catalyst for further investment and an example for revitalization efforts in other neighbourhoods. In the Atlanta neighbourhood of Summerhill, redevelopment added many single-family homes, and was successful in making Summerhill a more desirable place to live (Dann 2004)]. With the new houses that have filled in vacant lots and improved sidewalks, Summerhill is a different place than it was 15 years ago. Summerhill looks like a very pleasant residential community, a good place to raise children, convenient to downtown but with the luxury of private homes and yards. Unfortunately, that success may lead to gentrification and displace some of the original residents (Dann 2004).

Centennial Olympic Park is another investment in downtown, which was built as a gathering place for the Games. The park is to be used later as a green space in what had been a rundown area of the city. The area has been transformed, and new development continues around the edges of the park. Downtown revitalization continues throughout

Atlanta, although the Olympic investments are just one factor among many driving that transformation (Dann 2004).

2.1.2 Sydney Olympics 2000

Sydney is trying to convert the sports complex they built into a vibrant, useful district and an asset to the surrounding communities. While in Atlanta, the results of the venues were integrated into the fabric of the city relatively quickly and seamlessly (Dann 2004). On the other hand, in Atlanta, there is almost no indication that the Games were held there, while in Sydney's Olympic Park provides a destination for tourists who want to see where the Olympics were held.

Sydney was focused on making the best Olympics possible, and was being pressured by sporting organizations to build permanent facilities. As a result, Sydney built many permanent venues that were only needed during Games, which does not appear to have been a good investment for New South Wales financially. In Sydney, the Western suburbs not only had less money and fewer services than the rest of Sydney area, but were also heavily polluted in some areas. The area around Homebush Bay had been used as a munitions dump, a slaughterhouse, and a dumping ground for toxic industrial chemicals. That site was selected for the Olympic park, which would include a majority of the new venues and the Athlete's Village. Remediation had been planned for a long time, but finally began in earnest when Sydney won the bid. The contaminated soil has now been contained, wetlands have been restored, and additional parklands have been created. That transformation is a major success of the Games and also helped improve property values in the area immediately around the park (Lenskyj). Furthermore, because Sydney's bid included a strong commitment to the environment, the facilities built in the Olympic Park were built according to Environmentally Sustainable Design (ESD) codes.

2.2 Sandwich Plate System

2.2.1 Technologies

Sandwich Plate System (SPS) is a new material technology developed using two relatively thin steel plates with a second material of polyurethane elastomer core. The Sandwich Plate System was created by Dr. Stephen Kennedy which was trying to create a new composite material that suitable for maritime environment. Steel as the shell material will be subjected to high impacts; therefore the core material should be strong enough to withstand the pressures of the impacts. However, the core should be flexible enough to be able to pass shear from one steel plate to the other.

The use of polyurethane elastomer core could eliminate the need of stiffeners and associated joints because this core provides continuous support to the plates, thus structurally equivalent to a stiffened steel plate (IE). Furthermore, local plate bucking is also prevented at the same time. Another advantage of SPS, by changing the thickness of the sandwich elements, the stiffness and the strength of the sandwich plate system can be tailored to meet specific structural performance criteria. In flexure condition, the plates act as flanges and the core as the web.

The engineered plastic, polyurethane elastomer core was developed by Elastogran GambH in accordance with Intelligent Engineering's material characterization specification for the anticipated extremes of the full ranger operating temperatures between -45^{o}C and +100^{o}C (D. Braun 2002). Furthermore, the material properties such as density, tensile strength, compressive strength, shear modulus and Poisson's ratio have been verified in accordance with the ASTM and DIN standards.

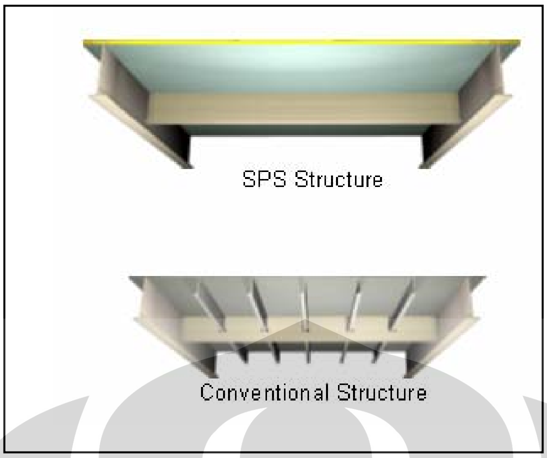

Figure 2.1. Sandwich Plate System

Strength and other performance characteristic have been tried, tested, established and quantified over the last decade. Form testing that includes stocky columns and more slender columns, Sandwich Plate System could obtain the plastic moment capacity or compressive strength without local buckling, delamination of the faceplates or any other local failure modes generally associated with laminates (D. Braun 2002).

2.2.2 Benefits

Some benefits of the use of Sandwich Plate System according to Intelligent Engineering are:

- Simplicity. SPS eliminates the need for stiffeners; therefore the SPS structures are less complex and time consuming than the all-steel equivalents. Moreover, SPS shortens the labour intensive erection process, such as plate assembly, coatings, insulation and fit-out. In civil engineering, SPS reduces on-site schedules by allowing significant prefabrication, avoiding cast-in-place processes and reducing inter-trade complexity. In terms of less risky repairs, SPS Overlay avoids the need to remove and replace the existing structure. SPS Overlay saves three quarters on the downtime, avoids the risk of damage to services and coatings below the structure.

- Performance. SPS have four important performances that over the normal concrete or steel structures. First, improved fatigue and corrosion resistance which SPS structures eliminate the majority of fatigue and corrosion prone details. The second performance is super performance under in-service loads. In SPS structures, the elastomer core, which is polyurethane, dissipates strain energy over a large area, reducing load concentrations that lead to permanent deformations and crack. SPS offer more robust structures with a longer working life. Reduced weight and through-thickness is the another SPS performance. SPS structures have lower substructure costs, increase internal volume and allow extra floors in height restricted structures. The last performance is dampened structure which is the performance that the visco-elastic polyurethane core allows SPS plates to provide significant damping to structure borne vibration and noise.

- Safety. Engineers are allowed to provide heightened safety across an entire structure rather than adding structure or coatings to particularly sensitive areas. A structure is more likely to survive a major event because the protection itself is naturally embedded between the two face plates. Furthermore, SPS acts as an effective fire barrier without the need for the external insulation typically required on steel structures. The other safety issue that SPS could overcome is increased resistance to accidental or extreme loads. SPS structures naturally resist crack propagation have fewer impact sensitive connection details and are able to absorb greater loads through plastification, thus transferring less load to the primary structure. In the case of extreme load events, SPS offers substantial improvements in safety and a reduction in environmental risk.

- Security. Not only safety, but SPS also have a number of built-in security benefits that are of growing importance in light of today's heightened security environment. Those two features are built-in blast and ballistic resistance. SPS structures can naturally resist crack propagation, have fewer impact sensitive connection details and are able to absorb greater loads through plastification and thus transferring fewer loads to the primary structure. In addition, SPS stop projectiles at shorter strike ranges and higher attack angels than steel plate. Different from the normal protection which is added around areas of high risk, SPS has built-in protection across the entire structure. Furthermore, the structure is more likely to survive a major event as the protection is naturally embedded between the two face plates.

- Economics. Due to the simplification of SPS structures, the construction costs could be reduced by reduces support structures, shortens erection schedules and reduces downtime due to repair schedules. For the repair schedules, SPS Overlay can reduce repair schedules by 75%. SPS can increases operating profits by increasing cargo volumes, increasing charter incomes, lower fuel consumption, and reducing through-life maintenance costs. Moreover, in spite of SPS have lower cost of safety protection, it provides increased overall protection and reduces the chance of accidental loss.

2.2.3 Current Applications

2.2.3.1 Maritime Environment

Sandwich Plate System is designed to fulfil the needs of Naval Architects to design lightweight, greatly simplified and better performing ships. The built-in fire resistance, enhance puncture, impact , blast and ballistic resistance, inherent structural damping to control vibration and to minimize transmission of structural borne noise, increase fatigue resistance and reduced susceptibility to corrosion leading to less maintenance and a longer and safer service life are some of the benefits that accrue from SPS composite construction (Intelligent Engineering).

The SPS plates are very stiff and easy to erect, thus prefabricated using repeat use moulds to ensure that the perfect flatness is nearly achieved. The other benefit of SPS plates is located on the manufacturing process; Compared to traditional stiffened steel panels, SPS can be produced in large numbers with significant reductions in weld volume and labour input.

2.2.3.2 Grandstand Stadium

The SPS material which was developed for use in harsh condition of the Alaskan waters is set to revolutionise stadium construction and also construction budgets with its staggering 73% weight saving over concrete. The potential for SPS in stadium construction is as great as in shipbuilding (D. Braun 2002). SPS material is lighter in weight, but outperforming concrete in dynamic and static performance tests. In addition, this material will deliver significant cost savings in substructure construction and total stadium erection times.

In stadium construction, the alternative material to SPS is concrete, whereas is ships the alternative material is ordinary steel. Although the lightweight of ordinary steel makes it an attractive proposition, it is not viable as a construction material for stadium because of its vibration performance and inherent harmonics. These difficulties could be overcome by using SPS and at the same time delivers numerous advantages over concrete.

The most obvious advantage of SPS material is weight-saving. Concrete risers require strong, heavy rakers for support, which in turn require numerous thick columns to bear them, which then demand deeper foundations to take the accumulates weight (D. Braun 2002). SPS offers a revolutionary performance as a riser material with its inherent strength in a lightweight form. SPS is 73% lighter than traditional concrete when formed into stadium risers. Even after load factoring is taken into account, it is 60% lighter. The benefits of using SPS are immediately obvious with the well-known direct correlation between weight savings and cost savings in stadium construction. Lighter risers mean a lighter structure required to support them, where the load patterns are from rakers, to columns, to foundations. It means a significant reduction in the cost of material and construction.

2.3 Material Properties of SPS

2.3.1 Fibre Reinforced Concrete

Fibre Reinforced concretes are best suited for thin-section shapes rather than the conventional steel reinforcement; where for conventional reinforcement the correct placement of the reinforcement is extremely difficult. A thin FRC sections have strength equivalent with the thicker concrete sections and it would be substantial of weight savings. The main advantage for using fibre reinforcing systems as opposed to conventional reinforcing systems is realized in the fabrication processes. Fibre reinforced concrete is either cast or sprayed, therefore eliminating the labour-intensive activity of placing reinforcement. However, fabrication of FRC products can be more labour intensive activity than conventional precast concrete products. In addition, spraying of FRC accommodates the fabrication of irregularly shaped products (J.I.Daniel 1991).

Production of fibre reinforced concrete composites has been developed over the last 30 years. There are several different fibre types that have been used to reinforce cement concrete. Fibres could be made of synthetic organic or synthetic inorganic materials, and natural organic or natural inorganic materials (J.I.Daniel 1991).

Glass Fibre Reinforced Concrete is a composite material which made up of cement, Fine Silica Sand and Alkali Resistance. The Alkali Resistance of the Glass Fibre Reinforced Concrete (GRSC), provides long-term durability because Glass Fibre used effectively. Nowadays, GFRC material is made a significant contribution to the economics, to technology and to the aesthetics of the construction industry worldwide. The material properties of GFRC are: the density 1800kg/m^3, Elastic Modulus of 20000MPa and Poisson's ratio of 0.26

2.3.2 Polystyrene

Polystyrene is a member of plastic family which derived from styrene monomer. They are relatively old material since the styrene polymers were recognized in the laboratory over 100 years ago (W. C. Teach 1960). Regular polystyrenes are essentially polymerized styrene monomer and are frequently referred to normal, conventional or standard polystyrenes. One of the advantages of polystyrene is it is possible to vary such properties as ease of flow, speed of set-up, physical strength and heat resistance by modifying manufacturing conditions and adding small amounts of lubricants both internal and external (W. C. Teach 1960)

Polystyrene is a vinyl polymer. Structurally, it is a long hydrocarbon chain, with a phenyl group attached to every other carbon atom. From the monomer styrene, polystyrene is produced by free radical vinyl polymerization (PSLC 2005).

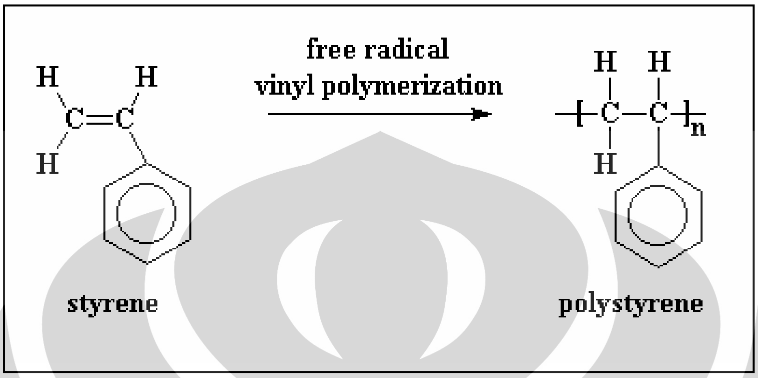

Figure 2.2. Polystyrene Hydrocarbon Chain

Polystyrene is one of the four plastics, which are Polyethylene, Polypropelene, Polyvinyl Chloride and Polystyrene, whose combined usage accounts for 75% of the worldwide usage of plastics. It can be produced by several mechanisms, via free-radical, atomic, cationic, and Ziegler mechanisms. Polystyrene is popular due to its transparency, low-density, relatively high modulus, excellent electrical properties, low cost and ease of processing (Frados 1976).

According to Teach and Kiessling, the good properties of polystyrene are as follows:

- It is light in weight
- It is easy to mold. Like many other thermoplastics, polystyrene may be molded in high-speed injection machines.
- It maintains its dimensions. Articles made of polystyrene hold their shape well at room temperature, even better at lower temperature.
- It is odourless, tasteless and nontoxic.
- It has unusual optical properties. The clear, colourless polymer 9usually designated as “crystal” has a high degree of transparency. It is capable of piping light, that is, transmitting light through the plastic from one edge to another.

- It is water resistant. Conventional polystyrenes are among the best plastics in respect to low water absorption. Modified types absorb slightly more water but are still outstanding.

- It is resistant to many ordinary chemicals, especially corrosive, inorganic liquids such as acids or bases. Some tests have been made on the chemical resistance of polystyrenes, which obtained by ASTM methods. Although these tests are made in the absence of mechanical vibration, flexing, shock and impact or strain conditions which almost always present in industrial applications, in general it can be said that polystyrene withstands all concentration of non-oxidizing acids and aqueous solutions of alkalies.

The first modified polystyrene was offered to the trade in 1984 which did not have the brittleness characteristic of the conventional types. In earlier times, polystyrene had developed a bad name because of its brittleness characteristic (W. C. Teach 1960).

Impact Polystyrene extends the uses of polystyrenes into those areas where high impact strength and good elongation are required. Most products of modified polystyrenes have the rubber molecularly grafted to the polystyrenes and some are produced by mechanical mixing(Harper 2006). For load bearing applications, high impact polystyrene is generally recognized in the filed as the best styrene material. This type of polystyrene is a very rigid material; however it does not withstand long-term tensile loads as well as some other thermoplastics (Harper 2006). According to Frados, modulus of elasticity at tension is 300,000 psi or 2,000 MPa. When loads are to be held longer than 500 hours, most high impact polystyrenes have design strength between 800 and 1500 psi. Stress strain data show high impact polystyrene to be very stable at low loads (15% strain at 300 psi for 1000 hours). However, strain is as much as 35% for greater loads (1000 psi) and for less time (100 hours) (Frados 1976).

2.4 Structural Dynamics

2.4.1 Dynamic Loading

Dynamic loadings can be classified as harmonic load, periodic load, transient load, and impulsive load as shown in Figure 2.3. Harmonic or sinusoidal loads are usually associated with rotating machinery. Periodic loads are caused by rhythmic human activities such as dancing and aerobics and by impactive machinery. Transient loads occur from the movement of people and include walking and running, Impulsive loads are caused by human activities such as single jumps and heel-drop impacts.

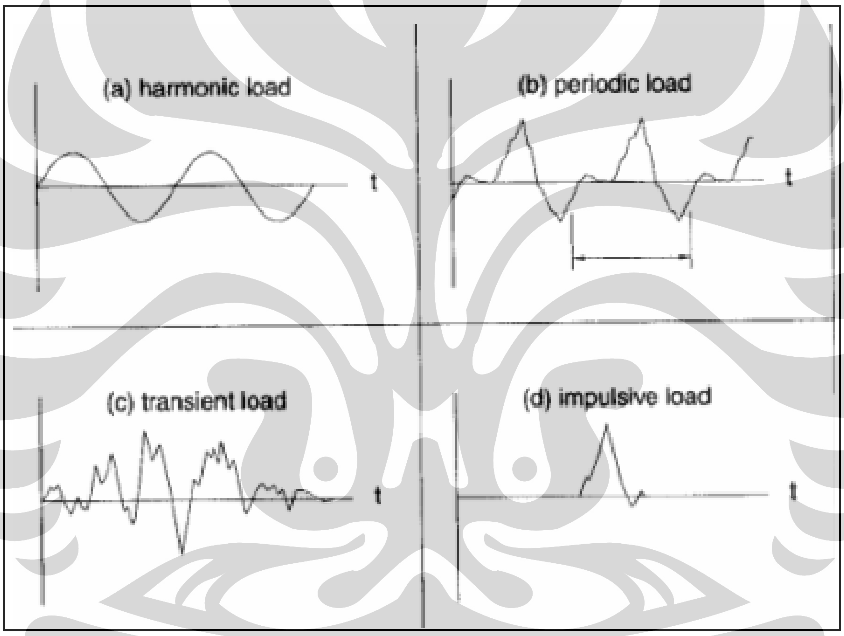

Figure 2.3. Types of Dynamic Loading

2.4.2 Dynamic Response Effects

There are three different response effects in structures due to loading actions. The three distinctly separate responsive areas based on action effects are as follows (Perera):

1. Dynamic action effects that cause small amplitude vibrations with a relatively high frequency of occurrence during the design life of the structure. The mainly excitation of a structure, for instance floor plates and bridge decks, are due to

human activities such as walking, running, dancing, aerobics and jumping. Structure respond elastically to the forces generated accompanied sometimes by occupant discomfort resulting from the perception of motion – see Table 2.1. These are most commonly known as nuisance effects that tend to cause panic when crowds are involved. They have significant social and economic consequences resulting in high incidence of litigation, coupled with the economic consequences due to loss of occupancy, retrofit/repair costs, damage to expensive ornamental fixtures and fittings.

Table 2.1. Human Perception of Motion

Acceleration (m/sec^2)	Perceived Motion Effects by Humans
<0.05	Rarely ever perceived by humans
0.05-0.10	Perceived by sensitive humans; Very slight movement of hanging objects
0.1-0.25	Motion perceived by most people with possible motion sickness due to long exposure; Possible effect on desk work
0.25-0.4	Desk work difficult to impossible; Ambulation still possible
0.4-0.5	Natural walking with difficulty and strong perception of motion; Loss of balance when standing
0.5-0.6	Majority unable to walk or tolerate motion
0.6-0.7	Impossible to walk or tolerate motion
>0.85	Objects begin to fall or topple with injury to occupants

2. Dynamic action effects resulting in moderate to large amplitude vibrations with a low probability of occurrence; loosely defined as events with return periods between 500 to 1000 years. The examples of these events are earthquakes, large wind storms, hurricanes, accidental explosions and construction activities. The

structures are to be designed to respond elastically with limited demand for inelastic energy dissipation at predetermined locations within structural systems. Seismic action effect produces significant vibrations accompanied by post yield deformations at ultimate load conditions. The design philosophy ensures strength and stability of the system and the vertical load bearing elements after the occurrence of such event classified as the ultimate limit state. Although the occupant's comfort is not a governing criterion at this approach, the large amplitude vibrations and deformations if not controlled with reasonable limits can have devastating effects on cladding/curtain wall systems, internal partition walls/ceilings, electro, mechanical and hydraulic services. Such damages has resulted in loss of life, fires, total disruption of post disaster evacuation processes with occupants trapped under collapsed ceilings and partitions. Technically, structures and components will respond to extreme forces in the elastic domain throughout their service life with added damping to control the serviceability criteria.

3. Dynamic action effects due to massive energy input from maximum credible events of unforeseen magnitude and frequency. Some of the examples of this event are acts of war, terrorism, large earthquake and tidal waves. The design aspects that are to be addressed in the result in order to restore public confidence and to ensure safety post disaster relief operations are energy absorbing capacity, redundancy, reserve strength, ability to undergo load reversal, fire and heat resistance, insulation and fire separation. The capacity of structural systems and components to absorb large amount of energy both elastically and in-elastically supplemented by damping is very important in such events.

2.4.3 Natural Frequencies and Free Vibration

The critical factor determining the severity of dynamic response is the lowest relevant natural frequency of the structure. Natural frequency is the frequency at which a body or structure will vibrate when displaced and then quickly released. This state of vibration is referred to as free vibration. All structures have a large number of natural frequencies; the fundamental natural frequency is of most concern. If a frequency of an exciting force is equal to a natural frequency of the structure, resonance will occur. At resonance, the amplitude of the motion tends to become larger to very large, as shown in figure 2.4.

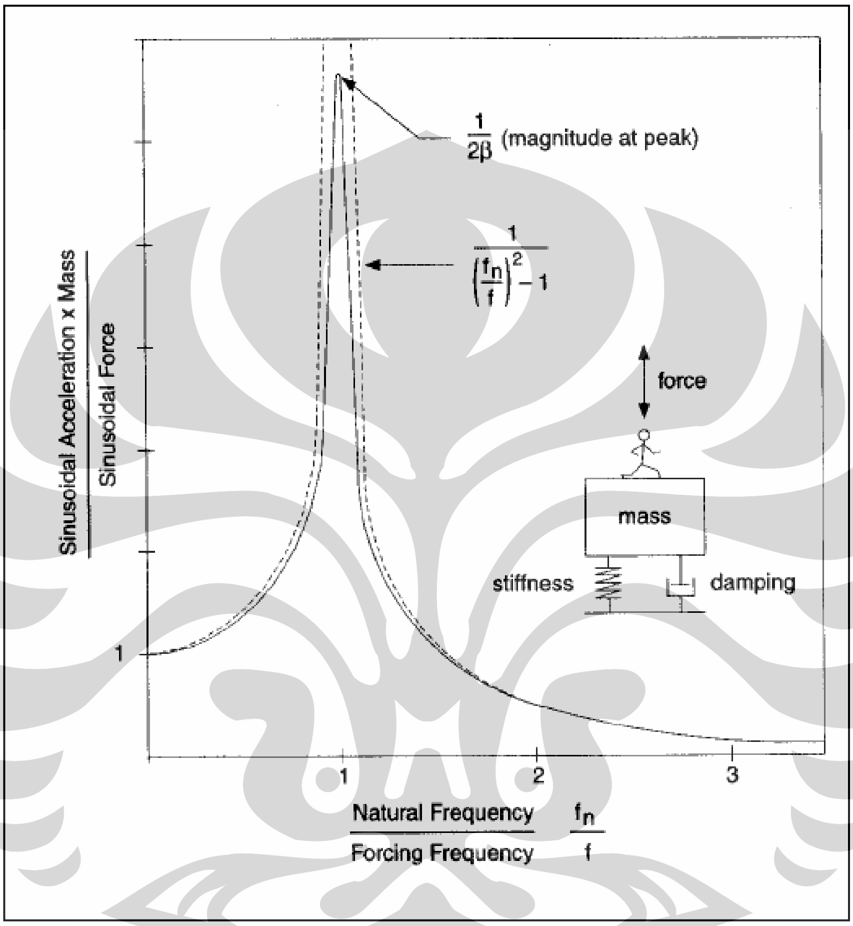

Figure 2.4. Response to Sinusoidal Force

In determining the natural frequency of a structure, the governing factor is the stiffness in relation to mass of the self-weight. Four levels of approach for evaluating natural frequencies, in increasing order of refinement are as follows(Wyatt 1989)):

1. From a global estimate of the self-weight deflection;

2. From a combination of component frequencies estimated from self-weight deflection or tabulated frequency formulae;

3. By iterative application of static analysis, using common static analysis software at the desk-top;

4. By use of dynamic analysis software packages, possibly including finite element modelling of the structure.

The first three levels are generally limited to an evaluation of the fundamental frequency but an extended sequence of modes will be the output by the fourth approach.

Below are the details of the four approaches(Wyatt 1989):

1. The self-weight deflection approach. The frequency of free oscillation of the system shown in Figure 2.5 depends on the stiffness in comparison with the mass according to the following equation:

$$f = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \qquad (2.1)$$

where f is the natural frequency in Hz.

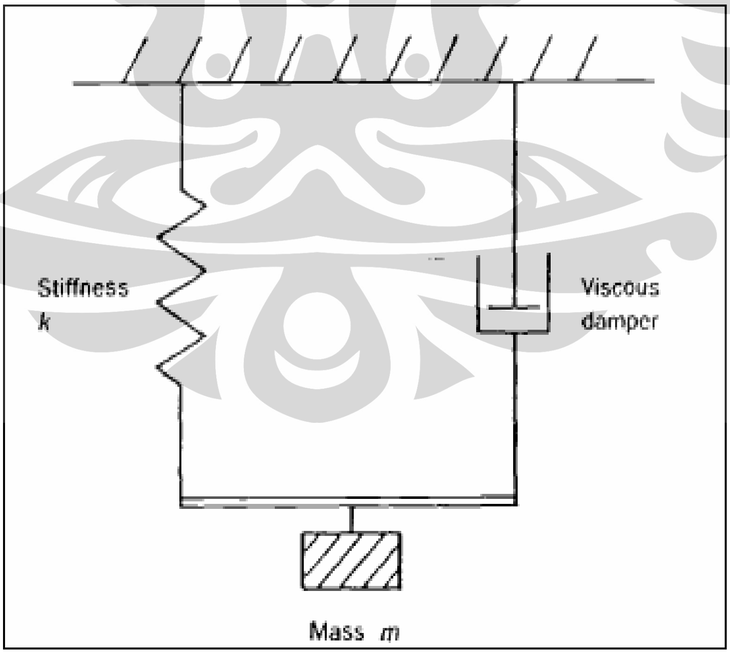

Figure 2.5. Simple Dynamic Model

Since the static deflection caused by the weight of the mass m (presumed to act in the appropriate direction, in line with the spring) would be $\Delta = mg/k$, thus the frequency equation can also be expressed in the form:

$$f = \frac{1}{2\pi}\sqrt{\frac{g}{\Delta}} \qquad (2.2)$$

The self-weight deflection is a quantity which the engineer can generally characterise quite closely without the need for detailed calculation, and which will follow a consistent pattern as a function of span for any given structural form. This equation thus offers a useful general approach to evaluating frequencies, and shows that conventional static design procedures, which include a limit on Δ actually constrain very strongly the value that will result for natural frequency.

Whereas the static deflection caused by the weight of the mass could be calculated from (Steel construction, beam formulae):

For single span simply supported beams:

$$\Delta = \frac{5}{384}\frac{wL^4}{EI} \qquad (2.3)$$

For two equal span continuous beams:

$$\Delta = \frac{1}{185}\frac{wL^4}{EI} \qquad (2.4)$$

where

Δ = static deflection

w = self-weight of a structure

L = length of the span

E = elastic modulus

I = second moment of area or moment inertia

2. The component frequency approach is likely to be helpful where there is a significant interaction with main beam deflections, especially where this results in a fundamental mode shape with significant deflections in further bays. If the component frequencies are estimated by the self-weight deflection method, it becomes effectively the same as the global self-weight deflection approach but aids a clear judgement of the critical mode shape.

3. Where the layout is insufficient regular to permit idealisation as uniform beam components acting in series, and/or a convincing picture of the fundamental mode shape cannot be obtain by simple judgement, the fundamental mode shape can be found by successive approximation using desk-top static analysis procedures. The fundamental frequency can then be obtained with excellent accuracy by a subsequent summation or numerical integration stage that is amenable to either 'spread sheet' computation or hand calculation.

4. All the established commercial structural analysis packages include appropriate dynamic capability, generally with provision for finite element modelling; these are readily available through computer bureaux. The degree of refinement in modelling should generally be somewhat superior to that indicated for the iterative approach. The mathematical solution can be obtained to any desired accuracy. It should be borne in mind, however, that tis will outstrip the quality of the input data, including joint and support continuity, stiffness prediction for elements and modelling of the excitation processes.

2.4.4 Modes of Vibration

Associated with each natural frequency is a 'mode shape'. Mode shape is a characteristic deformation pattern of the structure when it is undergoing vibration at one of its natural frequencies. With this terminology, the structure can vibrate in one or other of a combination of modes. Two types of modes are as follows (Wyatt 1989):

- A ‘vertical’ mode is one that has a significant component of vertical displacement at the seating deck, such that it may be excited by the vertical dynamic load due to the crowd.
- A ‘horizontal’ mode is one that has a significant component of horizontal displacement in the front-to-back (‘lateral’) or side-to-side (longitudinal’) directions at the seating deck, such that it may be excited by a horizontal dynamic load introduced by the crowd.

The distinction between vertical and horizontal modes is generally helpful one; however some vertical modes may involve horizontal motion and vice-versa. Occasionally significant horizontal responses are observed as a consequence of vertical jumping activity of a crowd. This horizontal response could occur because a vertical mode with a horizontal response component has been excited. Alternatively, the jumping motion could have introduced a horizontal load component that excited a dominant horizontal mode. Similarly, depending on the structure, vertical response can be expected to accompany sway excitation.

For ‘multi-degree-of-freedom’ systems with several masses elastically interconnected and especially the continuously distributed mass system such as the beam shown in Figure 2.6, there will be a series of natural frequencies and each associated with its own mode shape. The various modes are dynamically independent (orthogonal or ‘normal’ modes) so that response can be synthesised by adding modal solutions computed independently.

The lowest frequency mode is the fundamental. This mode has the simplest shape, and its frequency will still be strongly constrained. The higher modes have shapes of increasing complexity. This mode shape may be referred to as harmonics although their frequencies are not in general exact integer multiples of the fundamental frequency. For beams, the second mode of frequency is commonly at least three times the fundamental. It depends on the support conditions, mass and stiffness distribution and the span ratios where applicable (Wyatt 1989). For the simply supported uniform beam (Figure 2.6) the second frequency is four times the fundamental.

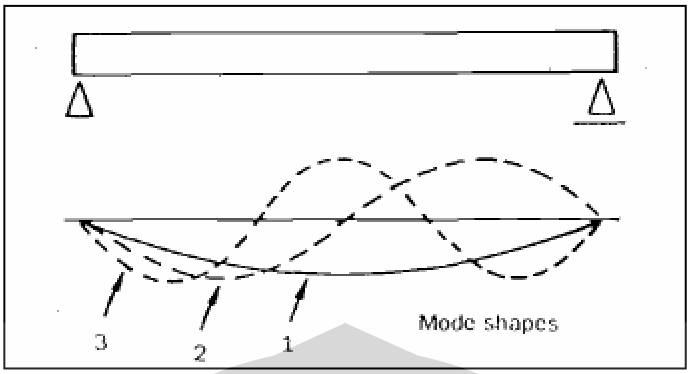

Figure 2.6. Beam Mode Shapes

An insight into the behaviour of some floors is given by the behaviour of an orthotropic plate shown in Figure 2.7. The fundamental mode shape resembles the corresponding beam mode shape in both directions. This principle applies also to the higher modes, but if the stiffness is highly orthotropic, the weak direction deformation has relatively little effect on the frequency, and a basic family of modes retaining the fundamental shape in the strong direction can occur at rather close frequencies.

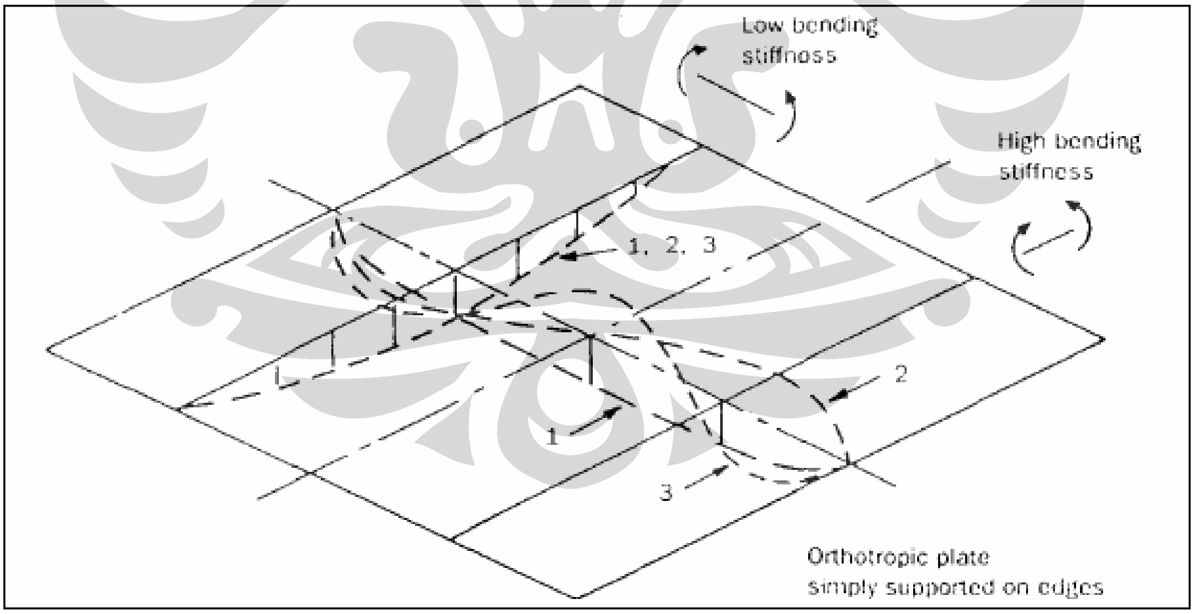

Figure 2.7. Orthotropic Plate Mode Shapes

For continuous beams the fundamental frequency is clearly associated with a shape of the form shown in Figure 2.8. The inertial loads act in the sense shown and enhance the deflections, whereas in static design process the self-weight effects on adjacent spans combine to reduce the corresponding stresses and deflections. Thus, if designed to the same static criteria, continuous construction of uniform spans may have a significantly lower fundamental frequency than a simple structure.

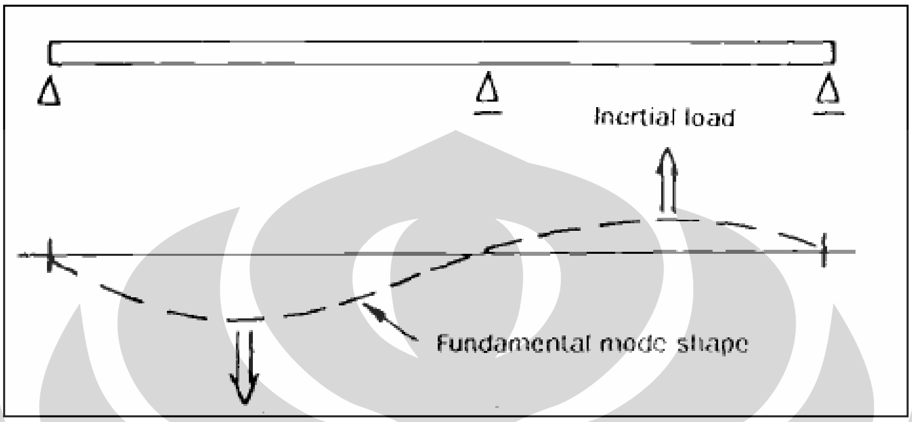

Figure 2.8. Continuous Beam Fundamental Mode Shape (Wyatt 1989)

2.4.5 Damping

The vibration levels in a structure can be reduced by increasing the structure's damping, but every effort should be made to reduce the vibration excitation at its source. While many activities generate a disturbing force of one sort or another, the forcing frequency should not be at, or near, a natural frequency or the structure otherwise the resonance will occur. Thus, it will result in high amplitudes of vibration and dynamic stresses and noise and fatigue problems. Resonance may also prevent the structure fulfilling the desired functions. Some reduction in excitation can often be achieved by changing the machinery generating the vibration, however, structural vibration caused by external excitation sources such as ground movement or vibration, cross winds or turbulence from adjacent buildings can only be controlled by damping. Damping refers to the loss of mechanical energy in a vibrating system.

It is desirable for all structures to possess sufficient damping so that their response to the expected excitation is acceptable (Beards 1996). Increasing the damping in a structure will reduce its response to a given excitation. If the damping in a structure is increased, there will be a reduction in vibration and noise. The dynamic stresses in the structure will be also reduced with a resulting benefit to the fatigue life. However, the increasing of damping in a structure is not always easy, it can be expensive and it may be wasteful of energy during normal operating conditions.

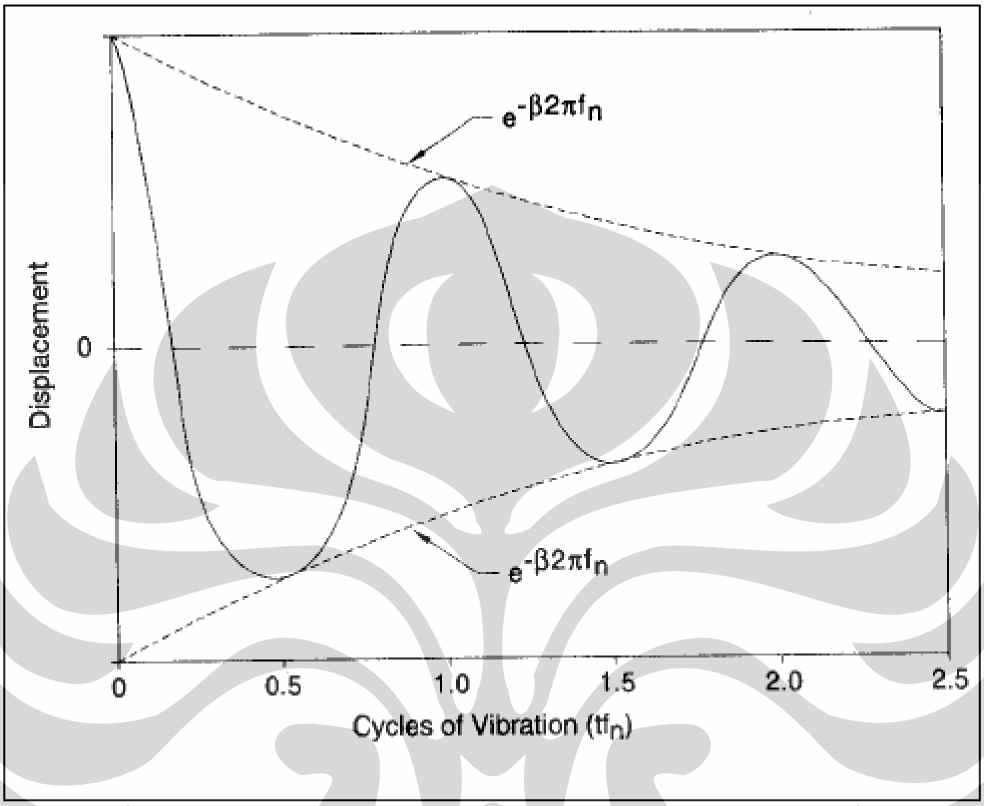

Figure 2.9. Decaying Vibration with Viscous Damping (Wyatt 1989)

Damping is usually expressed as the percentage of critical damping or as the ratio of actual damping to critical damping. Critical damping is the smallest amount of viscous damping for which a free vibrating system that is displaced from equilibrium and released comes to rest without oscillation. Viscous damping is associated with a retarding force that is proportional to velocity. For damping that is smaller than critical, the system oscillates freely as shown in Figure 2.9.

2.5 Floor Vibration

Floor vibration problems involve repeated forces caused by machinery or by human activities such as walking, dancing or aerobics. Walking is a little more complex than the others because the forces change location with every step. In some cases, the applied force is sinusoidal or nearly so. In general, a repeated force can be represented by a combination of sinusoidal forces whose frequencies are multiples or harmonics of the basis frequency of the force repetition, for instance the step frequency, f_{step}, for human activities. The time dependent repeated force can be represented by the Fourier series (David E Allen 2001):

$$F = P\left[1 + \sum \alpha_i \cos\left(2\pi i f_{step} t + \Phi_i\right)\right] \tag{2.5}$$

where

P = person's weight

α_i = dynamic coefficient for the harmonic force

i = harmonic multiple (1, 2, 3, ...)

f_{step} = step frequency of the activity

t = time

ϕ_i = phase angle for the harmonic

As a general rule, the magnitude of the dynamic coefficient decreases with increasing harmonic. For example, the dynamic coefficients associated with the first four harmonics of walking are 0.5, 0.2, 0.1 and 0.05, respectively. In theory, if any frequency associated with the sinusoidal forces matches the natural frequency of a vibration mode, then resonance will occur, causing severe vibration amplification. The effect of resonance is shown in Figure 2.4.

For vibration caused by machinery, any mode of vibration must be considered, high frequency as well as low frequency. For vibration due to human activities such as dancing or aerobics, a higher mode is more difficult to excite because people are spread out over a relatively large area and tend to force all panels in the same direction simultaneously, while adjacent panels must move in opposite directions for higher modal response. Walking generates a concentrated force and therefore may excite a higher mode (David E Allen 2001).

It is possible to control the acceleration at resonance by increasing damping or mass, since acceleration = force divided by damping times mass (see figure 2.4). The control is most effective where sinusoidal forces are small, as they are for walking. Natural frequency also always plays a role, because sinusoidal forces generally decrease with increasing harmonic. Therefore, the higher the natural frequency, the forces become lower.

Where the dynamic forces are large, resonant vibration is generally too great to be controlled practically by increasing damping or mass. The natural frequency of any vibration mode significantly affected by the dynamic force must be kept away from the forcing frequency. Thus, the fundamental natural frequency must be made grater than the forcing frequency of the highest harmonic force that can cause large resonant vibration.

2.5.1 Human Response to Floor Motion

Human response to floor motion involves the magnitude of the motion, the environment surrounding the sensor, and the human sensor. The threshold of perception of floor motion in a busy workspace can be higher than in a quiet apartment. The reaction of people on the fiftieth floor can be considerably different from that of a person on the second floor of an apartment complex, if both are subjected to the same motion. The reaction of people who feel vibrations depends very strongly on what they are doing. People in offices or residences do not like “distinctly perceptible” vibration, while people taking part in an activity will accept larger vibration of about 5 times greater. Sensitivity within each occupancy also varies with duration of vibration and remoteness of source (Wyatt 1989).

In large amplitudes of oscillation at frequencies in the range 2Hz to 20Hz, there may be significant strains within the human body. It possibly includes resonance of specific organs, giving rise to acute discomfort, serious impairment of ability to perform mechanical tasks, and even injury. The criteria appropriate to residential of office environments are associated with intermediate levels of vibration at which purely physiological effects take second place to psychological factors.

The importance of psychological factors makes it difficult to quantify human reaction at these levels. There are wide variations between individuals; range amplitude exceeding a factor of 2 exists between the top and bottom 5% of the population for any given reaction. Reactions at these levels may be influenced by a number of factors. At the lower end of frequency range, reaction is strongly linked to a feeling of insecurity, based on instinctive association of perceptible motion in a 'solid' building structure with an expectation of structural inadequacy or failure. At the higher end of the frequency range, reaction is strongly linked to associated noise levels. It is rational to assume that human reaction would be related to the former at very high frequencies, since the body mass will not follow the floor motion and the perception will be of strain in the legs and spine. At the other extreme of very low frequency, human reaction would be related to the rate of adjustment of the inertia forces of the body. Typical broad qualitative contours of reaction to sustained uniform vibration are shown in Figure 2.10.

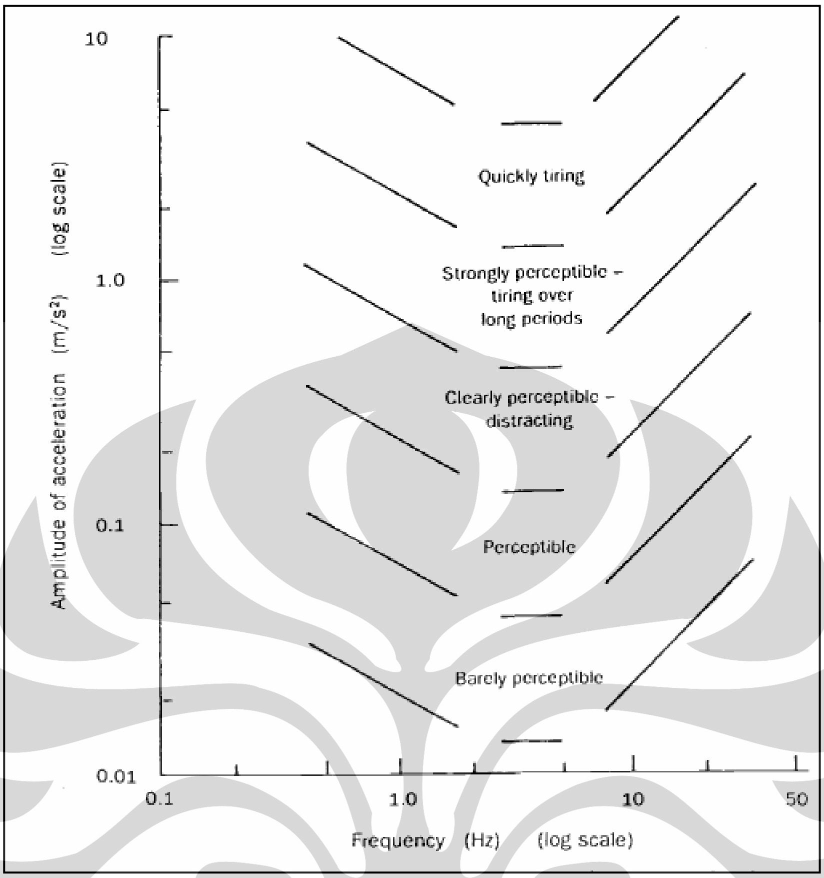

Figure 2.10. Qualitative Description of Human Reaction to Sustained Steady Oscillation (Wyatt 1989)

The Canadian Specification CAN3-S16.1 *Steel Structures for Buildings* includes a very useful Appendix entitled 'Guide for floor vibrations', although this is not a mandatory part of the code. The proposed annoyance criteria for floor vibrations are shown in Figure 2.11. In these curves, the curves labelled 'walking vibration' are to be used for assessing the response to heel drop impulse, and the curve labelled 'continuous vibration' is to be used for the assessment of the motion caused by a person walking across the floor.

It is even more difficult to extend the criteria to non steady vibrations. It has been suggested that noise directly associated with the oscillation is an adverse factor. However, for high-quality environments, for instance residential or office, where an occupant will resent intrusion on his mental concentration, it may be that the appropriate vibration limit would actually be higher where there is substantial ambient noise from other causes.

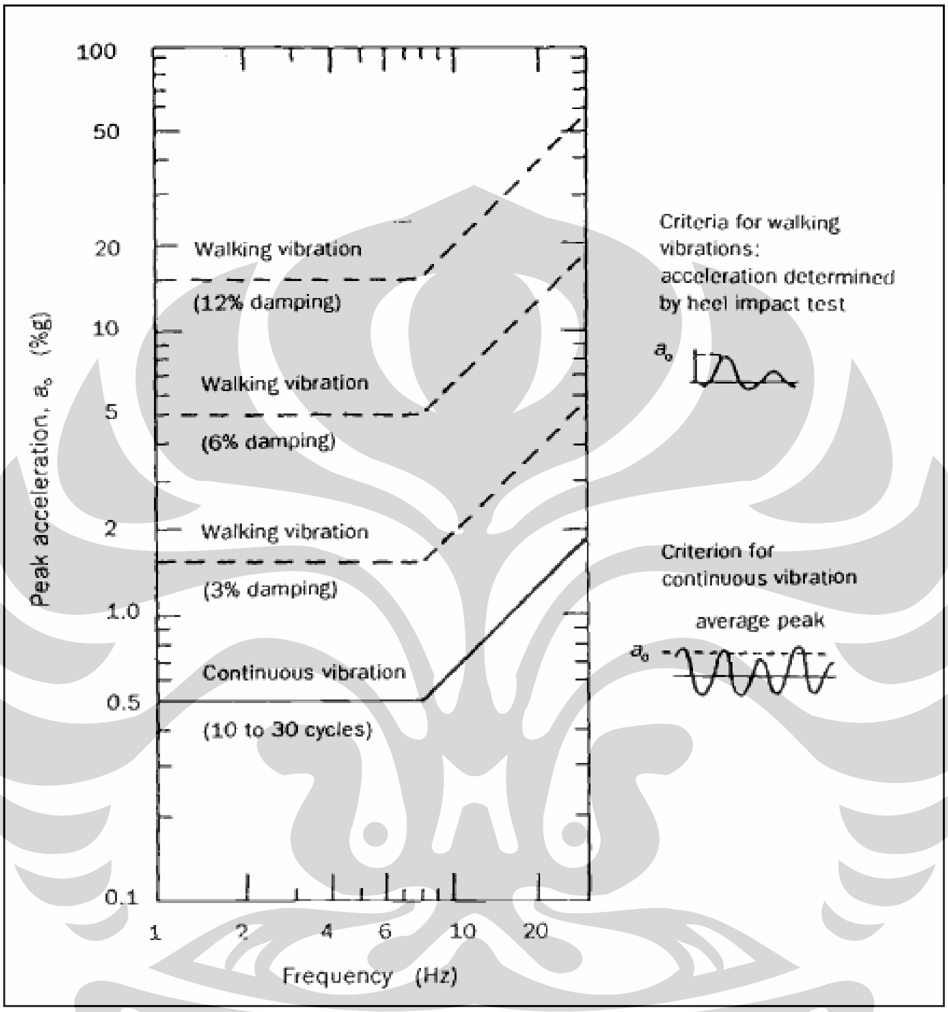

Figure 2.11. Annoyance Criteria for Floor Vibrations (Residential, School and Office Occupancies) (Wyatt 1989)

The three curves in Figure 2.11 labelled 'walking vibration' are specifically linked in the Canadian Code with the 'heel drop' impact test. The Canadian suggests 6% of critical damping for typically-furnished floors without partitions. The sensitivity to the level of damping reflects the greatly reduced annoyance caused by impulsive event when the subsequent decay is very rapid.

2.5.2 Design for Walking Excitation

The obvious and almost universal possible cause of dynamic excitation of floors is the effect of walking on the floor. The vertical accelerations of the body mass are necessarily associated with reactions on the floor, whereas they will be closely periodic at the pace frequency.

The criterion for vibration caused by walking is based on the dynamic response of steel beam- or joist-supported floor systems to walking forces. It can be used to evaluate structural systems supporting offices, shopping malls, footbridges, and similar occupancies (David E Allen 2001). The design criterion for walking excitation is based on:

- Acceleration limits as recommended by the International Standards Organization (International Standard ISO 2631-2, 1989), adjusted for intended occupancy. The ISO standard suggests limits in terms of RMS acceleration as a multiple of the baseline curve shown in Figure 2.13. The limits can be range between 0.8 and 1.5 times the recommended values depending on the duration of vibration and the frequency of vibration events.

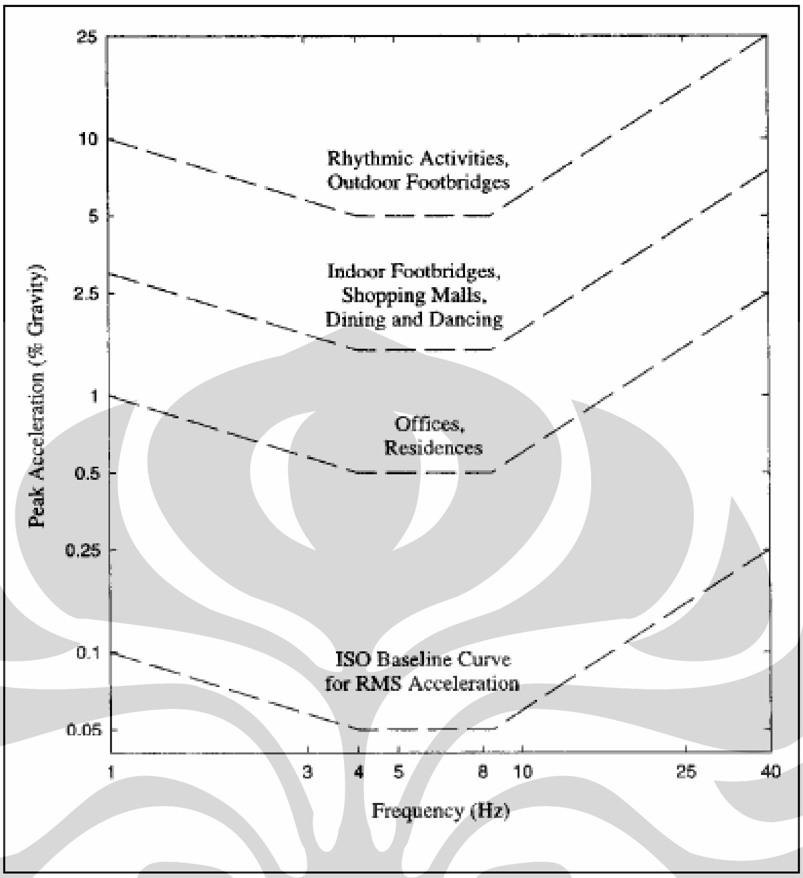

Figure 2.12. Recommended Peak Acceleration for human Comfort for Vibrations due to Human Activities (Allen and Murray, 1993; ISO 2631-2:1989)

- A time dependent harmonic force component that matches the fundamental frequency of the floor:

$$F_i = P\alpha_i \cos\left(2\pi i f_{step} t\right) \tag{2.6}$$

where

P = person's weight, taken as 0.7 kN for design

α_i = a dynamic coefficient for the *i*th harmonic force component

i = harmonic multiple of the step frequency

f_{step} = step frequency

Recommended values for αi are given in Table 2.2. Only one harmonic component of Equation (2.5) is used since all other harmonic vibrations are small in comparison to the harmonic associated with resonance.

Table 2.2. Common Forcing Frequencies and Dynamic Coefficients

Common Forcing frequencies (f) and Dynamic Coefficients (αi)						
Harmonic	Person Walking		Aerobic Class		Group Dancing	
I	f (Hz)	αi	f (Hz)	αi	f (Hz)	αi
1	2.2	0.5	2.8	1.5	2.8	0.5
2	4.4	0.2	5.6	0.6	5.6	0.1
3	6.6	0.1	8.4	0.1		
4	8.8	0.05				

- A resonance response function of the form:

$$\frac{a}{g} = \frac{R\alpha_i P}{\beta W}.\cos\left(2\pi i f_{step} t\right) \tag{2.7}$$

where

a/g = ratio of the floor acceleration of gravity

R = reduction factor

β = modal damping ratio

W = effective weight of the floor

The reduction factor R takes into account the fact that full steady-state resonance motion is not achieved for a walking person and the person annoyed are not simultaneously at the location of maximum modal displacement. It is recommended that R be taken as 0.5 for floor structures with two-way mode shape configurations.

For evaluation, the peak acceleration due to walking can be estimated from equation (2.7) by selecting the lowest harmonic, i, for which the forcing frequency, $f = i \cdot f_{step}$, can match a natural frequency of the floor structure. The peak acceleration is then compared with the appropriate limit in figure 2.12. For the design purposes, equation (2.2) can be simplified by approximating the step relationship between the dynamic coefficient, α_i, and frequency, f, by the formula $\alpha = 0.83\exp(-0.35f_n)$. With this substitution, the following simplified design criterion is obtained:

$$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W} \leq \frac{a_o}{g} \tag{2.8}$$

where

a_p/g = estimated peak acceleration (in units of g)

a_o/g = acceleration limit from Figure 2.12

f_n = natural frequency of floor structure

P_o = constant force equal to 0.41 kN

The numerator $P_o\exp(-0.35f_n)$ represents an effective harmonic force due to walking which results in resonance response at the natural floor frequency f_n.

2.5.3 Design for Rhythmic Excitation

Vibrations due to rhythmic activities were first recognized in a commentary to the 1970 National Building Code of Canada (NBC), where it was stated that resonance due to human activities can be a problem if the floor frequency is less than 5 Hz. The recent criteria have been developed for design of floor structures for rhythmic excitation (Allen 1990, 1990a; NBC 1990) are based on the dynamic response of structural systems to rhythmic exercise forces distributed over all or part of the floor/seating deck. The criteria can be used to evaluate structural systems supporting aerobics, dancing, audience participation and similar events. Figure 2.13 shows a time record of the dynamic loading function and an associated spectrum for eight people jumping at 2.1 Hz. Table 2.3 gives common forcing frequencies and dynamic coefficients for rhythmic activities.

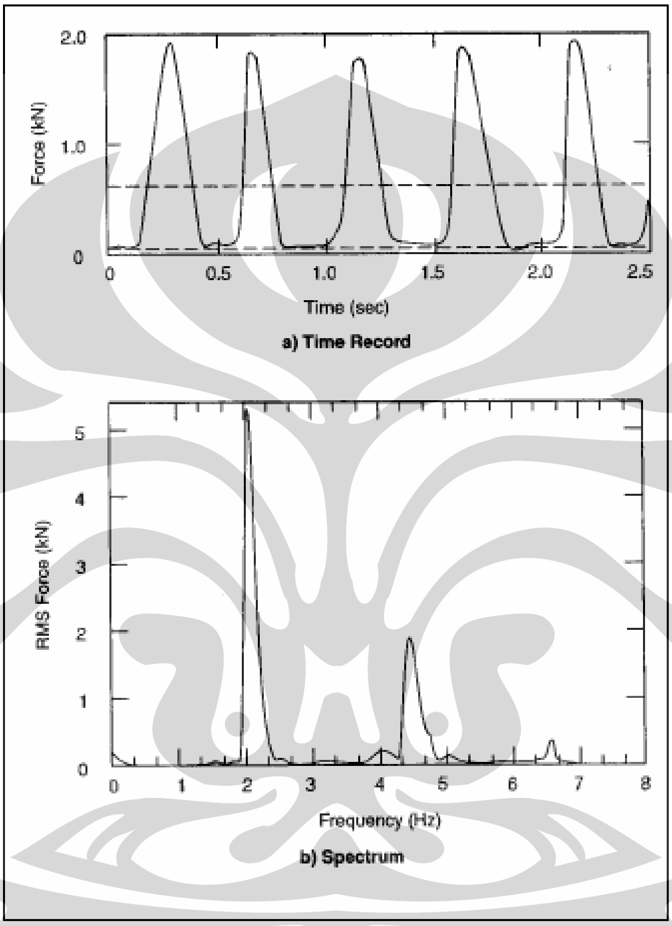

Figure 2.13. Example Loading Function and Spectrum from Rhythmic Activity (David E Allen 2001)

Table 2.3. Estimated loading During Rhythmic Events (David E Allen 2001)

Estimated Loading During Rhythmic Events					
Activity	Harmonic	Forcing	Weight of	Dynamic	Dynamic Load
		Frequency	Participants	Coefficient	α_i wp
	i	f, Hz	Wp, kPa	α_i	kPa
Dancing: First Harmonic	1	3	0.6	0.5	0.3
Lively Concert or sport event:					
First Harmonic	1	3	1.5	0.25	0.375
Second Harmonic	2	5	1.5	0.05	0.075
Jumping exercises:					
First Harmonic	1	2.75	0.2	1.5	0.3
Second Harmonic	2	5.5	0.2	0.6	0.12
Third Harmonic	3	8.25	0.2	0.1	0.02
* Based on maximum density of participants on the occupied area of the floor for commonly encountered conditions. For special events the density of participants can be greater.					

The peak acceleration of the floor due to a harmonic rhythmic force is obtained from the classical solution by assuming that the floor structure only has one mode of vibration (Allen 1990):

$$\frac{a_p}{g} = \frac{1.3\alpha_i w_p / w_i}{\sqrt{\left[\left(\frac{fn}{f}\right)^2 - 1\right]^2 + \left[\frac{2\beta f_n}{f}\right]^2}} \qquad (2.9)$$

where

a_p/g = peak acceleration as a fraction of the acceleration due to gravity

α_i = dynamic coefficient (see table2.3)

w_p = effective weight per unit area of participants distributed over floor panel (see table 2.3)

w_t = effective distributed weight per unit area of floor panel, including occupants

f_n = natural frequency of floor structure

f = forcing frequency

= $i \cdot f_{step}$ where f_{step} is the step frequency (see table 2.3)

β = damping ratio

Equation (2.9) can be simplified as follows:

At resonance ($f_n = f$):

$$\frac{a_p}{g} = \frac{1.3}{2\beta} . \frac{\alpha_i w_p}{w_t} \quad (2.10)$$

Above resonance ($f_n > 1.2f$):

$$\frac{a_p}{g} = \frac{1.3}{(f_n / f)^2 - 1} . \frac{\alpha_i w_p}{w_t} \quad (2.11)$$

Most problems occur if a harmonic forcing frequency, $f = i \cdot f_{step}$ is equal to or close to the natural frequency, f_n, for which case the acceleration is determined from equation (2.10). The acceleration for a lower harmonic is determined from equation (2.11). The effective maximum acceleration, accounting for all harmonics, can be estimated from the combination rule (Allen 1990a):

$$a_m = \left[\sum a_i^{1.5}\right]^{1/1.5} \quad (2.12)$$

where

α_i = peak acceleration for the *i*'th harmonic

The dynamic forces for rhythmic activities tend to be large and resonant vibration is generally too great to be reduced practically by increasing the damping or mass. This means that for design purposes, the natural frequency of the structural system, f_n, must be made greater than the forcing frequency, f, of the highest harmonic that can cause large resonant vibration.

The following design criterion for rhythmic excitation (Allen 1990a) is based on the dynamic loading function for rhythmic activities and the dynamic response of the floor structure:

$$f_n \geq (f_n)_{req'd} = f\sqrt{a + \frac{k}{a_o / g}\frac{\alpha_i w_p}{w_t}} \qquad (2.13)$$

where

f_n = fundamental natural frequency of the structural system

$(f_n)_{req'd}$ = minimum natural frequency required to prevent unacceptable vibrations at each forcing frequency, f

f = forcing frequency = $i \cdot f_{step}$ (see table 2.3)

i = number of harmonic = 1, 2, or 3 (see table 2.3)

f_{step} = step frequency

k = a constant (see table 3.4)

a_o/g = acceleration limit (0.05, or less, if sensitive occupancies are affected)

Table 2.4. 'k' Values (David E Allen 2001)

Constant 'k' Values for Rhythmic Excitation	
Activity	**k**
Dancing	1.3
Lively Concert or sport events	1.7
Aerobics	2.0

2.6 Dynamic Performance of Grandstand Due to Crowd Activity

2.6.1 Dynamic Crowd Loads

Dynamic crowd loads are generated by the movement of people. The largest loads are produced by synchronized rhythmic movements which arise from people dancing or engaging in jumping type movements and usually in response to a musical beat. Although usually associated with lower numbers of people, aerobic activities will give rise to similar dynamic effects. A crowd of people jumping rhythmically can generate larger loads and this may be a concern for both safety and serviceability conditions. However, the rhythmic excitation most frequently encounter does not involve everyone jumping in perfect synchronization but includes people dancing and clapping, often with some people stationary. Thus, this situation will produce smaller loads than if everyone was jumping and usually a much smaller structural response (Ellis and Ji 2004).

The peak load produced by jumping is significantly larger than the load resulting from standing still. If a person keeps jumping the resulting dynamic load will be cyclic and in certain situations it can generate a resonant response of the loaded structure. Although there are some human activities which involve repeated jumping, for crowds the dynamic loads will only be significant when the movement is synchronized. Structures which are designed for static loading, there are many safety factors built into the design which allow for most dynamic loads. If a group of people on a structure were jumping in an uncoordinated manner, the peak displacement induced by the load would not be significantly larger than that induced by static load. The load becomes significant only when coordination is achieved.

Powerful dynamic loading can occur during a pop concert when a crowd responds physically on a rhythmic manner to music having strong beat. On the other hand, a crowd can also behave rhythmically during a football or other sporting event, when music is played, when a team ‘scores’ or when spectator sing without musical accompaniment. Dynamic loading will occur when a group of people jumps, sways, claps or stamps in harmony. A single event, such as crowd suddenly rising or sitting together, can also produce a dynamic response but it is repeated events that can lead to significant motion (Ellis and Ji 2004). Loading can occur in both the vertical and horizontal directions, it can cause both vertical bounce and horizontal sway of either component part or of the facility as a whole.

Annex A to BS 6399 recommends that a frequency range for vertical excitation of 1.5Hz – 3.5Hz should be considered for individual loads from dancing and jumping but that the range reduces to 1.5Hz – 2.8Hz for larger groups due to difficulties of coordination at the higher frequencies. More recent work suggests that this range might be narrowed to 1.8Hz – 2.3Hz for large groups as occur at pop-concerts. For horizontal excitation due to crowd action, the frequency by a seated crowd could be as high as 0.9Hz for energetic sway. However, this is not likely to be sustainable and that 0.7Hz might be considered a more representative value for purposes of design. These values are on the basis of previous experience with demountable stands (Ellis and Ji 2004).

2.6.2 Rhythmic Loading due to Crowd Activity

Rhythmic load time history may be represented as the superposition of a number of load components corresponding to integer multiples of the frequency of the basic motion. For instance, crowd motion at frequency f will lead to load components at f (first harmonic or fundamental frequency), 2f (second harmonic), 3f (third harmonic), etc. The magnitudes of the component decrease as the harmonic order increases, thus, the first harmonic's (fundamental frequency) is the largest.

For an individual, the magnitude of the different harmonics increases with the intensity of dynamic activity and whether it involves an impacting motion such as jumping. Rhythmic jumping leads to the highest loads, which include significant components from the third and fourth harmonics. This pattern of loading is modified when a large number of individuals attempt to jump simultaneously, even with accompanied with a musical beat (2001).

In case where a group of people moves rhythmically, imperfect coordination between the load time histories associated with each person leads to some attenuation of the dynamic load generated by the group. This reduction, in average load per person, is greater for the higher harmonic components. Thus, the third and fourth load harmonics become far less significant for crowd loading than for jumping by an isolated person or a small well-synchronised group. However, the ability of a group to act in a coordinated manner improves in the presence of an external stimulus of increasing strength, be it aural and/or visual.

Thus, the importance of higher harmonic load components is believed to increase as the event scenario progresses to increased levels of likely crowd dynamic participation and as the nature of the motion progresses from non-impacting to impacting, for instance:

- Spectator event with no singing or music played. First harmonic will be dominant in this case.
- Event with singing, but without musical accompaniment. Some of the second harmonic will influence.
- Event with some audience participation with singing to musical accompaniment but without impacting motion. First and second harmonics will be significant.
- Dedicated pop rock concert with audience participation and likelihood jumping. In this case, first and second harmonics are very important with the third harmonic becoming influential only if the audience reaction were unusually well synchronised.

This progression is utilised in an Interim Guidance on assessment and design of Dynamic Performance Requirement for Grandstand Subject to crowd Action in November 2001 by the IStructE because the higher load component causing a significant dynamic response will increase as the strength and impacting nature of the stimulus increases.

The effect of dynamic loading from rhythmic activity of a crowd depends to the proximity of the first or higher harmonic frequencies of the activity to any of the natural frequencies of the structure. The worst case occurs when one of the excitation frequency components is at or near to a natural frequency of the structure which means that a resonant dynamic response can occur. For resonance, the response builds up rapidly cycle by cycle until a maximum limiting response is reached. The resulting cyclic motion can lead to discomfort or, in extreme cases, spectator panic or structural damage. Even though resonance occurs in the structure, the response is limited by the mass of the structure, level of damping and by the inability of the crowd to sustain the motion in a synchronised manner.

3 INVESTIGATION

3.1 Material Selection

The grandstand sandwich plate system will be developed using fibre reinforced concrete as the shell material and polystyrene elastomer core as the second material. As discussed in the literature review, the use of fibre reinforced concrete has several benefits over using steel as the shell material. However, the use of fibre reinforced concrete itself instead of the normal steel reinforced concrete is due to its better material properties which can provide the high strength with a relatively thin section. The Sandwich Plate System using steel plates has several weakness for application in grandstand stadium due to the use of steel which create the noise while people or the spectators walking or jumping in the floor. Furthermore, the steel prices are more expensive compare with fibre reinforced concrete as shown in Figure 3.1. Although it shows that the prices of fibre reinforced concrete is higher than steel in terms of per kilogram, the material density of fibre reinforced concrete is much lighter than the steel. Thus, from the overall consideration, steel has been become much more expensive material compared with fibre reinforced concrete, in this case the glass fibre reinforced concrete.

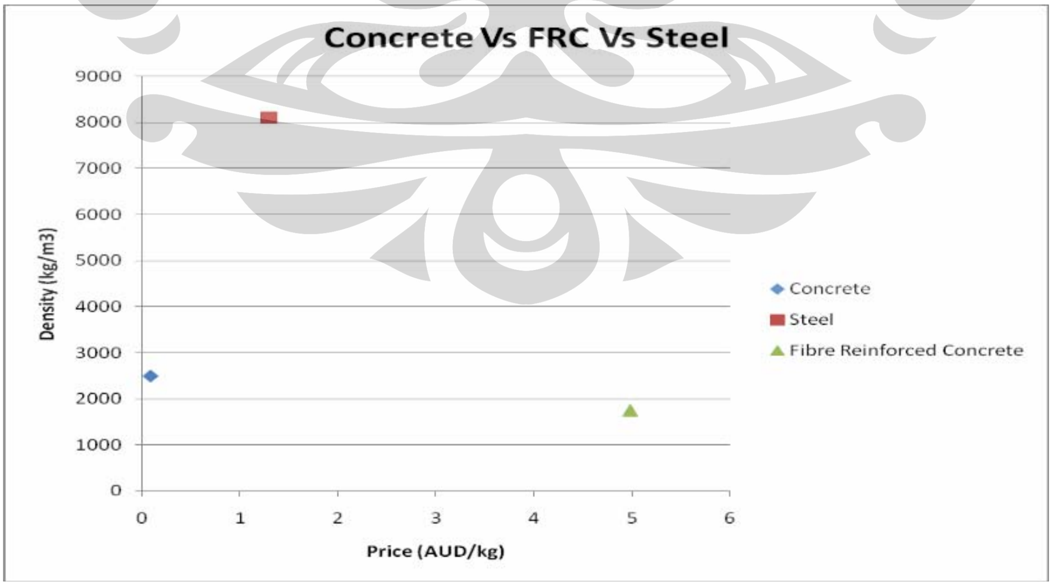

Figure 3.1. Comparison between Concrete Vs FRC Vs Steel (CESEdupack 2006)

Not only for the shell material, the elastomer core using in this investigation also different from the sandwich Plate System created by Dr Stephen Kennedy. Polystyrene is used rather than polyurethane. The reason is because of the polystyrene can provide the expected plastic behaviour without the high prices as can be seen in Figure 3.2, although the polyurethane have a better material properties than polystyrene. Figure 3.2 also shows the comparison between the prices and the material density of four common plastics worldwide, polystyrene, polyurethane, polyethylene and polyester. As can be seen, polystyrene is the cheapest among others plastic with the low material density.

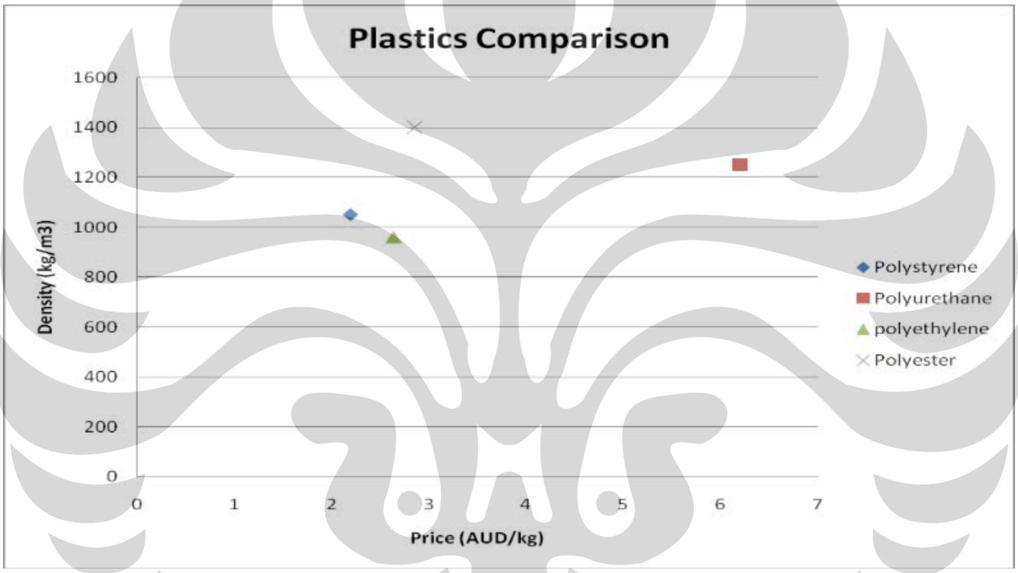

Figure 3.2 (CESEdupack 2006)

The material properties of Glass Fibre Reinforced Concrete and High Impact and Flame Retardant Polystyrene are shown in Table 3.1.

Table 3.1. Material Properties of FRC and Polystyrene

Material Properties of Fiber Reinforced Concrete and Polystyrene		
	FRC	Polystyrene
Density (kg/m3)	1800	1130
Elastic Modulus (Mpa)	20000	2000
Poisson's Ratio	0.26	0.33

The fibre reinforced concrete used is the Glass Fibre Reinforced Concrete and the polystyrene is High Impact and Flame Retardand Polystyrene. High Impact Polystyrene is chosen due to the need of structure to resist high impact loads and General Purpose Polystyrene could not provide this behaviour. Furthermore, because the fibre reinforced concrete is relatively thin, the ability of concrete to protect the structure from fire is not enough, thus the polystyrene elastomer core should be designed to be able to retard the high temperature of the fire.

3.2 Dimension and Support Selection

The preliminary design of the sandwich plate system is made of 10mm thick of fibre reinforced concrete as a shell and 130mm thick of polystyrene elastomer core as can be seen in Figure 3.3. There are two types of cross section that are going to be considered in design, the L-shape and Double-L-shape as shown in Figure 3.4.

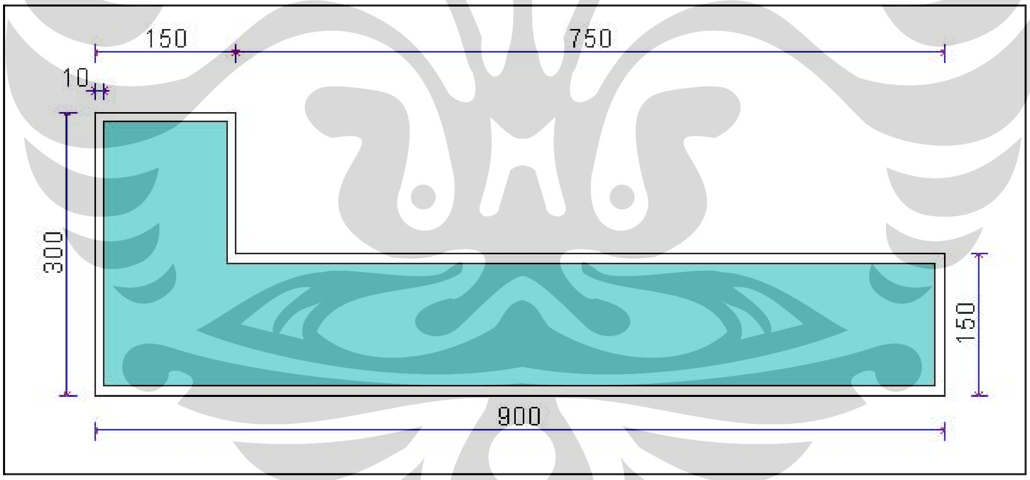

Figure 3.3. L-Shaped Cross-section

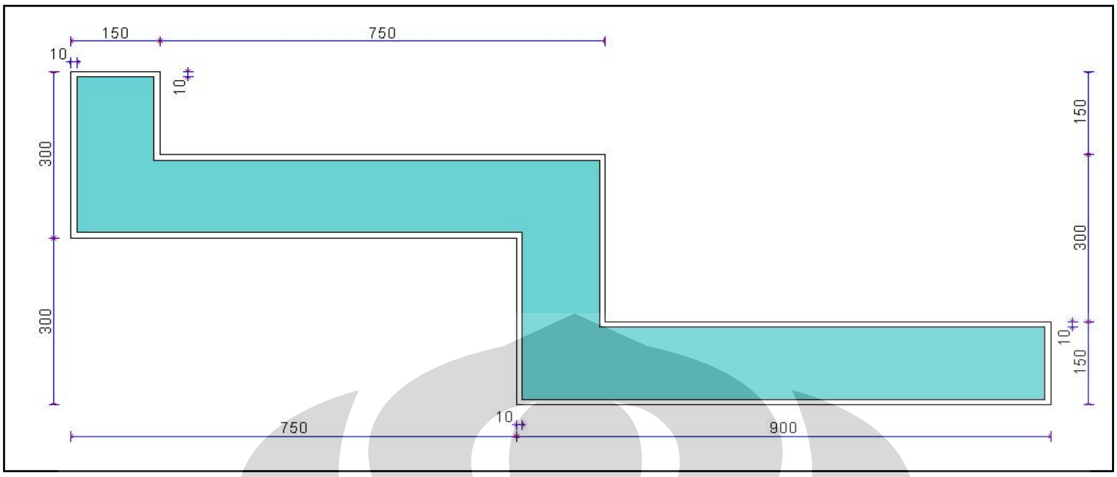

Figure 3.4. Double-L-Shaped Cross-section

The design investigation will consider three different cases:

1. L-shaped with simply supported boundary constraint (see Figure 3.5)
2. L-shaped with two spans continuous beam (see Figure 3.6)
3. Double-L-shaped with simply supported boundary constraint (see figure 3.5)

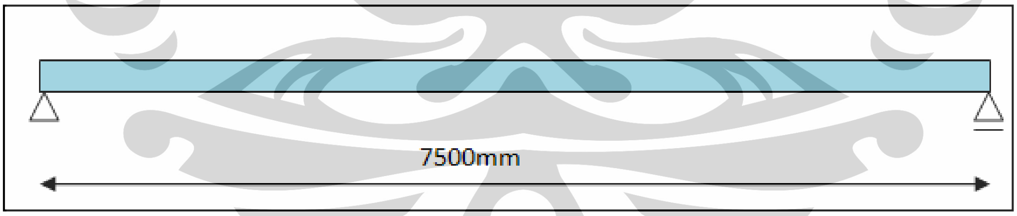

Figure 3.5. Simply Supported Span

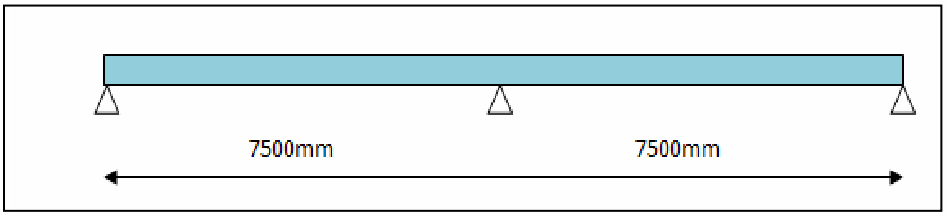

Figure 3.6. Continuous Span

3.3 Method of Investigation

3.3.1 Natural Frequency

An analytical approach and computer analysis (using SpaceGass and Patran) were conducted to determine the natural frequency of the grandstand. Three methods are used in lieu of only one method to calibrate the results:

1. The self-weight deflection approach is used to find the natural frequency for simply supported types of boundary condition. In this calculation, the structure is assumed to be act as a beam element. Firstly, the static deflection caused by the weight of the mass can be calculated using Equation (2.3):

   $$\Delta = \frac{5}{384}\frac{wL^4}{EI}$$

   Where

   w = weight of the composite material, concrete + polystyrene

   L = Length of the span, 7500mm

   E = Elastic modulus of concrete, since the elastic modulus of plastic is very small, thus the stresses are taken only by the concrete.

   I = second moment of inertia of the concrete, since the plastic does not take any stresses.

   The next step is finding the natural frequency from the deflection calculated above. For the simply supported beam, the natural frequency is found using Equation (2.2):

   $$f = \frac{1}{2\pi}\sqrt{\frac{g}{\Delta}}$$

2. The component frequency approach is used to find the natural frequency of continuous beams. the analytical solution may be written as:

$$f = C_B \left(\frac{EI}{mL^4} \right)^{1/2} \tag{2.14}$$

where

m = the mass per unit length

L = the span of 7.5m

C_B = frequency factor from Figure 3.7

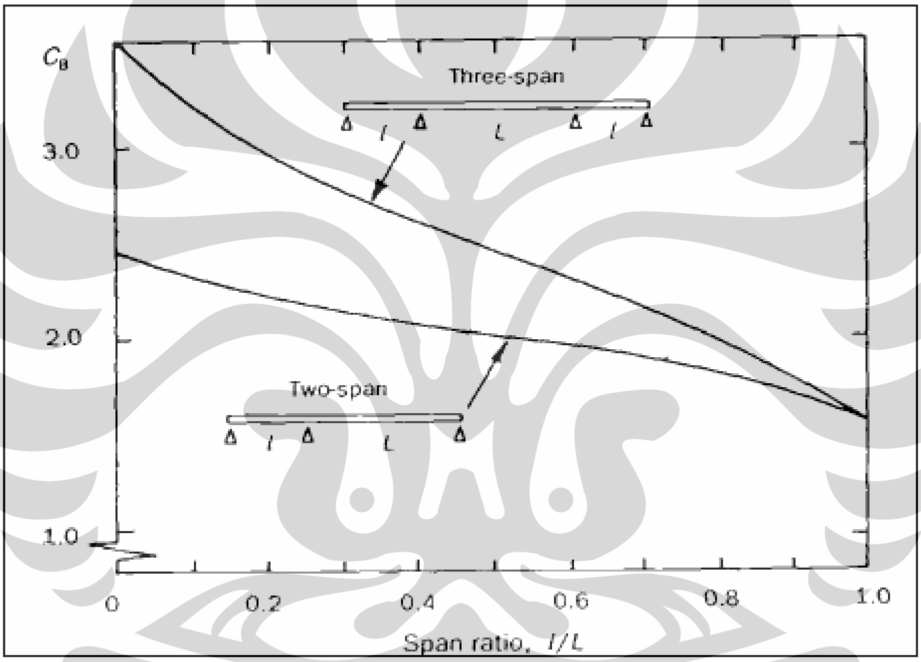

Figure 3.7. Frequency Factor C_B for Continuous Beams

3. SpaceGass Analysis Packages is a general purpose structural analysis and design program. The grandstand structure is modelled as a beam element and using a composite material properties. The dynamic frequency analysis module in SpaceGass uses the self mass in calculating the natural frequency and also the mode shape for any defined number of vibration modes.

   The material properties needed as input parameters are Elastic/Young's Modulus, Density and the Poisson's Ratio. In this case, the Elastic modulus to be used is the concrete elastic modulus due to the elastic modulus of concrete is very large, 20000 MPa, compared with the elastic modulus of plastic which is only 2000 MPa. Thus, the concrete will act to taking all the stresses and the plastic will only take a very small amount of stresses and can be ignored in the calculations.

4. Patran/Nastran FEM Analysis Package – The solver is MSC.Nastran 2005 and the pre and post-processing is using MSC.Patran 2005. Two different materials were considered, the fibre reinforced concrete as the shell and the polystyrene elastomer core. The structure is to be modelled as a HEXA 8 noded element. The element sizes are 20mm x 20mm for cross section and 100mm longitudinally. The use of Finite Element Method Analysis was to find the natural frequency required to get the better results and to get a better overview of what will happen in the grandstand structure. Figure 3.8 shows the boundary constraints were used in modelling of the L-Shaped structure in Patran, whilst Figure 3.9 shows the meshes that were used.

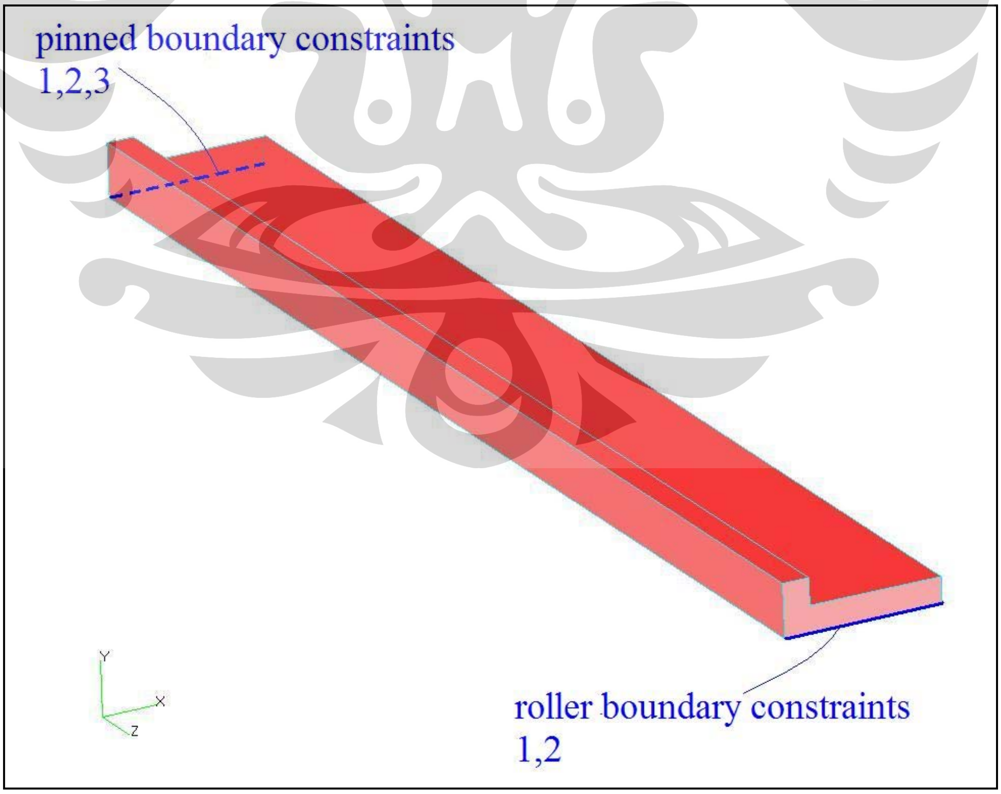

Figure 3.8. Bounday Constraints in Patran Modelling

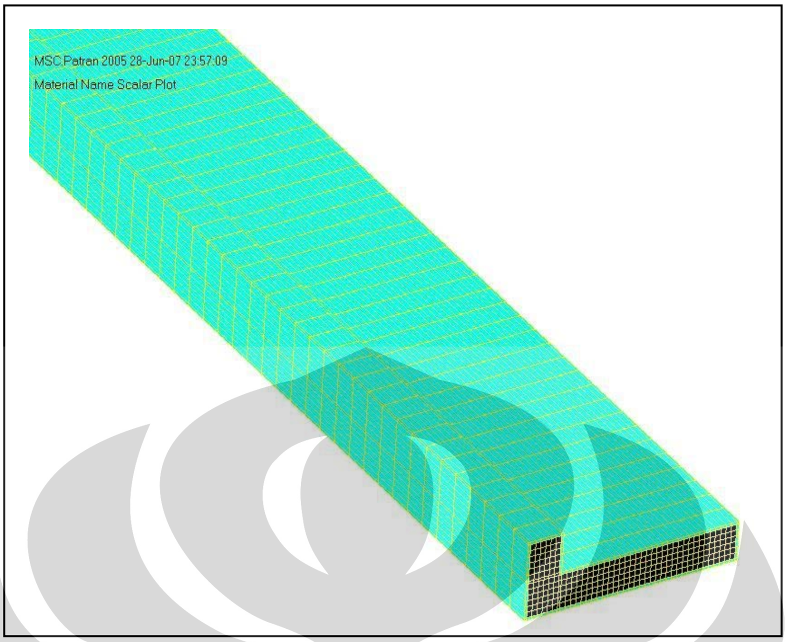

Figure 3.9. Meshes in Patran Modelling

3.3.2 Acceleration Limit

Ad discussed in literature review, there are two excitations caused by dynamic loadings which should be considered in designing a grandstand:

a) Walking excitation

The criterion for a floor structure to satisfy the guidance is when the peak acceleration, a_p, as a fraction of the acceleration of gravity does not exceed the acceleration limit a_o/g for the appropriate occupancy. Following is the Equation (2.7) to determine the peak acceleration from the forcing frequency:

$$\frac{a}{g} = \frac{R\alpha_i P}{\beta W}.\cos\left(2\pi i f_{step} t\right)$$

Where

R = taken as o.5 for floor structures

P = varies from single person up to the crowd loads (taken as 5 persons)

β = damping ratio, varies from 1%-3%

In this equation, the damping ratio, β, is considered as a variable which varies from 1% up to 3%. The value of 3% is taken as the maximum damping ratio being considered due to the relationship between the damping and the prices. The higher damping ratio increases the price of the material because the damping ratio can be tailored to meet the criteria.

For evaluation, the peak acceleration due to walking excitation can be estimated by selecting the lowest harmonic, fundamental frequency, for which the forcing frequency can match the natural frequency of the structure. For the design purposes, the formula which is approximating the step relationship between the dynamic coefficient, α_i, and frequency is used. Following is the simplified design criteria (Equation 2.8):

$$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W} \leq \frac{a_o}{g}$$

b) Rhythmic Excitation

The design criterion is based on the dynamic loading function for rhythmic activities and the dynamic response of the floor structure (Allen 1990a):

$$f_n \geq (f_n)_{req'd} = f\sqrt{1 + \frac{k}{a_o / g}\frac{\alpha_i w_p}{w_t}}$$

Where

fn = the natural frequency from the calculation above

k = a constant shown in table 2.4

The peak acceleration was calculated using this equation, whereas the input parameter will be the natural frequency, the forcing frequency, the dynamic coefficient and the effective weight of the grandstand and the participants. Thus, the peak acceleration will be obtained through the back or reverse calculation.

4 RESULTS AND ANALYSIS

This chapter presents the investigation results. The analysis and comparison of the results with the various types of analysis using hand calculations and computer programs are also presented to evaluate the designed demountable grandstand stadium using sandwich plate system.

4.1 Investigation Results

4.1.1 Methods of Calculations/Analysis

The use of Patran Finite Element Analysis Method in determining the natural frequency and the mode shape of the grandstand was undertaken to verify the likelihood of the grandstand behaviour due to the dynamic loadings. Since the modelling of the composite sandwich plate system grandstand in Patran used HEXA element or 3D element of two different materials, concrete and plastic, the results and the structure behaviour are more accurate. Finite Element Analysis clearly shows that almost all stresses were taken by the fibre reinforced concrete due to the big differences between the elastic modulus of fibre reinforced concrete and the polystyrene (see Figure 4.1 – stress distribution). It means that the plastic only contribute as a lightweight filling material and the damping provider. Thus, the assumptions were used in the investigation using hand calculation and SpaceGass analysis methods are accurate. The assumptions taken were considering the material properties of polystyrene, elastic modulus and Poisson’s ratio was ignored and only using the elastic modulus and Poisson's ratio of fibre reinforced concrete. Additionally, the second moment of area or moment inertia of the plastic also was not considered in the calculation. Although the polystyrene does not contribute in determining the stresses of the structure, the self-weight or the density of polystyrene still should be taken into account in determining the deflections and also the natural frequency of the grandstand. See Appendix A for detailed calculations.

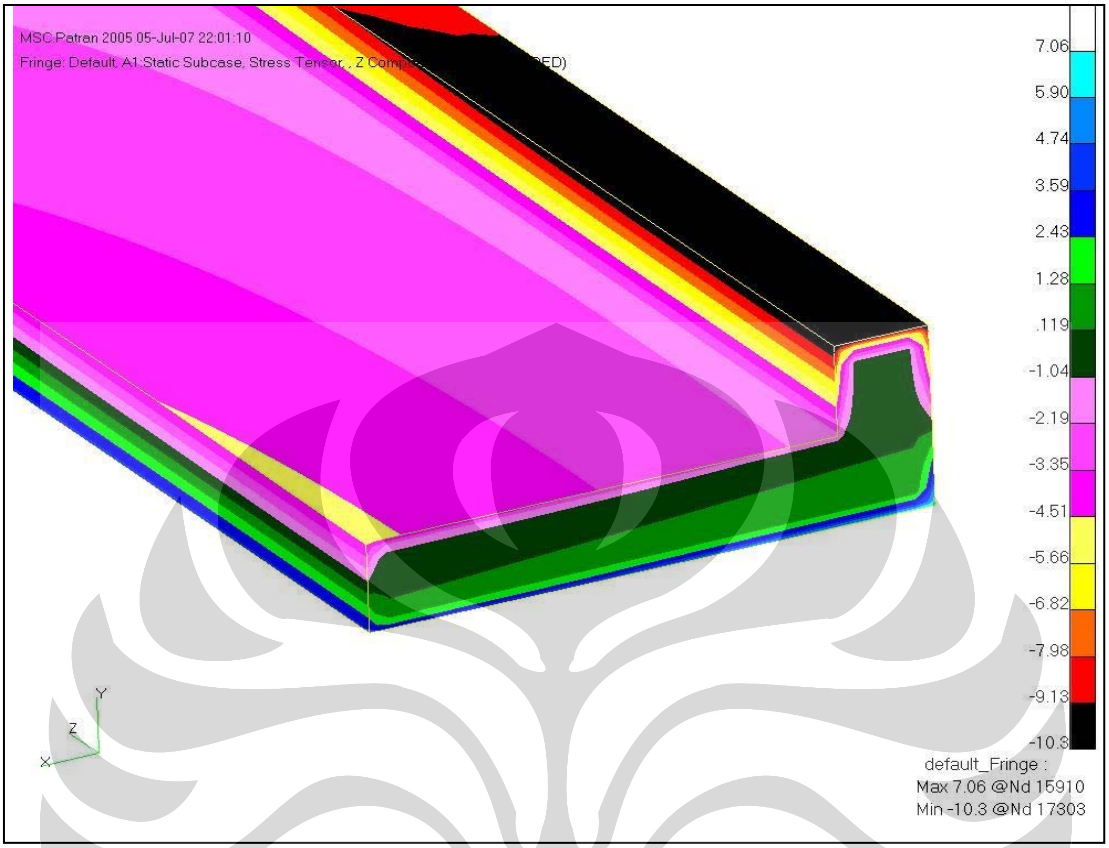

Figure 4.1. Stress Distribution

4.1.2 Structural Deflections

Structural deflection is also an issue in analysing the structural behaviour. As shown in Figure4.2, the deflection of simply supported L-Shaped section is greater than a continuous two spans beams L-Shaped section and also the Double-L-Shaped section. It can be seen that the continuous beams induced smaller deflection due to the self-weight effects of adjacent spans combined to reduce the corresponding deflections. Furthermore, the simply supported Double-L-Shaped section has smaller deflection than for the L-shaped. It is because the double-L-shaped structure is stiffer than the L-Shaped; therefore, with the same length of span, the double-l-shaped will be produced smaller deflection. Figure 4.3 is the deflection shape of the simply supported L-Shaped section from Patran Finite Element Modelling.

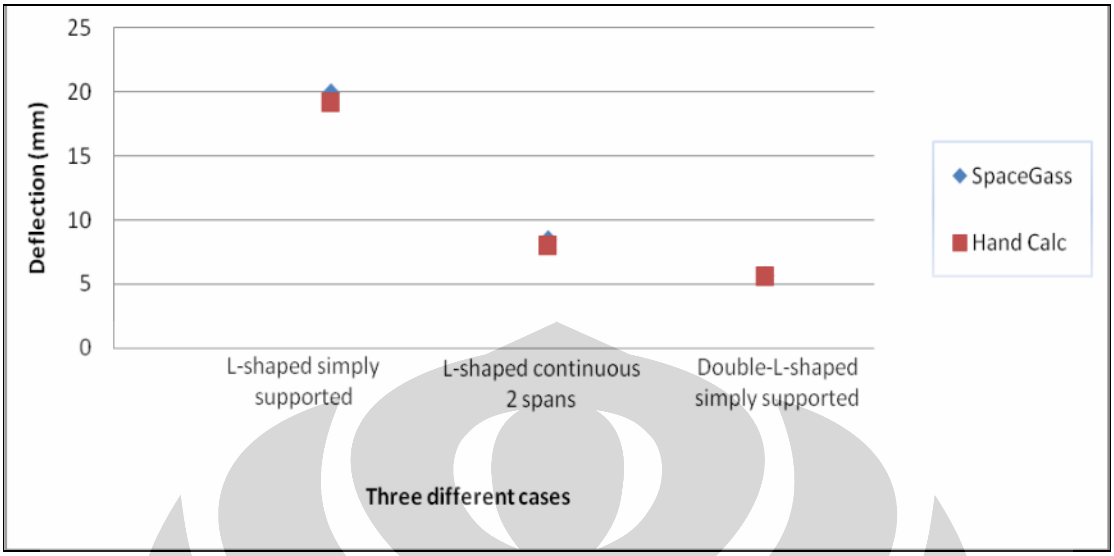

Figure 4.2. Deflection Results using Hand Calculation nd SpaceGass

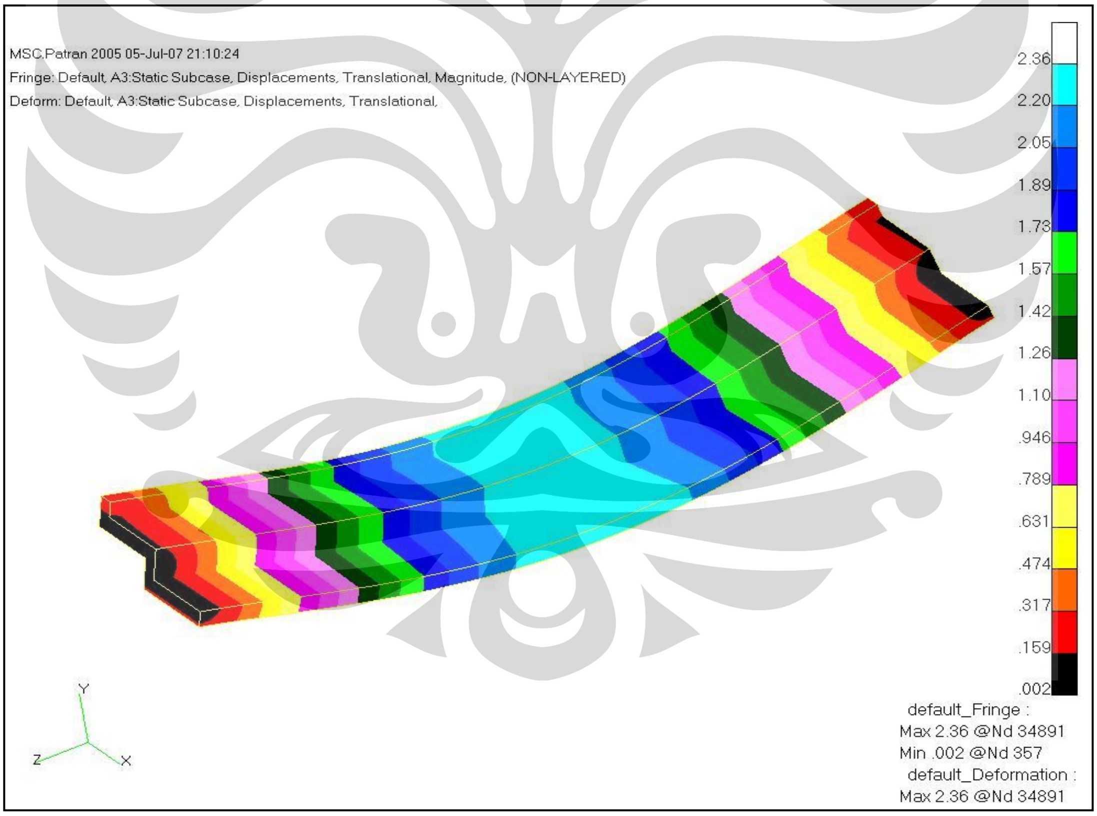

Figure 4.3. Deflection Curve-Patran

4.1.3 Natural Frequency

The results investigation of the natural frequency of grandstand stadium with three different cases using three different methods of analysis are shown in Figure 4..5, Figure 4.6 and Figure 4.7 for the simply supported L-shaped, two continuous span L-shaped and simply supported double-L-shaped, respectively. Figure 4.2 presents the results of the deflection for different cases and also from different method of calculations. Figure 4.4 shows the fundamental natural frequency with its mode shape. The complete results of natural frequency calculation can be found in Appendix A and the complete mode shapes can be found in Appendix B.

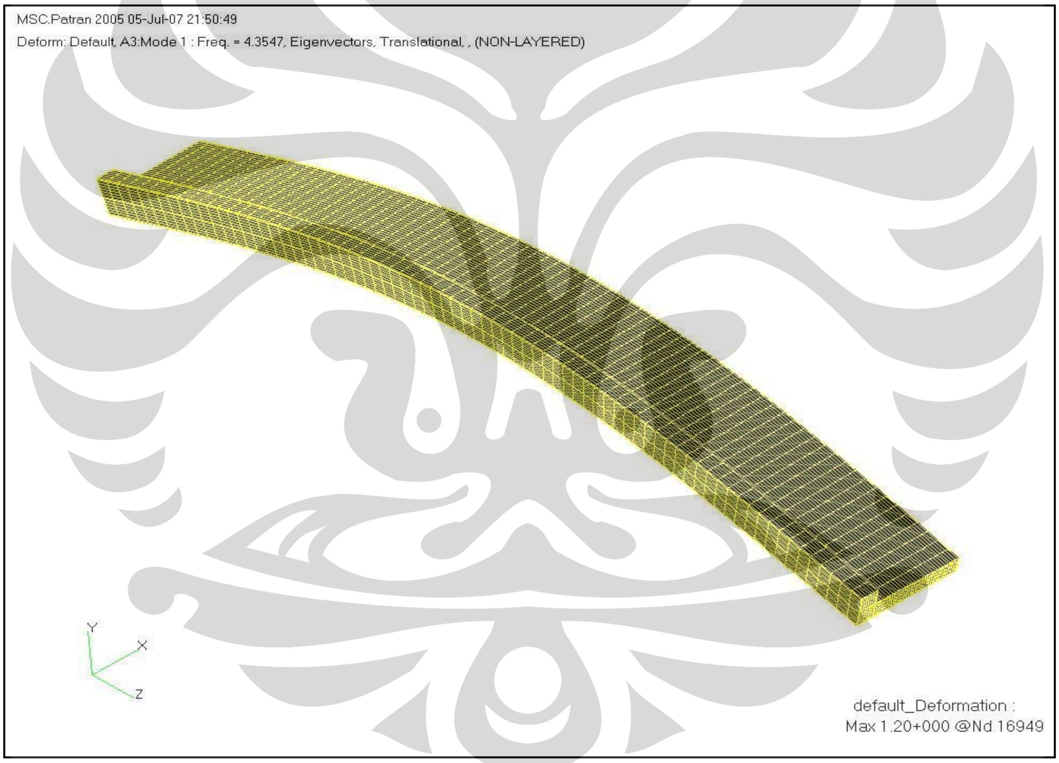

Figure 4.4. Natural Frequency with Mode Shape

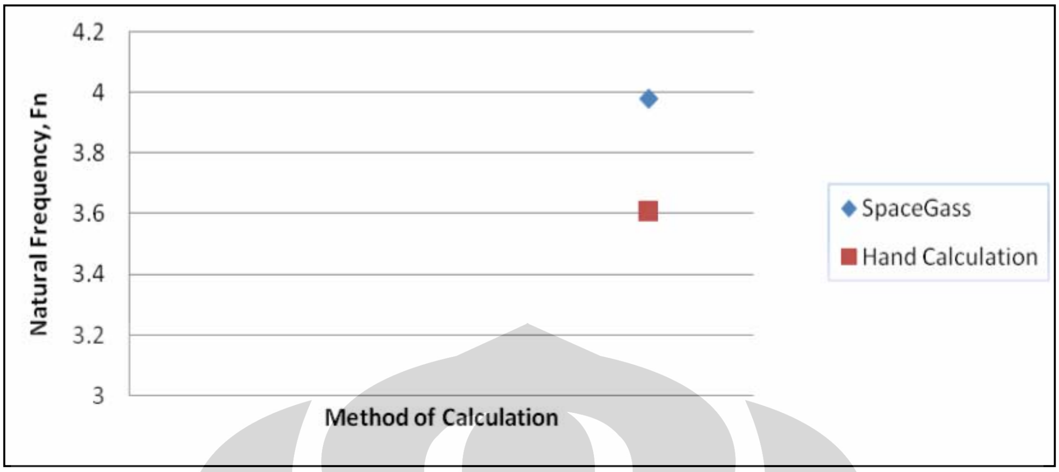

Figure 4.5. Natural Frequency of L-Shaped Simply Supported Beam

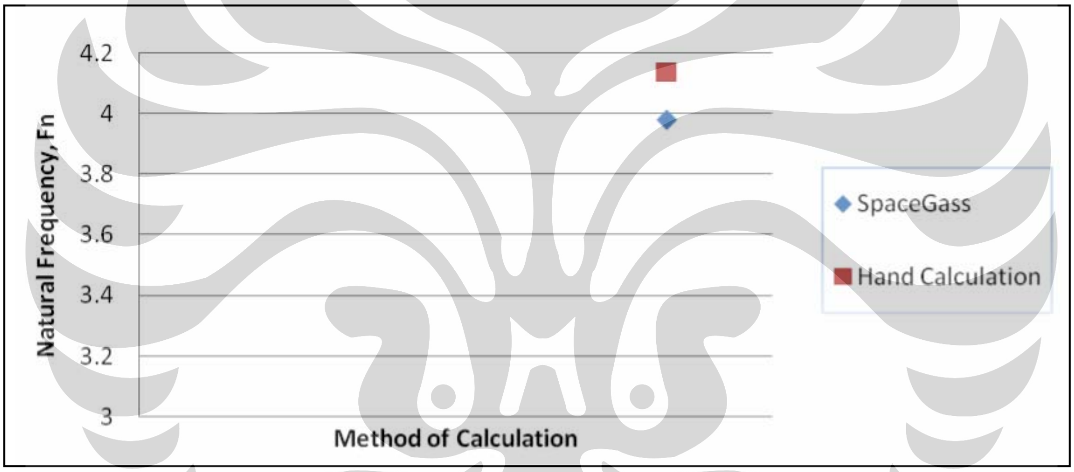

Figure 4.6. Natural Frequency of L-Shaped Continuous Beam

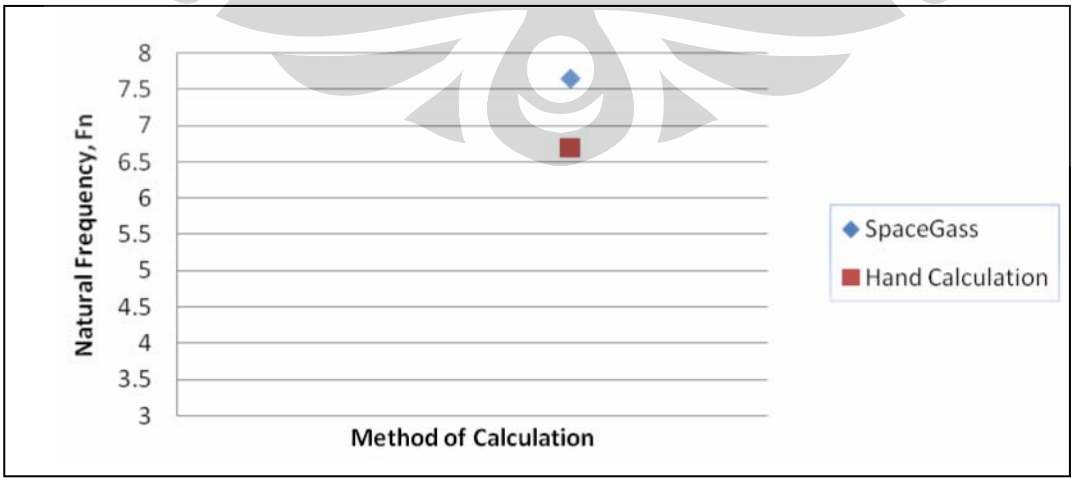

Figure 4.7. Natural Frequency of Double-L-Shaped Simply Supported Beam

From the investigation results, it shows that the natural frequency obtained by using three different methods of calculation and analysis are reliable. The differences between one and other methods are small enough. For the L-Shaped simply supported type of grandstand, the natural frequency is found to be 3.979 Hz and 4.35 Hz using Space Gass and Patran/Finite Element Analysis, while from hand calculation, the natural frequency is 3.606 Hz. It means that the assumptions taken for hand calculation and beam element analysis using SpaceGass were appropriate to be considered and used.

Even though the proposed boundary constraint for the grandstands is simply supported with pinned and roller supports, in practical or real conditions, the grandstand would not be acting as a fully simply supported structure. It would behave in a condition between simply supported and fixed constraint conditions due the floor systems which allow the floors to interact each other although the system is designed to act independently. Those values in between as shown in Figure 4.8 are the reasonable values that might be achieved by the structure. However, in the design consideration regarding safety issues, the simply supported boundary constraint is considered.

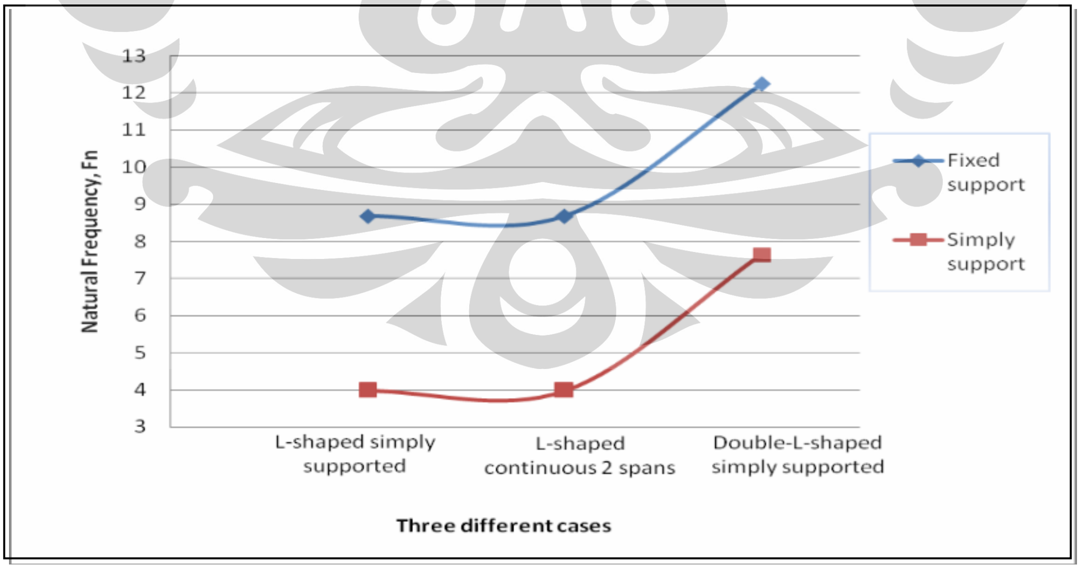

Figure 4.8. Natural frequency of Simply Supported Vs Fixed Support

The following is summarizing of Natural Frequency results from Space Gass analysis with three different cases of investigation from the fundamental frequency up to the fifth harmonic.

Table 4.1. Summarised of Deflections and Natural Frequencies

	Deflection mm	f_n (Hz)				
		1	2	3	4	5
Simply Supported L-Shaped	19.913	3.979	15.763	27.793	36.2922	60.731
Contunuous Span L-Shaped	8.437	3.979	6.14	15.763	19.404	27.793
Simply Supported Double-L-Shaped	5.65	7.642	26.971	34.229	61.32	86.975

4.1.4 Peak acceleration criteria

From the natural frequency calculation and analysis, the natural frequency used in determining the peak acceleration is the result from Space Gass analysis as shown in Table 4.1

4.1.4.1 Walking Excitation

The investigation results of peak acceleration criteria of the grandstand for the walking excitation are shown in Table 4.2 and 4.3, for both L-Shaped and Double-L-Shaped cross sections are based on the forcing frequency using Equation (2.7), while Table 4.4 shows the peak acceleration based on the natural frequency using Equation (2.8).

By calculating the peak acceleration of walking excitation based on the recommended forcing frequency (Table 2.1) and taking the damping ratio as one percent, the peak acceleration achieved was 5.83% gravity with the assumption the grandstand is full of people or in crowd conditions, as shows in Table 4.7a and Table 4.7b. Those values are the same for both the L-Shaped and Double-L-Shaped sections, although the cross sections are different. It is because the weight of the Double-L-Shaped is twice as the L-Shaped and the number of people in the crowd is also twice as much as the L-Shaped. Thus, the peak acceleration will end up with the same values. However, the peak acceleration of 5.83% gravity had not passed the recommended acceleration limit of 5% gravity (Allen and Murray, 1993; ISO 2631-2: 1989). Therefore, the damping ratio of

2% and 3% were considered and the peak acceleration achieved were 2.91% and 1.94%, respectively. Since the use of 2% damping ratio had passed the recommended acceleration limit, the damping ratio of 2% was undertaken as the design consideration in this design stage for walking excitation. Furthermore, the Double-L-Shaped was preferred rather than the L-Shaped due to easier construction issues of the Double-L-Shaped section.

Table 4.2. Peak Acceleration for L-Shaped Section

No of ppl	a/g		
	Damping Ratio		
	1%	2%	3%
1	1.17	0.58	0.39
5	5.83	2.91	1.94

Table 4.3. peak Acceleration for Double-l-Shaped Section

No of ppl	a/g		
	Damping Ratio		
	1%	2%	3%
1	0.58	0.29	0.19
2	1.17	0.58	0.39
10	5.83	2.91	1.94

By assuming the resonance condition where the natural frequency is equal to the forcing frequency, the peak acceleration due to the walking excitation of the grandstand structure is summarized in Table 4.4. It shows that only the Double-L-Shaped with simply supported boundary condition passed the recommendation acceleration limit of 5% (Allen and Murray, 1993; ISO 2631-2: 1989) with peak acceleration of 4.96% and 3.30% gravity for both the damping ratio of 2% and 3%, respectively. Thus, for the walking excitation investigation, the Double-L-Shaped is the most suitable shape for the designed grandstand stadium due to ita stiffness that will reach higher natural frequency and also lower peak acceleration percentage of gravity.

Table 4.4. Peak Acceleration Limit

β (%)	ao/g (%)		
	Simply Supported L-Shaped	Continuous Span L-Shaped	Simply Supported Double-L-Shaped
1	71.44	71.44	9.91
2	35.72	35.72	4.96
3	23.81	23.81	3.30

4.1.4.2 Rhythmic Excitation

Table 4.5 summarizes the peak accelerations and the natural frequencies for all different types of shaped and support conditions. The use of L-Shaped with simply supported condition induced peak acceleration of 18.941% of gravity while used for dancing activity, 23.247% of gravity for lively concert or sport event and 23.729% for jumping exercises activity. Since the recommended acceleration limit for the rhythmic activity is 5% gravity (Allen and Murray, 1993; ISO 2631-2: 1989), the peak accelerations for simply supported L-Shaped model seem to have unsatisfied the design criteria. It will exceed the acceleration limit while the grandstand is subjected to dancing, lively concert or sport event and jumping types of activities. The similar results for the L-Shaped continuous span, the peak acceleration for all types of activity also failed compared with the acceleration limit of 5% gravity.

Table 4.5. Peak Acceleration for Rhythmic Excitation

Activity	Simply Supported L-Shaped		Continuous span L-Shaped		Simply Supported Double-L-Shaped	
	fn (Hz)	ao/g (%)	fn (Hz)	ao/g (%)	fn (Hz)	ao/g (%)
Dancing:						
First Harmonic	3.979	18.941	3.979	18.941	7.642	2.620
Lively Concert or sport event:						
First Harmonic	3.979	23.247	3.979	23.247	7.642	3.215
Second Harmonic	15.763	0.395	6.14	6.948	26.971	0.126
Jumping exercises:						
First Harmonic	3.979	23.729	3.979	23.729	7.642	3.860
Second Harmonic	15.763	1.439	6.14	42.148	26.971	0.450
Third Harmonic	27.793	0.167	15.763	0.653	34.229	0.107

Different from the L-Shaped section, the Double-L-Shaped section with simply supported boundary condition has passed the entire peak acceleration criterion, for dancing, lively concert or sport event and jumping exercises. The higher natural frequency of the Double-L-Shaped section compared with the L-Shaped sections explains a decrease of peak acceleration limit percentage. It can be concluded that Double-L-Shaped with simply supported boundary condition is the only option that passed the rhythmic peak acceleration criterion.

4.2 Discussion

From the above analysis, it shows when an L-Shaped section either as a simply supported beam or as a continuous beam is used; the grandstand structure failed the dynamic performance criterion due to the low natural frequency and the high peak acceleration. Whilst, when the Double-L-Shaped section with simply supported boundary condition is used, it satisfied the dynamic performance criterion of the grandstand stadium.

As the composite material used was the Glass Fibre Reinforced Concrete as the shell material and Polystyrene as the elastomer core, the large differences of the Elastic Modulus between those two materials made the concrete took almost all the stresses and the polystyrene only took a small portion of the stresses. Thus the polystyrene only acted as a light weight core material which provides high damping ratio.

Table 4.6. Summarized of Deflection and Natural Frequency

Cross Section	Deflection mm	Natural frequency Hz
L-Shaped	19.11	3.98
Double-L-Shaped	5.65	7.64

It can be concluded that the structure with stiffer shapes or materials will perform better under static and dynamic loadings. The reasons can be seen from the deflection results between L-Shaped and Double-L-Shaped and from the natural frequency results listed in Table 4.6. The deflection of L-Shaped is more than twice larger than the Double-L-Shaped section. And the same case with natural frequency which is twice smaller than the Double-L-Shaped section.

The peak acceleration for walking excitation is found to be less than the acceleration limit of 5% only for the Double-L-Shaped with 4.96% gravity while using the damping ratio 2%. However, the peak acceleration of 4.96% is too close to the acceleration limit. Thus, due to the safety reasons, the damping ratio 3% should be taken as it will provide better peak acceleration with 3.30% gravity. In fact, as the only section that had passed the acceleration limit for walking excitation is the Double-L-Shaped, the L-Shaped did not fulfil the peak acceleration criterion with the damping ratio of 3%. Thus, the L-Shaped section could perform better by increasing the damping ratio, but it means the prices will be increased as the higher the damping ratio, the material becomes more expensive. Therefore, it is not recommended to use the high damping ratio.

For rhythmic excitation, the acceleration limit that the grandstand will achieve is 3.86% of gravity; it is when the structure is subjected to jumping exercises type of activity. Similar to the walking excitation, the only cross section that passed the peak acceleration limit is only the Double-L-Shaped while the L-Shaped has peak acceleration much larger than the recommended limit. Therefore, the Double-L-Shaped is the most suitable section to be chosen.

Nevertheless, due to the limited time in this thesis, a proper investigation and experiment using the appropriate dynamic loadings has not been undertaken. Therefore, further investigation is recommended to fully discover the behaviour of the grandstand performance under the dynamic loadings.

5 CONCLUSION AND RECOMMENDATIONS

The main objective of this thesis is focused on the evaluation of the dynamic performance of grandstand stadium with different types of cross sections using fibre reinforced concrete and polystyrene as the sandwich plate system composite material. This thesis were commenced with literature review, followed by investigation studies, and ended by analyses of the results. Analytical analysis using design guide of floor vibration by Allen & co and also by Wyatt A. T. and interim design guide for grandstand stadium were used for investigation of the grandstand dynamic performance. The investigations were using manual calculations and also software analysis. The analysis was also supported by numerical studies using finite element methods to further study the behaviour of L-Shaped structures.

From the investigational and numerical studies, the following conclusion can be drawn:

- Finite element analysis of L-Shaped section indicated that almost the stresses of the structures were taken by the Glass Fibre Reinforced Concrete and the Polystyrene only took the small portion of the stresses. It is due to the big differences of the Elastic Modulus of those two materials. The elastic modulus of Glass Fibre Reinforced Concrete is ten times bigger than the elastic modulus of Polystyrene.

- The investigation revealed that the L-Shaped section have smaller natural frequency than the Double-L-Shaped due to the structure stiffness. Double-L-Shaped section is twice stiffer than the L-Shaped section. Thus, the Double-L-Shaped can perform better then the L-Shaped section under dynamic loadings and fulfil the peak acceleration limit criterion using 3% damping ratio of the structure.

- The investigation finding that the Sandwich Plate System using Fibre Reinforced Concrete and Polystyrene as the composite material and using the Double-L-

Shaped as the cross section with the span of 7.5m will perform safely under the dynamic loadings.

Regarding future researches, following is the recommendation:

- Due to the time constraint, the experimental investigation of grandstand under dynamic loading might be required to verify the results from the hand calculation method of analysis and to fully discover the behaviour of the grandstand performance.

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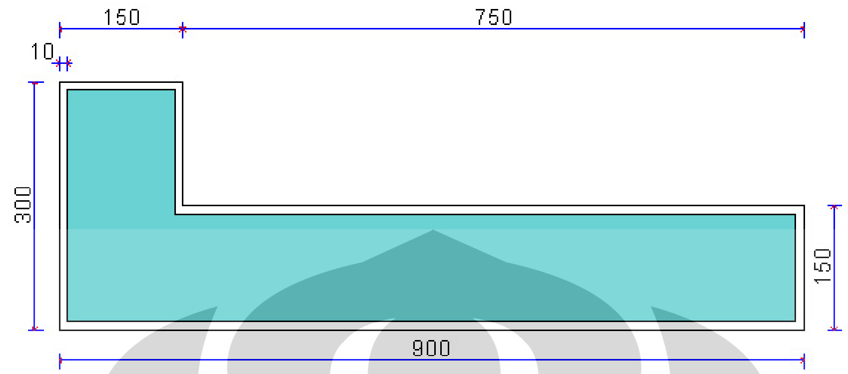
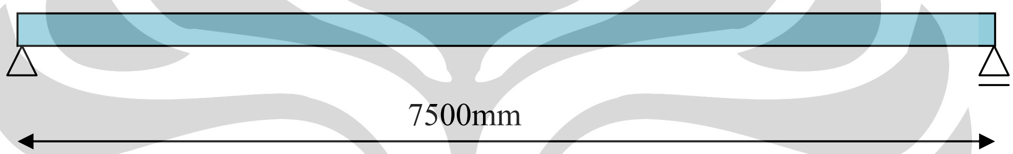
APPENDIX A

- **Calculation of Natural Frequency for Simply Supported L-Shaped Section**
- **Calculation of Natural Frequency for Continuous Span L-Shaped Section**
- **Calculation of Natural Frequency for Simply Supported Double-L-Shaped Section**

Simply Supported L-Shaped





Material Properties:

	FRC	Polystyrene	Combined
Density (kg/m3)	1800	1130	1230.39
Elastic Modulus (Mpa)	20000	2000	4697.14
Poisson's Ratio	0.26	0.33	0.32

A = 0.1575 mm^2

$I_{concrete}$ = 2.05 x 10^8 mm^4

w = 1.901 N/m

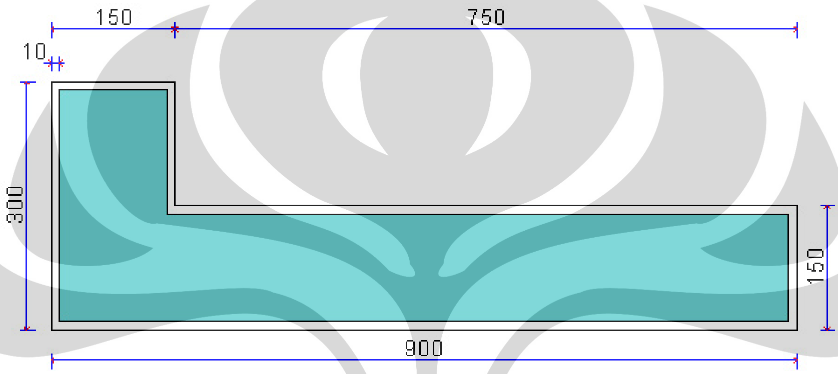
Displacement:

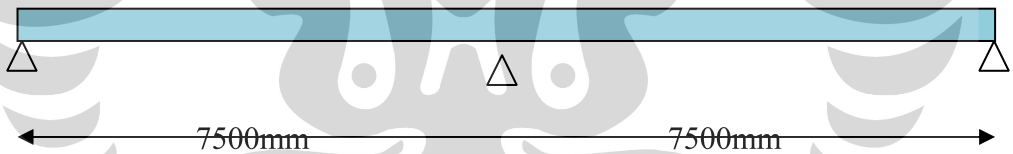
$$\Delta = \frac{5wL^4}{384EI} = \frac{5 \times 1.901 \times 7500^4}{384 \times 20000 \times 2.05 \times 10^8} = 19.113mm$$

Natural Frequency:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{g}{\Delta}} = \frac{1}{2\pi}\sqrt{\frac{9810}{19.113}} = \underline{\underline{3.606 Hz}}$$

Continuous Span L-Shaped





Material Properties:

	FRC	Polystyrene	Combined
Density (kg/m3)	1800	1130	1230.39
Elastic Modulus (Mpa)	20000	2000	4697.14
Poisson's Ratio	0.26	0.33	0.32

A = 0.1575 mm^2

$I_{concrete}$ = 2.05 x 10^8 mm^4

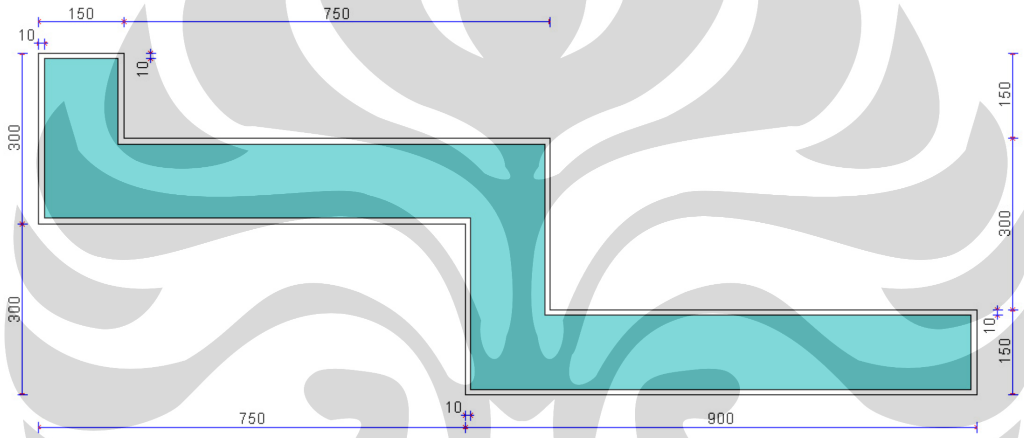
C_B = 1.6 (frequency factor from the component frequency approach)

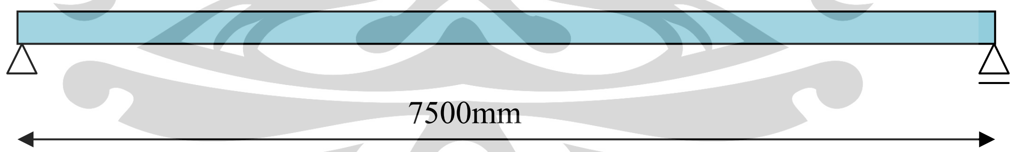
m = 193.79 kg/m

Natural Frequency:

$$f_n = C_B \left(\frac{EI}{mL^4} \right)^{1/2} = 1.6 \left(\frac{20000 \times 2.05 \times 10^8}{193.79 \times 7.5^4} \right)^{1/2} = \underline{\underline{4.137 Hz}}$$

Simply Supported Double-L-Shaped





Material Properties:

	FRC	Polystyrene	Combined
Density (kg/m3)	1800	1130	1227.63
Elastic Modulus (Mpa)	20000	2000	4622.86
Poisson's Ratio	0.26	0.33	0.32

A = 0.315 mm^2

$I_{concrete}$ = 1.41 x 10^9 mm^4

w = 3.794 N/m

Displacement:

$$\Delta = \frac{5wL^4}{384EI} = \frac{5 \times 1.901 \times 7500^4}{384 \times 20000 \times 1.41 \times 10^9} = 5.533mm$$

Natural Frequency:

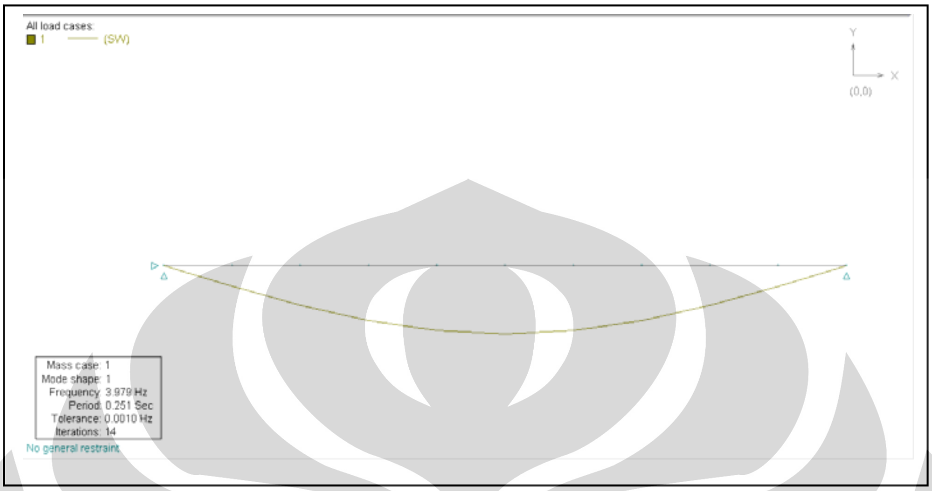
$$f_n = \frac{1}{2\pi}\sqrt{\frac{g}{\Delta}} = \frac{1}{2\pi}\sqrt{\frac{9810}{5.533}} = \underline{\underline{6.701Hz}}$$

APPENDIX B

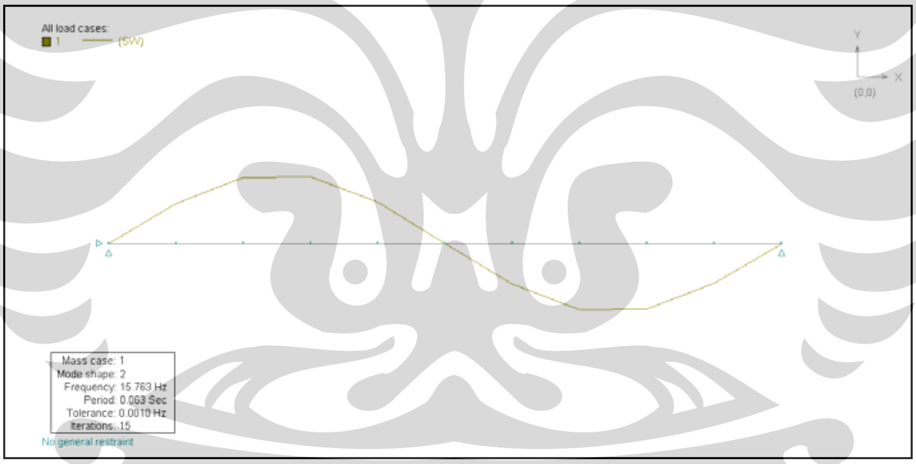
- **Analysis Results from Space Gass: Natural Frequency and Mode Shapes**

Simply Supported L-Shaped

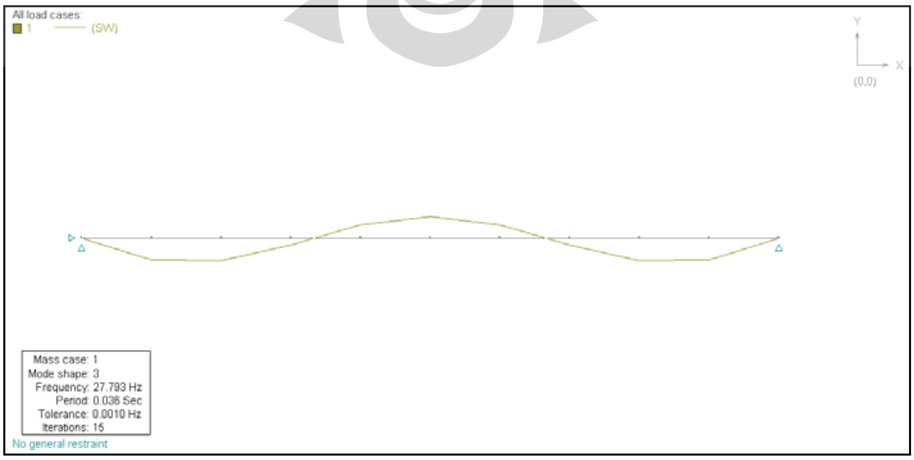
- Fundamental Frequency



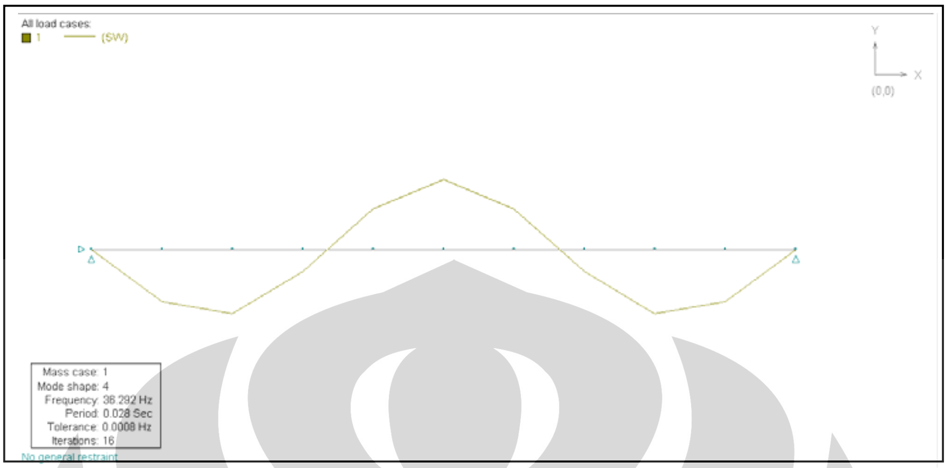
- Second Harmonic



- Third Harmonic



- Fourth Harmonic



- Fifth Harmonic