SHORE LOAD MONITORING DURING CONSTRUCTION

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ABSTRACT

Statistical accuracy of live and dead loads occurring in permanent structures has been known for many years. However, loads occurring during construction are known to be both highly variable and statistically less certain. Furthermore, the design of formwork has typically relied on engineering judgment, past experience and an empirical understanding of the construction process. The purpose of this work is to determine and compare the mean and theoretical axial loads occurring among shores in scaffold-type formwork supporting systems. A detailed on-site survey was conducted across three varying construction sites in Sydney, Australia. In total, ten pour areas were investigated, accumulating valuable data. Both dead and live construction loads were measured prior to, during and post concrete pour.

KEYWORDS

Shore
Load
Construction
Survey
Relative Load
Mean to Nominal
Construction Live Load
Construction Dead Load
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LITERARY REVIEW

INTRODUCTION

The following research report aims to provide an explicitly detailed synopsis of a two year shore load monitoring investigation. The report first provides a brief overview of the history and development of support scaffolding systems, including joint types and types of materials used. The configurations and components of support scaffolding is then detailed to understand how these lightweight and slender structures are erected and remain structurally stable. The identification and critical findings from previous scaffolding collapses are interrogated to qualify the various types of failure modes. A detailed literary review of past research into construction live and dead loads as well as relative shore loads is then described.

Steel scaffolds are used as support structures in reinforced concrete construction. Statistical information of applied live and dead loads on permanent structures have been known for many years. However, loads occurring during construction are known to be both highly variable and statistically less certain. The design of formwork has typically been the responsibility of the subcontractor and thus has relied on engineering judgment, past experience and an empirical understanding of the construction process. It appears that an engineered approach is becoming more and more applicable, particularly considering the frequency of failures during construction compared with failures during service. Although steel scaffolds are temporary structures, their failure often has fatal consequence. A survey of falsework collapses has shown that 74% of falsework failures occurred during concrete placement operations and are a consequence of overloading (Hadipriono, 1987).

A two-year long shore load monitoring investigation was undertaken in Sydney, Australia. The investigation aimed to determine and compare the mean and nominal axial loads occurring among shores in scaffold supporting systems. A detailed on-site survey was conducted across three varying construction sites. In total, 188 shore load measurements were recorded in ten site investigations, accumulating valuable data. Both dead and live construction loads were measured prior to, during and post concrete pour.

Past research has focused on the applied load (i.e. actual loads on the slab) occurring during construction, through spatial inventory surveys of equipment loads (H. Ayoub, & Karshenas, S., 1994). Attention to the shore load effect (i.e. the amount of load which a vertical shore receives) has been limited to a few papers investigating these effects (D. Rosowsky, & Stewart, M., 2001), (G. Fattal, 1983), (J. Ikaheimonen, 1997). This paper and the site investigations that it documents, is thus considered a valuable and important contribution to the lack of statistical information regarding shore loads and the shore load effects. Furthermore, the data collected as part of this investigation is deemed to be valuable due to its comprehensiveness, the difficulty in accessing the job site and the time taken to install and gather such a complete data set. This paper seeks to address the load requirements on scaffolding systems, in order to produce a probabilistic / statistical model for loads imposed on support scaffolds. The work presented in this paper is part of an ongoing project which aims to develop a probability-based design methodology for scaffold-type formwork supporting systems.

SCAFFOLDING SYSTEMS

Scaffolding Systems are used as temporary support for structures under construction or those needing repairs. Their primary function is to support various types of loads. These loads include, the vertical loads imposed by laborers, construction equipment, frameworks, and construction materials. Scaffolds must also be designed to withstand lateral loads, including wind loads, impact loads, and earthquake loads.

Scaffolding systems are typically modular to aid in erection and transportation. Furthermore scaffolding generally is slender in design, to reduce material usage for mass production and transportation.

History

Origins in scaffolding utilisation date as far back as ancient Greece. The Berlin Foundry Cup dates back to early 5th century BC, and its exterior, depicts a bronze workshop. The workshop produced a number of bronze statues, and there is wooden scaffolding used to support the statue of a warrior (Zimmer 2007). There is documentation of ancient Egyptians, Nubian’s, as well as Chinese using scaffolding structures to support buildings.
Materials

Many forms of materials have been used in the construction of scaffolding; timber and bamboo were common place scaffold materials and still are predominantly used across Asia. In western countries the industrial revolution introduced cold-formed steel to the scaffold industry, cold-formed circular hollow sections were considered advantageous due to their high strength (in both tension and compression) and usability. Cold form sections are modular in nature and are typically erected and deconstructed each time they are reused. Being modular also means that transportation becomes much easier as they are deconstructed into short tubular components. In recent years, the drive for efficiency in construction has paved the way for aluminium scaffolding to be used in construction, since it is lighter, easier to handle and reduces gross tonnage for transportation.

Steel Scaffolding Classification

Scaffolding is classified as either access or support scaffolding, depending on its use or application. Access scaffolds are typically used around the perimeter of buildings are used to provide access for workers on a construction site. Access scaffolds are designed to support small loads from workers and their equipment, and their single bay design is tied to a building for lateral stability. On the other hand, support scaffolds are those typically referred to as ‘false work’ and are used as platforms to support timber formwork for reinforced concrete slab construction. Support scaffolds are typically heavily loaded under the weight of formworks, newly poured concrete, stacked materials and construction workers. Support scaffolds have been the focus of this investigation.

Figure 1-1: Typical scaffold systems: (a) access scaffold, and (b) support scaffold

Support scaffolding systems are used to support an array of timber bearers which pass through the top “U-head” connection. These bearers are orientated along their strong axis and typically span between three U-head connections. Timber joists are then placed on top of the bearers in a perpendicular direction at various centers, typically approx. 500mm apart. A 17mm laminated plywood panel deck is then applied without mechanical anchoring onto the array of joists. This entire system is known as the ‘formwork system’ supported by the scaffolding system or ‘false work system’. In some cases the metal decking products, Bondek, Condek etc. are used. In these cases, the metal deck spans between the timber bearers with no need for joists of plywood.

SCAFFOLD CONFIGURATIONS

SSS is commonplace forms of temporary support within the Australian construction industry and have basically remained similar in their configuration and component usage. Support scaffold systems are typically constructed from a modular system of circular hollow steel tubes and feature joints which can be easily assembled for quick erection and dismantling. The common configurations of steel support scaffolds include standard door type, knee-braced door type and stick type. (See on Figure 1-2)
Figure 1-2: Various types of scaffold unit: (a) simple (knee-braced) door type; (b)-(e) standard door type; (f) Stick construction with Cuplok joints or wedge-type joints

This investigation considers only stick-type steel support scaffold systems with cuplok joints, more specifically Boral Formwork and Scaffolding Supercuplok Scaffolding System. Stick-type scaffolding systems are made up of a slender framework of ledgers (horizontal members), standards (vertical members), braces and jacks. (Figure 1-3) shows a typical ‘stick-type’ single bay SSS.

Standards and Ledgers
The standards are connected to create a lift via couplers, and also known as spigot joints (Figure 1-5), and to connect ledgers (horizontal members) to standards, Cuplok or wedge-type joints (Figure 1-6) are usually preferred because no bolting or welding is required. The steel tubes used for standards and ledgers typically have an outside diameter of between 42 mm to 48 mm with a wall thickness of 3 mm.

Cuplok
The cuplok joint actually transfers a partial bending moment in the joint as a result of higher joint stiffness’ at the Cuplok connection. The Cuplok connection increases the scaffolding strength and its load carrying capacity.

Bracing
Bracing is typically connected by simple hooks for rapid assembly; however some systems use pin-jointed couplers (T. Chandrangsu & Rasmussen, 2006). Various types of steel sections are currently used as bracing. Boral uses a brace constructed with two periscopic tubes that can slide inside one another to adjust the brace length.

Jacks
At the top of a scaffold system there are adjustable U-head steel screw jacks which ensure the formwork is of level and able to support the timber bearer’s, formwork and concrete. These top jacks extend to a maximum 600mm. Whilst jacks at the scaffolding base, adjustable by large wing nuts up to 600mm, allow for level setup in irregular ground.
Scaffold systems are adaptable to a particular job and as such they can range from one storey (lift) up to many storeys as well as have many bays and rows depending on the type of construction. Being a Temporary structure, scaffold members are reused from one job to another, and for that reason, quality control than is required to ensure that geometric imperfections, notably the crookedness of standards, remains within stipulated tolerances.

![Figure 1-3: Typical components and configuration of a single bay scaffolding system](image)

These stick-type scaffolds are prefabricated and assembled on site for quick erection. Prefabrication allows multiple bays to be erected depending on the requirements and therefore scaffolds can be erected from anywhere between 1.2-25m in height with any number of bays.

![Figure 1-4: Typical design of a Cuplok Joint (adopted from (T. Chandrangsu, & Rasmussen, K., 2006))](image)
Figure 1-5: Schematic of spigot joint

Figure 1-6: Schematic of (a) Cuplok joint; and (b) wedge-type joint
Figure 1-7: Schematic of brace connections: (a) hook connection; and (b) pin connection

Figure 1-8: Schematic of jack base
In general, scaffolding systems vary in height from 1.2m to 25m and are comprised of a number of lifts or individual panels, constructed using vertical 'standards'. These lifts are between 1.0m and 2.5m and are typically connected vertically by a spigot joint. The horizontal separation of standards is known as the bay size, and this is dictated by the size of 'ledger' used which vary from 0.7m to 2.5m. The versatility of scaffolding systems means that their configuration in all three axes can be varied depending on construction requirement.

**SCAFFOLD COLLAPSE**

The collapse of scaffolding structures is not only an economic burden but more importantly can endanger the lives of workers. There is a significant amount of research which suggests that failures of reinforced concrete structures occurring during construction, are in many cases traceable to the collapse of formwork shoring systems (Hadipriono, 1987) and (KL., 1987).

The recent buckling and part collapse of an access scaffolding in Sydney's CBD on February 26, 2009, forced a work shutdown and left workers fleeing. The failure of scaffolding at the John Hancock Centre in the US (Zimmer, 2007), highlights the potentially fatal consequences of overlooking the design and safety of scaffolding systems. Furthermore, the Guangxi (China) Medical University library accident in 2007 (Z. Y. Zhang, Ke, L., 2008) killed seven construction workers and is a recent example of a catastrophic failure of steel scaffold shoring system. Although only temporary structures, their failure often causes tragic consequences for workers and the public, as well as large legal and financial associated costs. There is therefore a prudent need to understand why other scaffolding systems have collapsed and what measures can impounded in the design process to mitigate the risk of failure.

**Findings from Scaffold System Failure Case Studies**

In order to minimize future failures in scaffolding structures, one must look at the failures of the past which justify the implementation of new codes of practice in the design of scaffolding systems. In most cases it is clearly evident that failure occurs as a standard result of inadequate accuracy in the evaluation of various acting loads and material strength in formwork design (Chen, 2009)).

**Bojnourd Cement Factory, Iran**

In the construction of a bypass clinker silo in the Bojnourd Cement Factory, the failure of the scaffolding system resulted in the collapse of a newly poured concrete slab. The collapse led to the death of three construction workers, the injury of seven others and a one-month delay in the project. Failure occurred when the supporting structure collapsed and caused the 600mm of freshly poured concrete to fall 11.5m.
A full forensic engineering investigation used Finite Element Methods, the modeling of scaffolding configurations and various analyses considering the presence and absence of proper lateral bracing and P-delta effects (Pisheh, 2009), (Chen, 2009).

This full forensic investigation, which included the findings of the forensic engineers, determined that the main reasons for collapse where (Peurifoy, 1995):

- Inadequate shoring or support elements
- Incorrect stripping and shore removal
- Insufficient bracing of members – specifically, weak lateral bracings in two orthogonal directions
- Deficiency in control of the rate of concrete placement
- Improper or inadequate connections in vertical elements of scaffolding piers
- Improper or inadequate bearing detail

Just like the PO's Supercuplok system, the Forensic investigation noted that the application of scaffolding tubes with internal diameters of 44mm (12mm greater than the connecting bar diameter) led to the formation of a void in the joint and a subsequent eccentricity in the upper and lower elements. This behavior obviously causes P-delta effects and consequently increases stress and reduces the strength of the elements (Pisheh, 2009). In fact, the contributions of the P-delta effect have been evaluated to increase the internal forces by 40% or more (Rutenberg, 1982). This is obviously very similar to the behavior of the spigot joint in the supercuplok system and therefore it is a must to understand the limitations of this connection in designs. Further, the enhancement, advancement and/or retrofit of this connection may significantly augment the consistency and security of these connections in designs.

**Willow Island, West Virginia, United States**

The collapse of a cooling tower on April 27, 1978, at a power station being constructed at Willow Island, caused the death of 51 construction workers. The scaffolding structure which supported the tower construction was quite unconventional at that time; now known as a “jumpform” support structure. A jumpform support formation occurs when the scaffolding is bolted to the structure being used to support. In this case, the concrete is poured and the scaffolding is raised and bolted into the new section once the concrete has cured.

The cooling tower had reached a height of 61m of its planned 131m. Construction was being completed on lift 29; the scaffolding was being supported by lift 28, which was placed the previous day, when the collapse occurred (Lew, 1982).

A team from both the National Bureau of Standards and the Occupational Safety and Health Administration (OSHA) board nominated the following as triggering events:

- Scaffolding was attached to concrete that did not have time to sufficiently cure
- An elaborate concrete hoist system was modified without an engineer’s review
- Contractors were rushing to speed construction

This failure demonstrated the critical nature and importance of measuring in-place concrete strengths before initiating a critical construction operation (Wright, 2003).

**Riley Road Interchange Freeway Ramp, East Chicago, Indiana**

On April 15th, 1982, three spans of the Riley Road elevated highway Interchange freeway Ramp collapsed, killing thirteen construction workers and injuring 18 other workers. The first of the three 180ft spans collapsed entirely, destroying the stairway and leaving workers stranded on the two remaining spans. A cherry picker had been brought in to save the remaining workers left stranded, however five minutes after the initial collapse, approximately 160ft and 135 ft of the 180ft second and third adjoining spans collapsed, respectively.
The scaffolding structure bore all of the dead and live load at the time of failure, since post-tensioning of the fresh cast-in-place concrete structure, had not occurred.

The National Bureau of Standards (NBS) determined that the cracking of a concrete pad supporting the scaffolding tower was the triggering mechanism of the collapse. However there were other “significant” contributors to the disaster including (Wright, 2003):

- Omission of wedges between stringers and crossbeams (i.e. Bearers and joists)
- Lack of stabilization of scaffolding towers against longitudinal movement
- Inadequate strength of the concrete pads
- Poor weld quality in the U-heads supporting cross beams at the top of the scaffolding towers

The investigation further highlighted the importance for careful consideration of the design of all components of the temporary support system used in concrete construction (Wright, 2003).

**Skyline Plaza Apartment Tower and Parking Garage, Virginia**

On March 2, 1973, a significant portion of the Skyline Plaza Apartment tower collapsed whilst concrete pouring was occurring on the 24th floor and shoring removal was occurring on the 22nd floor. The load and impact forces that resulted caused the progressive collapse of the entire parking garage under construction adjacent to the tower (Leyendecker, 1977).

The collapse resulted in the death of fourteen construction workers both in the tower and in the parking garage, with another 34 workers injured.
The investigation by the NBS and other forensic engineers concluded that the following causes were to blame (Wright, 2003):

- The premature removal of shoring on the 22nd floor caused a punching shear failure of the slab around one or more columns on the 23rd floor.
- The weight of the debris then resulted in the failures of the lower floors for the full height of the building.

Main Causes of Scaffolding Failures

There are quite significant findings from the forensic investigations of each of these scaffolding failures and it is clearly evident that amongst other causes the main reasons for scaffolding collapses are:

1. Overloading of scaffolding systems
2. Poor professional design and judgement
3. Insufficient strength and bracing of system

Furthermore, in a report by (Hadipriono, 1987) it was determined that 74 percent of scaffolding collapses between 1961 and 1982, occurred during the pouring of concrete as a result of the impact forces. He determined that the main cause of scaffold collapse is overloading. The other significant cause of failure was due to the premature formwork removal, inadequate bracing and the absence of inspection, inadequate design and vibration from equipment. Although this data is arguably out-dated, more recent studies have confirmed these results. In a study of high clearance scaffolds by (J. L. Peng et al., 1996a), the possible causes of support scaffold collapses were identified as overloading of the scaffold systems, instability of shoring components, partial loading of fresh concrete in the formwork, specific concrete placement pattern on the formwork, and load concentration from concrete placement.

(Milojkovic, Beale, & Godley, 2002) documented the results of an inspection by the Health and Safety Executive (HSE) in the UK regarding the typical faults in access scaffold systems. The most common cause of the collapse was insufficient tying to a permanent structure. Some other structural faults included in the
The report were the settlement of supports, out-of-plumb and out-of-straightness of standards, overloading and inadequate bracing.

SCAFFOLD FAILURE MODES

Steel scaffolding is a temporary structure and as such serviceability limit states are of no concern, rather ultimate limit state design is of critical importance. Due to the slender form of scaffolding, ultimate failure generally occurs as a result of buckling.

The two most common types of buckling are out-of-plane and in-plane buckling (as seen in Figure 1-12), with standards buckling in a single or double curvature, depending on Boundary support conditions and system configuration. The critical mode of failure is found from the relative stiffness of the connecting members in each direction. In a series of vertical load tests undertaken by (Yu, 2004) with multi-story door-type steel scaffold, it was determined that both single and double storey scaffolds buckled out-of-plane, and deflected in single and double curves respectively. The in-plane direction was found to be substantially stiffer. This conclusion is clearly an obvious one, the door-type design results in a substantially stiffer frame with a much greater second moment of area in the in-plane direction. (Yu, 2004) confirms this rather simplistic observation after measuring large displacements of the standards in the plane of the cross bracings at failure.

![Figure 1-12: Out-of-plane and In-plane failure modes (Chandrangsu and Rasmussen 2006)](image)

Portal frame type scaffolding was also tested; (Huang, 2000) performed experimental tests on one-to-three storey knee braced portal frame scaffolds. Again, the one-storey scaffold failed out-of-plane; however, quite interestingly the two and three-storey scaffolds showed in-plane buckling failure.

![Figure 1-13: Schematic failure modes of one-to-three storey knee braced portal frame scaffold](image)
(J. Peng, Pan, A., Rosowsky, D. C., Yen, T., & Chan, S., 1996) found that the deformation modes of high clearance steel scaffolds were dependent on the relative strength between the steel scaffold frames and the cross-braces providing lateral stability. Through a three dimensional analysis it was determined and straightforward to comprehend, that if the cross-braces offered more lateral support, the scaffold units would deform in-plane and vice-versa.

As it will be discovered in this thesis, the modes of failure will be of critical importance in determining the resistance capacity of SSS.
LOADING

In 1995 the Australian Standards Board produced (AS3610, 1995) which incorporated construction loads on formwork, this document's sole purpose is to act as a guide and a recommended minimum for construction design loads. The document includes load combinations and load factors to satisfy a minimum level of safety for typical temporary works in construction. Due to the fact that scaffolding systems are only in place in the short term, construction load factors are needed to ensure the safety of construction workers, the public and the building itself. Construction load factors are required and have come about as a result of some significant construction orientated issues. Firstly, the issue centres on the lack of guidance provided in the construction industry in specifying design values for construction loads. Secondly, there have been many significant and catastrophic structural failures of scaffolding in Australia and overseas, due to load underestimation. Both access and support scaffolding collapses have occurred at construction stage. Thirdly, there is a large amount of variability associated with construction loads; different construction activities produce highly variable loads and load combinations. Fourthly and possibly most importantly, to date construction safety has been considered a regulatory issue (D. Rosowsky, & Stewart, M., 2001). This thesis argues that the amount of time given to the design of a structure and its in-service loads also needs to be met equally by the time given to designing temporary support scaffolding that debatably could be as catastrophic in failure.

The following chapter seeks to synthesize all current and relevant information affecting and addressing load requirements on scaffolding systems, in order to produce a probabilistic / statistical model for loads imposed on scaffolding systems. This process will be addressed by first reviewing the standards for occupancy loads on permanent structures; identifying and classifying the components and types of loads on a scaffolding/shoring structure. Secondly, using these occupancy statistics as a guide as well as current research, the statistics of both constructions dead and live load will be reviewed in order to select the most relevant probability distribution for each loading variable. Finally, there will be an attempt to determine the distribution parameters (COV and mean) which will be used as inputs in the probabilistic modelling of steel scaffolding performed in this thesis.

Inadequacies of Current Practice

In order to accurately design a scaffolding system, or any structural system for that matter, an engineer must fully understand the loads that are expected to act on the structure during its construction and service life. Historically however, the latter issue of service life design has been of key focus; that is structural engineers, rightly or wrongly, have focused more on the safety of permanent structures than about the safety of temporary structures used for construction. From research and intuition, it is clear that there is a distinct lack of data available for loading of scaffolding in construction and this gives evidence of the imbalance and the potential for shortcomings or failure (Ferguson, 2003b). The lack of data means that there is a distinct lack of understanding about the load-resistance interaction of scaffolding/shoring systems. This seems especially ‘short-sighted’ (D. Rosowsky, & Stewart, M., 2001) in view of the fact that the ‘majority of structural failures occurs during construction’ (Ross, 1984), (H. Ayoub & Karshenas, 1994) and some recent catastrophic failures have occurred closer to home in Melbourne and Sydney.

Of particular concern is the knowledge that researchers (G. S. Fattal, 1983), (J.L. Peng, Rosowsky, Pan, Chan, & Yen, 1996), (S. Kamala & and Ayoub, 1994), (J. Ikaheimonen, 1997b) who have independently measured the loads in formwork shores, have all consistently documented that during concrete placement (when almost 74% of failures occur), the mean load in formwork shores varies from predictions. Further these researchers note that the current Australian Standards (AS3610, 1995) result in an underestimation of loads. It is the aim of this thesis to conduct a probabilistic assessment of the strength of steel scaffolding, and this process poses a challenge due to the lack of load data available as well as the aforementioned underestimation by AS 3610 in design loads.

Loads not being considered

In order to simplify the number of load cases in AS 3610, certain types of loads will not be considered for the analysis. This thesis will not be considering any form of horizontal loading found in Cl 4.4.5 (AS3610, 1995) and include; lateral concrete pressure (P), horizontal load due to sloping formwork, wind loads (W_{p} and W_{w}), loads due to water (X_{w}), earthquake loads and horizontal live loads (Q_{uh}). Further, the loads that act on a
structure are typically separated by natural (snow, wind, and earthquake) vs. man-imposed (dead, live) phenomena. To simplify the analysis and considering the temporary nature of scaffolding, the loads resulting from natural phenomena are not being considered.

**OCCUPANCY LOADS ON PERMANENT STRUCTURES**

Structures must have the ability to resist imposed loads. Most national standards developed to date use combinations of occupancy loads, and a limit state design methodology to ensure that the strength of a system is adequate to resist these loads. There is a significant amount of statistical data available with respect to occupancy loading, particularly considering the variability of mean versus code required long and short-term occupancy loads exerted on a structure. In direct contrast to the lack of data available for construction loads, the vast amount of statistical data available for permanent structures means that probabilistic modelling of occupancy loads has been studied by many researchers including (Chalk, 1980), (M. C. Harris, 1981),(B. Ellingwood, 1980b). Statistical models have been developed by researchers (Melchers, 1999) for each type of occupancy load using load scenario analysis, engineering judgment and load surveys. Other researchers (D. Rosowsky, & Stewart, M., 2001) have developed complex models of load intensity, duration and frequency of occurrence.

The Australian Standard and most other national standards (The Israeli Standard, European standard, American Standard and British Standard) deal with occupancy loads by modelling them as a combination of dead and live loads. The dead load includes the weight of the structure and its permanent fixtures, whilst the live loads are split into sustained occupancy loads (weight of furniture, people and portable equipment) and extraordinary live loads (weight due to crowding of people or temporary storage during renovation). Section 3 –Imposed actions of AS 1170.4 suggests that sustained occupancy loads be considered uniformly distributed actions, ranging in magnitude from \(0.5 − 7.5 \text{kPa}\) depending on the type of building being considered. However extraordinary live loads have no direct input values and are incorporated into the safety factor.

Since this thesis is considering steel scaffolding systems which are temporary structures, it is evident that occupancy loads have no direct effect; however by analysing the large amount of statistics, errors and models produced for occupancy loading, large amount of information can be synthesized and transferred to investigate construction loading. Further due to the large correlation between occupancy and construction loads, a review of specific probabilistic models by occupancy load researchers will enhance the ability of the authors to analyse and postulate of construction loads.

**Occupancy Dead loads**

There is little debate over the statistical modelling of occupancy dead loads due to the accuracy and vast quantity of data produced by academics and standards boards. Occupancy dead loads follow a normal distribution with a mean-to-nominal ratio of 1.05 and COV of 0.10(B. Ellingwood, 1980b). And since dead loads generally remain constant, they are modelled as a single pulse over the lifetime of the structure.

**Occupancy Live Loads**

Sustained Live loads are similar to dead loads in that they remain relatively constant within a specific occupancy. Live load survey statistics analyzed by Chalk (1980) depend on the area of loading, however for the purposes of this thesis, sustained live loads can be assumed to have a mean bias value of \(L_{\bar{L}} = 0.3\) and COV of 0.6(Hendrickson, Ellingwood, & Murphy, 1987). Sustained live loads have been shown to also follow a gamma distribution (Corotis, 1977) (D. Rosowsky, & Stewart, M., 2001).

The parameters and distributions for extraordinary live load on the other hand are more complex to determine. The transient nature of this load type means that load surveys over small periods of time do not provide reasonable estimates. Of the academics, McGuire, et al. (1974) has produced an accurate load scenario analysis of crowding people to gain a statistical understanding of the magnitude of extraordinary live loads. Further noting that a gamma distribution best represents the crowding load scenario(McGuire, 1974). To address the difficulties in taking load surveys, McGuire (1974) modelled extraordinary live loads as a ‘Poisson pulsed process’ whereby the load only acted for a short duration, less than 2 weeks. As a result,
these methods produced a more accurate mean bias value of $L_{\text{mean}} = 0.19$ and COV of 0.66 for a typical office building (Philpot & Rosowsky, 1992). Further, this accuracy is confirmed by the academic community, who have used these figures and adopted them into other probabilistic studies (Rosowsky, et al., 2001).

**CONSTRUCTION LOAD**

Until authorities (most likely prompted by the high incidence of structural failures of support scaffolding in construction) identified a need for reliable data on construction loads, there had in fact been little research undertaken to measure the mean loads in support scaffolding (Ferguson, 2003a). After a review of recent research conducted by (G. Fattal, 1983), (H. Ayoub, & Karshenas, S., 1994), Peng, et al. (1996), Rosowsky, et al. (1997), Kamala, et al. (1994), (J. Ikaheimonen, 1997a) and (Kothekar, 1998); it was noted that the complications of modelling construction loads, as well as an evaluation of the current Australian and international scaffolding standards should be undertaken.

The current steel scaffolding code (AS3610, 1995) utilises available statistical information and LRFD design to establish of a consensus target reliability (M. E. Harris, Bova, C.J., and Corotis, R.B., 1981). This involves using engineering judgement and expert opinion, known as the 'Delphi method', to select appropriate values for live load combinations and partial safety factors. As the aim of the probabilistic analysis is to determine accurate safety factors for a generic design equation, construction loads are divided into generic dead and live load components. For the purposes of this report, Dead Loads will be defined as the total vertical load exerted by other formwork components not included in the scaffolding system, such as timber joists, bearers and plywood. The Dead load also includes the weight of concrete, formwork and reinforcement. Live Load on the other hand includes the extra materials, workers and machinery that are supported by the scaffolding before and after concrete placement. Live Load also includes the dynamic loading effect of pouring/dropping concrete from a pump/kibble.

**CONSTRUCTION LOADS ON TEMPORARY STRUCTURES**

In the study of permanent structures, complex models of load intensity, duration and frequency of occurrence have been studied (Rosowsky, et al., 2001). Hence occupancy load statistical results for live and dead load are known with a fair amount of certainty. However, due to the complexity and lack of statistical information regarding construction loads, the duration of load and frequency of occurrence of construction loads has not been considered in this analysis.

Further, there are significant difficulties in modelling construction loads including; problematic use of 'professional judgement' in creating scaffolding standards, the unpredictability of construction loads with different magnitudes of load and load arrangements, the complexity in dealing with concrete pouring paths and finally the anomalies occurring as a result of hardening concrete. Further discussion of the difficulties in modelling construction loads can be found in BELOW

**DIFFICULTY IN MODELLING CONSTRUCTION LOADS**

Since there is very little information available, and little consensus exists on the magnitude and variability of different construction loads; code developers have had to rely on experience from developing service life codes as well as their own professional judgment in selecting load factors and combinations. It is clear then that this process is entirely subjective and directly resembles the development of earlier codes and standards (for example the development of the in-service code). The Delphi studies have revealed a consensus amongst professional engineers in regard to the appropriate target reliability in construction design (e.g. Level of safety during construction must be equal to safety during service). However it is difficult to ensure this occurs in a probabilistic context, since the statistical information that is required to perform any calibration studies is scarce. Rosowsky (D. Rosowsky, 1996) makes it clear that the development and acceptance of the construction live load design process are both iterative processes. As data and
probabilistic models become available, calibration studies can be performed and resulting factors can be adjusted to achieve the desired level of reliability.

A structure will experience vastly different loads in construction than it will whilst in service. Service design involves the analysis of possibly simultaneous structural and environmental load events, including dead, live and environmental loads. Construction loads on the other hand are not so linear and predictable; different magnitudes and load arrangements are associated with different construction procedures and operations. Work by Huston et al. (Huston, et al., 1996) and other academics (Ayoub, et al., 1994) identify this problem, illustrating the way in which construction processes dramatically alter the magnitude and type of load that is imposed on the structure. Further the temporary nature of many temporary construction systems causes concentrated point loads on scaffolding members. Like for example, heavy equipment and stockpiles of material on supports with only a small contact area.

Construction loads are vastly different from loads a structure may be subject to during its service life (D. Rosowsky, 1996) and can come from a wide variety of construction related sources. Therefore, the ability to develop a single construction load model is problematic, and specific models must be developed for analysis purposes (Rosowsky, et al., 2001).

There are some other significant documented anomalies in modelling loads. Firstly, support scaffoldings is subject to various different load patterns as a result of concrete pouring paths and type of pouring; pump vs. crane kibble. Since design usually assumes uniform loads over an area, many studies have been compiled to determine whether load path variations are worse or better than the assumed uniform loads. In particular research conducted by (Peng, et al., 2003) nominates three possible sequential load patterns occurring on 3-storey scaffolding systems (L, R and U shaped). For each of these load patterns, the sequential paths were investigated and compared to uniform loads. The results were in correlation with earlier research (Peng, et al., 1996), in that there was in fact negligible difference between the sequential paths and uniform loads. Hence the following paper has not considered sequential load patterns as they are similar to uniform loads.

Further between construction stages two and three, a loading anomaly comes as a result of the hardening concrete that begins to support its own weight, thus reducing the dead load supported by the scaffolding system. Hence the lowest LL/DL ratio will occur just after all the wet concrete is placed, before it has begun to set and take up its own weight (Karshenas, et al., 1994). Another load modelling anomaly is that additional levels are built on a newly poured slab before the slab is able to completely support it, meaning that some scaffolds may actually support more than one slab, workmen and materials. This will lead to significant dead load increases in the first support scaffolding, however to simplify this paper, these animals will not be taken into consideration.

There are some anomalies that are required to be taken into consideration due to their significance, namely load eccentricity. Eccentricity is of particular concern in the design of scaffolding as it introduces bending moments in a shoring member which subsequently reduces axial capacity. Load eccentricity significantly reduces the load carrying capacity of scaffolding systems, recent research by Peng and Chen (Peng, et al., 2009) analyzed the effects of load eccentricities using a number of cross bracing configurations. The results concluded that regardless of the configuration, load eccentricity significantly reduced the ultimate strength of the system as a result of increased imposed vertical loads. However the eccentricity section in (AS3610, 1995) is simplistic and merely acts as a check that the eccentricity of the load is greater than the minimum \( e = e' + L/200 \), it does not actually contribute to the load on the structure. However for the purposes of this investigation the effect of eccentricity will be accounted for in the resistance component of the study and therefore can be neglected in the loading component of the limit state equation.
Stage of Construction effects

The stages of construction mentioned in AS3610 Australian Standards, were used in the assessment of construction loads. It is intuitive to understand that dead load increases as the concrete placement progresses, and reaches its maximum at the end of the placement. The live load consists of the weight of construction personnel, equipment, stacked material, and the effects of any impact during concrete placement. The magnitudes of the live loads depend on the stages of construction. There are three stages of construction, before, during, and after placement of concrete. Figure 14 shows a typical shore load histogram (a combination of the dead and live load effects) adapted from (Fattal 1983).

In stage I, before concrete placement, the live to dead load ratio is high since no concrete has been placed, yet there is a significant live load due to workmen and materials. However in stage II (placement period) the dead load significantly increases as concrete is placed, and since there are no additional live loads present during this stage, the live-to-dead load ratio falls significantly below 1.0. It must be noted that stage two is the critical stage which this paper is investigating, since this stage is where the majority of support scaffolding failures occur and it is the period at which the maximum shore loads occur. During the end of the placement period, live load reduces as a result of decreased construction activity.

In stage III, after concrete placement, the smaller live-to-dead load ratio will begin to increase due to increased construction activity as more workers and machinery begin to vibrate and later smooth the surface, increasing even further as materials begin to be stacked on the newly laid surface ready for the next level’s pour. It is important to understand that the difference between the maximum shore load and the load just after the casting provides an estimation of the live load effect.

Figure 14 Stages of construction

CONSTRUCTION LOADS

Shore Dead Loads

The following section seeks to describe previous research involving calculation of shore loads. A chronological summary of all previous research involving formwork and loading can be seen in APPENDIX 2. It must be noted that ‘relative shore load’ in this section refers to the ratio between the actual recorded load vs the calculated or measured load. The relative shore load can also be described as the ratio between mean and nominal loads.

Some of the first measurement’s of shore loads were conducted by Agarwal and Gardner (1974). They investigated the distribution of load between the floor slab and vertical shores, after the concrete had been poured. Agarwal and Gardner, recorded variation in the ratio of calculated to measured shore loads i.e. relative shore loads. Their results showed a small variation in the relative shore load. On their first two sites, 8 and 9 shores were measured yielding a mean relative shore load ($\bar{D}/D_n$) of 1.03 and 0.99 and a standard
deviation of 0.1 and 0.03, respectively. Although it is unclear as to whether these sites used a pump or a kibble to pour the 203mm (8 inch) slabs, the variations are clearly small.

It is argued that the results are not truly accurate since they assumed the load of the formwork was 5% of the total concrete load and the weight of the formwork was 10% of the concrete weight. Others suggested that the relative shore loads must be corrected by a factor of 1.05 (J. Ikaheimonen, 1997) have. The small standard deviation in relative shore loads is arguably due to the use of prefabricated ‘flying formwork’, which has a simple two support structural configuration (J. Ikaheimonen, 1997).

Five years later, Mohammed and Simon (1979) measure the maximum relative shore loads of a ‘flying form’ system in a fifteen storey office building. Mohammed and Simon (1979), measured the relative shore loads on the shores below the pour and a further five levels of re-shoring occurring below the concrete pour. Measurements were taken at two minute intervals and the maximum relative shore loads for concrete load only were 1.23, 1.51, 1.68, 1.09, and 1.46. Furthermore, with the inclusion of the weight of formwork (10% of the weight of concrete), it was found that the relative shore loads reduced, as expected, to 1.21, 1.46, 1.61, 1.08 and 1.42. That is, \((\bar{D}/D_m)\) of 1.394 and COV 0.168 for the arguably small data set of five shores.

In 1983, Fattal measured the loads applied to eleven shores in a multistory building. Information on applied loads was collected, and documented with extensive photographic records. In his research, a 203mm (8 inch) thick slab was poured with a 1.53 sqm (2 cubic yard) skip and as such there were significant dynamic effects measured. The maximum relative shore loads were 0.80, 1.10, 1.08, 0.27, 1.04, 1.36, 1.76, 0.95, 0.61, 2.00, and 0.77 with a mean \((\bar{D}/D_m)\) of 1.07, COV of 0.48 and a standard deviation of 0.50. These results included the weight of formwork of 0.465 kN/sqm in the maximum measured shore loads. The high standard deviation is thought to be as a result of the very stiff aluminum formwork components, steel shores and 16mm thick plywood. As well as the fact that there were 10 workers involved in concreting and the use of a large concrete skip, which had significant dynamic effects. Fattal (1983), observed that the load sharing was occurring between upright No. 4 and upright No. 10 (relative dead load of 0.27 and 2.0 respectively). By excluding these results the survey data had a mean \((\bar{D}/D_m)\) of 0.92 and a COV of 0.35 for the tributary area method, and a mean of 0.93 and a COV of 0.32 for the beam or “distributed” method.

In 1994, Ambrose et al. developed a solution to reduce the number of scaffolding failures. Loads in shores would be measured and monitored with load cells in real time. This ‘smart shoring system’ was used to detect non-uniform load distribution between shores, where some shores were overloaded and others not carrying enough load. Ambrose was able to confirm the validity of the ‘smart shoring system’ in a number of tests conducted on site before, during and after concrete placement and found there was in fact a reduction in shore load after the maximum load had been reached and concreting was finished. Ambrose et al. (1994) hypothesized that this was in fact due to the use of rapid setting cement being used and actually relieving the shores as the concrete begun to set.

Ambrose et al. (1994) was able to conclude that the ‘smart shoring system’ can give prior warning, and timber shores supported by poor soil or that are inherently weak can be identified. As such this system has the potential to avoid the failure of shoring systems, and further provide the information necessary for notification of potentially hazardous shoring failure mechanisms. Ambrose et al. (1994) suggests that excessive load impacts may be realized in real time, however the argument here is that the failure of shores has in the past been without much prior warning. As such any warning system would need to be calibrated at significantly lower loads to allow sufficient time for workers to escape before the collapse. If this pre-warning system could be implemented, then an excellent database of information on shore loads could be established. Ambrose et al. (1994) suggests that this data could provide a construction ‘Black Box’, which would provide critical information to investigators in the event of failure. In an investigation of the Courthouse building in Burlington, Vermont, Ambrose et al. (1994) collected actual construction load data on a beam form located at the lower levels of the car park. It was evident that differential shore loads can be measured and these instrumented shores are useful in the detection of potentially dangerous uplift situations. The maximum relative differences in shore loads that Ambrose et al. (1994) determined from laboratory experiments are extremely relevant to this research. In fact, the measured loads for identical tests and specimens are quite different, with maximum relative differences of 1.5, 1.3 and 2.0.

(J. Ikaheimonen, 1997) drew one important conclusion from the previous shore load investigation results, that it is not possible to predict the magnitudes of shore loads by calculation, however accurately they are made. It is evident that the uneven distribution of loads between shores is due to the uncertainties and small differences in load eccentricity, contact areas, material properties, dimensions etc.

In 1993 a joint U.S. and Taiwan investigation (Rosowsky et al. 1994) into high clearance scaffolding systems was initiated to quantify the actual loads during placement as well as to detect any placement pattern effects.
Altogether, 19 reinforced concrete construction projects were monitored across multiple building types including museums, hangars, schools, residential buildings and gymnasiums. All sites used a pump to convey concrete and a building layout was kept which documented the placement areas, placement pattern and the location of equipment. Rosowsky et al. (1994) also performed full scale laboratory tests of formwork construction to simulate the pouring of concrete. The aim was to compare the measured and calculated shore loads on 14 different shores, seen in Table 1. To simulate the load effect of concrete, containers were placed on the formwork and filled with water to simulate the weight of a 130mm thick concrete slab.

<table>
<thead>
<tr>
<th>Load Cell</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative Shore load</td>
<td>0.2</td>
<td>1.44</td>
<td>0.42</td>
<td>0.51</td>
<td>2.17</td>
<td>0.97</td>
<td>2.28</td>
<td>1.37</td>
<td>1.88</td>
<td>2.26</td>
<td>0.95</td>
<td>1.46</td>
<td>2.35</td>
<td>1.22</td>
</tr>
</tbody>
</table>

Table 1: Ratio of measured to calculated shore loads (Rosowsky et al (1994))

The results of the full scale model gave a mean value of the relative shore loads ($\bar{D}/D_n$) of 1.39, COV of 0.48 and a standard deviation 0.66. It is clearly evident that the measured loads are quite different from the calculated loads, in spite of the fact that Rosowsky et al. (1994) tested in a controlled laboratory environment. It is suggested that the two main reasons for the variation in shore load is the rigidity of the containers and the absence of reinforcement. Rosowsky et al. (1994) also proposed that the theoretical load be increased by a load factor of 2.0.

In 1996, Kamala, Dickens, Pallet (1996) measured shore loads on nine shores with good agreement between the calculated loads. Two particular shores however had relative shore loads of 0.88 and 1.28. The most likely reason for this, suggested by the authors, was due to uneven jacking. Theoretically, uneven jacking could lead to vastly different relative shore loads since a particular shore may bear the load of its neighbor until the differential vertical eccentricity (due to the uneven jacking sequence) has been restored.

Four years after his initial work, Rosowsky et al. (1997) measured actual shore loads in three structures. Two identical 200mm thick floor slabs of 8.1 sqm area were measured. Whilst the third structure was a large 200mm thick floor slab of 51 sqm area. Rosowsky et al. (1997) found large variation in measured shore loads compared to calculated shore loads. And again a multiplication factor of 2.0 was suggested for variation in individual shores. (J. Ikaheimonen, 1997) suggests that this multiplication factor is "plainly wrong", since each shore had a different loaded area and the multiplication factor is calculated by dividing the maximum shore load by the mean of all shore loads. Further, the shore loads were obtained during concreting when the maximum load on shores had not yet been reached. On the other hand, Rosowsky et al. (1997) draws the conclusion that the variation in shore loads is due to initial prestress in shores and that an 'area effect' exists so that the maximum shore load for a given area decreased as the area increased. i.e. the design load of ACI is too conservative when the area is increased.

The results of Rosowsky et al. (1997) work shows that there is a large variation among relative shore load values and the conclusion can be drawn that it is quite difficult to predict the magnitudes of shore loads by calculation. Furthermore (J. Ikaheimonen, 1997), determined that there are significant and very large variations in expected (calculated) loads on shores and the mean (measured) loads occurring on site. This indicates that it may be extremely problematic to have a linear relationship between load and the tributary area or theoretically loaded area methods. It is postulated that the large scatter of relative shore loads may be due to different concreting methodologies or due to workmen on the slab and the use of motorized equipment.

Simple conclusions can be drawn regarding actual shore loads from the results of (J. Ikaheimonen, 1997a) research. In each of the nine sites investigated a curve was plotted for the measured shore loads. It was clearly evident that when concrete was pumped there was minor short term increases in load indicated by small peaks or spikes in the curves. Four bridges were investigated with average relative dead loads of 1.053, 0.876, 0.768, 0.853 and COV of 0.094, 0.256, 0.334, 0.151, measured from a maximum of 8 load cells. Five apartment constructions were investigated with average relative dead loads of 0.725, 1.207, 0.730, 0.904, 0.958 and COV of 0.412, 0.178, 0.764, 0.134, 0.144 measured from a maximum of 12 load cells. The full set of relative shore loads have been summarised in APPENDIX 2. The measured to calculated dead load ($\bar{D}/D_n$) was found to have a mean of 0.9 and COV 0.29 for the tributary area method, and a mean of 0.99 and a COV of 0.3 for the beam method.
There was a noticeable difference in (J. Ikaheimonen, 1997a) results, when concrete skips or buckets were used to drop the concrete from a height onto the formwork surface. In this case spikes or peaks in the results indicated the unloading of concrete from a skip. (J. Ikaheimonen, 1997) noted that these short term peak loads were characterized by two distinct phases. In the first phase, the peak load was of short duration, equating to the 5-10 seconds that it took to empty the skip. The second phase was of a 1-2 minute duration, and it was postulated that this was a result of the excess heaped concrete whilst it was being shovelled away from the drop site (J. Ikaheimonen, 1997). The magnitude of these short term peak loads was up to 30% of the static load in the shore and depended on the rate at which the skip was emptied, the viscosity of the concrete and the height above slab when dropped. It was quite interesting to note that when the concrete was pumped no such peak loads were observed, and it was again postulated that this was owing to the lower rate of flow of concrete and its more viscous consistency; evident in the distinctly different graphs in Figure 1-17.

Puente et al (2007), instrumented 34 steel shores during construction of a seven storey concrete building. Slab thickness was 250mm. The results suggested that mean dead load ratio was 1.0. i.e. the mean was approximately equal to the nominal and a COV of 0.25. The following table provides a summary of dead load statistics previously accumulated relating mean and nominal dead loads ($\bar{D}/D_n$).

<table>
<thead>
<tr>
<th>RESEARCHER</th>
<th>AVG</th>
<th>STD</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Agarwal &amp; Gardner (1974)</td>
<td>1.030</td>
<td>0.100</td>
<td>0.097</td>
</tr>
<tr>
<td>Mohammed &amp; Simon (1979)</td>
<td>1.394</td>
<td>0.249</td>
<td>0.178</td>
</tr>
<tr>
<td>Fattal (1983)</td>
<td>1.067</td>
<td>0.514</td>
<td>0.482</td>
</tr>
<tr>
<td>Fattal (1983) exc outliers</td>
<td>0.920</td>
<td>0.322</td>
<td>0.350</td>
</tr>
<tr>
<td>Rosowsky (1994)</td>
<td>1.391</td>
<td>0.666</td>
<td>0.479</td>
</tr>
<tr>
<td>Rosowsky (1997) small pour</td>
<td>NA</td>
<td>NA</td>
<td>0.333</td>
</tr>
<tr>
<td>Ikaheimonen (1997)</td>
<td>0.900</td>
<td>0.261</td>
<td>0.290</td>
</tr>
<tr>
<td>Puente et al. (2007)</td>
<td>1.000</td>
<td>0.250</td>
<td>0.250</td>
</tr>
</tbody>
</table>

Table 2: Summary of Dead Load Statistics

As can be seen from Table 1, the mean-to-nominal relative dead load ($\bar{D}/D_n$) calculated by researchers to date, varies between 0.9 and 1.4. Hence based on the tributary area method researchers (Zhang, 2012) have proposed that $\bar{D} \approx D_n$. Furthermore COV ranges between 0.25 and 0.5, thus under current normal construction practice and using the tributary area method, a $V_D$ of 0.3 is representative for the dead load effect of scaffolds.
Occupancy dead load statistics are known, through years of research, with certainty. The occupancy dead load COV is 0.1 (Ellingwood, 1982). The additional uncertainty or variability of dead load on scaffolds is not fully understood but will be considered in this paper. It is postulated to be due to differential settlement of uprights, imperfections in the scaffolding installation such as lack of bearing occurring between the steel shores and timber formwork. (D. H. Rosowsky, 1994) (J. Ikaheimonen, 1997).

Shore Live Loads

Imposed actions or live loads (Q) include the weight of workmen, stacked materials and dynamic forces due to concrete pouring. The value for construction live load during stage II of construction is 1.0 kN/m² in AS3610. The American Standard, ACI 347 (ACI 2004) specifies a design formwork live load of 2.4 kPa (50psf). Dynamic live load effects, as a result of placing concrete, vary from 5-30% (Ferguson, 2003) of the load in the formwork shore. It must be noted that the live load from stacked materials, is only applicable to the shores after concrete placement (in Stage III of construction), and is therefore not considered in this paper.

Construction live load data is very scarce and little is known about its statistical variability. The first arguably reliable data was collected by Fattal (1983). The research measured shore loads in scaffolds placed under 200mm thick floor slabs. Shore loads were measured continuously during and after concrete placement; importantly, the results showed that the loads were not uniform. Interestingly, Fattal presumed the disparity was a result of either the lack of bearing when the shores were first installed or residual forces in the shores.

Fattal showed that during concrete placement, the dynamic effect of pouring concrete was equivalent to a uniformly distributed load (UDL) of 1.8 kN/m², a worst case when the load was directly above the shore. Also it was determined that the maximum dynamic effect of discharging concrete from a 1.5 m² skip resulted in a UDL of 1.6 kN/m². In an analysis of the load effects, the maximum load in shores was compared with the design loads specified in the ACI 347. It was determined that the 2.4 kN/m² (50 psf) live load given in ACI 347 was adequate to compensate for the dynamic effects of concrete pouring (G. S. Fattal, 1983). Load in some shores exceeded ACI 347 design loads by up to 18%, yet this was considered to be tolerable since a 2.5 safety factor was used in shore design. Fattal (1983) was able to demonstrate that construction live loads, for the crane and bucket method) had a mean-to-nominal ratio (L̅/L_n) = 0.31 with a COV= 0.71 for the tributary area method, and (L̅/L_n) = 0.27 with a COV= 0.54 for the more accurate distributed method. A summery of live load statistics can be seen in Table 3

Karshenas, et al. (1994) suggested that a EUDL Q = 2.4 kN/m², based on 0.99 fractals, would be adequate for live loads over influence areas smaller than 15 m² and Q = 1.7 kN/m² for areas greater than 15 m². Furthermore, when the concrete is placed in buckets/ skips and motorized buggies are used, the combined static and dynamic loads may be substantially higher than the loads resulting from stacked materials and equipment.

Investigations by Peng et al, (J. L. Peng, 1994) concerning the load effects of influence surfaces associated with different concrete placement patterns, concluded that axial loads in scaffolding shores were not changed due to different load placement patterns. Hence concluding that load pattern is not significant.

Of critical importance were the results contained in papers by (Rosowsky D.V., 1994a) (D. V. Rosowsky, Huang, Chen, & Yen, 1994) which aimed at investigating the load effect of concrete placement patterns. As with the results of (J. L. Peng, Yen, Lin, Wu, & Chen, 1997), it was determined that the load distribution of a vertical shore was non-uniform, and further the maximum load in standards was found to be twice the calculated load.

(D. V. Rosowsky, Philbrick, & and Huston, 1997) suggested that the ACI design live load of 2.4 kPa, adequately estimated the maximum load in a vertical shore. However, (D.V. Rosowsky et al., 1997) proposed that for ultimate limit state design, the current load factor was inadequate and instead a factor of 2.0 was warranted, in order to account for the spatial variability of shores/standards. Rosowsky also noted that there was a relatively high uncertainty in the resistance capacity of the system, despite the low degree of uncertainty in the mean concrete dead load (COV of concrete = 0.1). An important discovery was also that the ACI 347 design loads become more conservative as the area of the slab increased. This led to the suggestion that the design load might be a function of pouring area, slab thickness, formwork arrangement and concrete placement procedures (Ferguson, 2003b).

A survey of concrete construction live loads (Karshenas, et al. (1994)) on slab formwork both before and after concrete placement was able to determine that the mean live load on newly poured slabs (which included the weight of workers, equipment and materials stored on the formwork) for 1 week duration was...
0.3 kPa with a significantly high standard deviation of 0.32 kPa. This result was used by Rosowsky’s (D. Rosowsky, & Stewart, M., 2001), who used a Monte Carlo method to simulate the statistical distribution of maximum construction live loads over a 6 month period. Rosowsky et al was able to demonstrate that construction live loads had a mean-to-nominal ratio \( \frac{\bar{L}}{L_n} \approx 1.0 \) with a COV of 0.4. Further, a Type I extreme distribution was found to provide the best fit to the simulated distributions. Rosowsky (1996) made the implication, after the publication of a new ASCE standard, that the code had been developed based on conservative assumptions and consensus amongst engineers, since there was very little statistical data or probabilistic models on construction loads.

The research by Ellingwood and Galambos (1982) from typical buildings and conventional structures, led them to suggest that a type I extreme value distribution with a \( \frac{\bar{L}}{L_n} \approx 1.0 \) and a COV of 0.25, was appropriate. (J. Ikaheimonen, 1997) produced reliable live load data by modeling 66 scaffolding shores in all three construction phases (before, during and after concrete placement) on nine different sites. (J. Ikaheimonen, 1997) measured the loads in vertical shores which supported concrete slabs ranging from 150mm to 1310mm in thickness. The analysis used two methods to predict the loads, the “simplified method” and the “beam method”, which were then compared to the mean loads. The results showed that when concrete was delivered by a pump, the variable loads had no significant effect on shore loads, however when concrete was delivered in skips, short term dynamic load effects were observed to be 5 – 30% higher. Furthermore, it was determined that the load due to the weight of workmen and equipment is very small in comparison to the dead load. (J. Ikaheimonen, 1997a) determined the mean-to-nominal ratio \( \frac{\bar{L}}{L_n} = 0.99 \) and COV \( (V_L = 0.31) \). And furthermore, when the crane and bucket method is used for concreting, the mean live load ratio \( \frac{\bar{L}}{L_n} = 0.3 \) COV is 0.7.

Other researchers, (T. Chandrangsu, 2010) also concluded that the maximum construction live load on scaffolding systems is assumed to be a Type I extreme distribution, with a mean-to-nominal value \( \frac{\bar{L}}{L_n} = 1.0 \) and a COV of 0.6. This was based on the Australian Standard AS3610 (Standards Australia, 1995), which specifies a design formwork live load during concrete placement of 1 kPa.

It must be noted that \( \frac{\bar{L}}{L_n} \) depends on the design formwork live load specified in the particular standard. Other researchers (Zhang, 2012) have based their statistics on the American Concrete Institute who specify a design live load of 2.4 kPa (50 psf) for all construction stages. Using this standard, a Type I extreme distribution was selected by Zhang, 2012 and a mean-to-nominal live load \( \frac{\bar{L}}{L_n} = 0.3 \) with a COV of 0.7. A full summary of live load statistics can be seen in Table 3.

It must be understood that due to the variability of construction processes as well as the effects of dynamic and peak live loads, there is still significant uncertainty about live loads. As a result generic construction live load models are not appropriate and despite all the research efforts, a statistical load model for construction live loads was of critical importance to this research. The following paper seeks to address this apparent lack of live load data and quantify more accurate construction live load results during concrete placement.

<table>
<thead>
<tr>
<th>RESEARCHER</th>
<th>AVG ( \frac{\bar{L}}{L_n} )</th>
<th>STD</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fattal (1983) trib. area Method</td>
<td>0.744</td>
<td>0.354</td>
<td>0.401</td>
</tr>
<tr>
<td>Fattal (1983) dist. Method</td>
<td>0.646</td>
<td>0.350</td>
<td>0.540</td>
</tr>
<tr>
<td>Rosowsky (2001)</td>
<td>1.000</td>
<td>0.400</td>
<td>0.400</td>
</tr>
<tr>
<td>Ellingwood et al. (1982)</td>
<td>1.100</td>
<td>0.250</td>
<td>0.750</td>
</tr>
<tr>
<td>Ikaheimonen (1997)</td>
<td>0.990</td>
<td>0.300</td>
<td>0.370</td>
</tr>
<tr>
<td>Chandrangsu (2009)</td>
<td>1.000</td>
<td>0.600</td>
<td>0.600</td>
</tr>
<tr>
<td>Zhang (2012) [based 2.4 kPa]</td>
<td>0.300</td>
<td>0.270</td>
<td>0.700</td>
</tr>
</tbody>
</table>

Table 3: Summary of Live Load Statistics (based on 1kPa design Live Load U.N.O)

Critical Shore Load Findings

There are some key findings that relate directly to this research. Firstly, the methods which (J. Ikaheimonen, 1997a) used to calculate loads in standards, namely the “beam method” and the “simplified method” were found to be “largely comparable”. And the simplified method was an accurate method used to investigate
relative shore loads. Secondly, when designing formwork beams and calculating the support reactions for beams acted upon by point loads, a uniform distributed load (UDL) can replace these point loads.

Critical to the following research report was the acknowledgement by other authors (D. Rosowsky, & Stewart, M., 2001) and (J. Ikaheimonen, 1997), that more measurements of shore loads where needed to establish a better basis for statistical analysis. Further, (J. Ikaheimonen, 1997) noted that standards should be instrumented and loads should be measured over a larger area. It is also suggested that further research would be extremely valuable if the causes of unexpected standard loads were investigated on site. E.g. If a standard has an unexpectedly high load as soon as the soffit formwork and reinforcement are placed.


An investigation of shore loads must be conducted to understand how construction loads are transposed into the vertical support elements.

**Difficulty in recording actual(mean) shore loads**

It has been noted by previous researchers that maximum shore loads occur immediately after concrete placement and the variable live loads were removed (J. Ikaheimonen, 1997)(apart from shores affected by dynamic loads). Furthermore, it has been suggested that when calculating shore loads, only the weight of formwork and concrete need consideration (Ferguson, 2003b). It is also arguable that this result gave some credence to a previous investigation by (Ashraf et al 1994), who determined that the mean of variable loads was quite small and the standard deviation was quite large. (J. Ikaheimonen, 1997a) results showed that the relative shore loads varied between 0.15 and 1.5, implying that there must be a critical link between the magnitude of relative shore loads and the way in which the formwork, joists, bearers, U-heads and top jacks distribute the load to the shores.

Some previous researchers have concluded that an initial load within the vertical standards is responsible for these large variations in relative shore loads. The initial load is thought to be due to the force induced by the formwork and reinforcement bar which is laid above prior to concreting. However, researchers (N. Kamala, Dickens, J., Pallett, P., 1996) have failed to prove this assumption since, in many cases, no initial shore loads were recorded. (J. Ikaheimonen, 1997a) provided a theoretical analysis which demonstrated that when an initial gap is present between the standard and the bearer; there are large variations in the relative shore load.

The measurements taken by (J. Ikaheimonen, 1997a) demonstrated that large relative shore loads do not necessarily correlate with large initial loads in standards, or vice-versa. (J. Ikaheimonen, 1997a) postulated that this occurrence could be as a result of the formwork components, after being subjected to a portion of the load; straighten up from an initially deformed position. Other reasons may be that different elements of the same formwork element may have been damaged or have different material properties than other elements, e.g. bearers or joists may have different moisture contents due to different storage conditions (J. Ikaheimonen, 1997a). The storage conditions would also affect the elastic modulus and creep of the timber formwork systems. Tsouris (1991) and Madsen (1992) proved that deformations increase in timber with higher moisture contents and temperatures.

The variation of the standard deviation parameters of elastic modulus, load eccentricity and initial inclination of shores, by Liu and Chen (1987b) and El-Sheikh and Chen (1989), demonstrated the difficulty in calculating actual shore loads due to different shore spacing and different formwork materials. Initial shore loads are also affected by the position of bar chairs which are used to support reinforcement. The location of a bar chair can affect the reinforcement load distribution, however this effect is considered to be minor and thus is ignored.

It has also been postulated that the large scatter in relative shore loads is also related to the retightening of shores after formwork and reinforcement has been placed. This would result in particular shores having more initial load when concrete is poured (J. Ikaheimonen, 1997a). However, it was found in this investigation, that re-tightening of shores after formwork and reinforcement has been placed does not occur.
The bearers are laser levelled which typically ensures similar initial loads. Hence the statement by (J. Ikaheimonen, 1997a) is not consistent with the construction of the Acrow Supercuplok system.

Why concrete placement pattern is important

It is important to understand the concrete load placement patterns on buildings during construction. And there are some significant reasons why this is critical namely:

- In most reinforced concrete construction the support scaffolding systems including its shores are not completely tied to the structure. Thus in the case of uneven or asymmetric concrete placement, the supporting elements may loosen and for all intensive purpose 'drop out' of the system (D. Rosowsky, Philbrick, T., & and Huston, D., 1997).
- The full load sharing capabilities of a concrete slab are not in place at early age concrete strengths when the slab stiffness are not fully developed.
- Support scaffold systems are generally not as stiff as a cured slab, hence the load transferring capabilities (from one level to another) that exist in the complete structure are not available during construction (D. Rosowsky, Philbrick, T., & and Huston, D., 1997).
- As noted earlier, there has been a recent trend in industry to shorten cycle times and minimize costs (in gross steel tonnage and transport), by removing braces, reducing the number of shores and prematurely removing formwork. This trend is driven by the support scaffolding contractor's requirement to reduce costs to maintain competitive advantage.
SHORE LOAD INVESTIGATION

INTRODUCTION

The purpose of this work is to determine and compare the mean and theoretical shore loads in steel scaffold-type formwork supporting systems. The research was undertaken due to the distinct lack of actual measured site data comparing the mean and theoretical loads. Ten detailed on-site surveys were conducted over a period of two years across three different construction sites. The first site survey was conducted throughout June and July 2011 and involved four individual tests on separate concrete pour areas of a multi-storey shopping centre project in Merrylands, Sydney, Australia. The second site survey was conducted throughout November 2011 to January 2012 and involved two individual tests on separate concrete pour areas of a multi-storey car park development at Sydney International Airport, Australia. The third site survey was conducted throughout April 2012 to June 2012 and involved four individual tests on separate concrete pour areas of a multi-storey car park and shopping center project in Merrylands, Sydney, Australia.

BACKGROUND

Steel support scaffolds are commonly used in construction as shoring systems while building the formwork to support reinforced concrete structures. A steel support scaffold frame normally consists of standards (column members), ledgers (beam members), and braces and jacks (Figure 1 – 3). The standards are connected to ledgers via various types of connections, such as wedge-type joints and cuplok joints. The base of scaffold frames consists of jack bases whose length can be adjusted to accommodate irregularity of the ground.

Although steel scaffolds are temporary structures, their failure often has fatal consequences. The main causes of scaffold collapses are overloading (Hadipriono, 1987). Current practice in the design of steel scaffold systems is to use the load capacity recommended by the manufacturers based on load tests, and then apply a judgmental safety factor. By investigating the dead and live loads that occur during multiple on-site investigations, and comparing these with theoretical shore loads, it will be possible to understand and make judgement on the variability of shore loads.

This chapter details the equipment and experimental procedure used to acquire shore load data on three multi-storey concrete construction sites. Uprights of the scaffold system were instrumented with load cells during both the concrete casting and curing phases to obtain the actual loads transmitted to the supporting scaffolds. The load survey data are then presented and compared with theoretical shore loads calculated using tributary areas. The two components of the shoring load, i.e., the dead load and the live load, are investigated. Statistical analysis of shore loads is performed. Factors that may cause variability and non-uniform load distributions in shores are identified. The shore load information will be used to investigate the adequacy of the current shore load calculation and provide design guidelines for safer and more reliable formwork supporting systems. The work presented in this paper is part of an ongoing project which aims to develop a probabilistic-based design methodology for scaffold-type formwork supporting systems.

SITE INVESTIGATION DETAILS

The purpose of this work is to determine and compare the actual and theoretical axial loads occurring among shores in scaffold-type formwork supporting systems. A detailed on-site survey was conducted across 3 varying construction sites in Sydney, Australia. In total, ten site investigations were completed, accumulating valuable and rare data. Both dead and live construction loads were measured prior to, during and post concrete pours. The load was directly proportional to the size of the slab or tributary area of the slab above each load cell. This tributary area was limited to the possible bay sizes of the scaffolding system being 1.83m, 1.5m, 1.2m and 1.0m. Thus the maximum tributary area possible was 3.35sqm (1.83m x 1.83m) and a minimum 1.0sqm. Furthermore, slab thickness varied from 180mm up to 250mm.
Instrumentation used

20 modified U-heads were utilized in the site investigation in all four tests. The modified U-heads contained a strain gauge based stainless steel load cell with 100kN capacity (Figure 4).

![Load cell contained in U-head support](image)

Figure 2-1: Load cell contained in U-head support

A site box was used to store the testing equipment including the data acquisition system and cabling, a single computer, a cooling fan, and camera equipment; whilst the test was being conducted. The site box was completely waterproofed, earthed, locked and chained at all times. A single 20 channel Vishay V5000 data acquisition module was used to collect data at a sample rate of 0.5 seconds for each of the 20 channels. A single computer was used to store the data automatically as it was recorded by the Vishay system. The data were then exported from Strain Smart V4.01 software and interrogated using Matlab.

Experimental Study

Each of the ten on-site experimental investigations required the following summarised procedure:

1. Determine the locations for each of the 20 load cells.
2. Unscrew old and install the instrumented U-heads in these locations ensuring that the top plate of the old and the top plate of the new U-heads are in the exact same position in the vertical plane.
3. Connect all 20 load cells back to the data acquisition system using each load cells associated and calibrated cable.
4. Initiate data recording prior to concrete placement to measure dead and live loads during concrete placement and curing.
As one could anticipate, experimental site investigations occurring on a commercial site are quite complex to undertake due to time pressures, coordination of activities with other subcontractors, health and safety issues, accreditation etc. In this respect the site experimentation did not have the same freedom and precision as a controlled laboratory environment.

LOGISTICS OF SITE INVESTIGATIONS

The difficulty and logistical procedure of setting up the site testing equipment cannot be overlooked. The successful installation of testing equipment as well as the physical collection of data is an extremely complicated undertaking on a full scale commercial construction site. Of critical importance was the relationship with the project managers on site, whose duty it was to ensure that all subcontractors were coordinated and did not delay the project. It is crucial to understand the difficulty, not just in planning and coordination, but in cooperation with the project managers. The collection of this data had no financial or intrinsic benefit to the developer or project managers, and as such it was a privileged to be able to be on site. Furthermore, at times this relationship had to be managed very carefully during times of project delays, particularly when there was pressure on the project manager to push ahead. The goal of the site investigations was to maximize the amount of data collected, whilst minimizing possible interference with the construction.

Understanding construction sequence

The first logistical issue, which inhibited the investigation, was being able to fully understand the construction sequence and more importantly, the speed of construction. On a commercial construction site, it is understandable that the cycle time (between concrete pours) must be minimized to reduce the overall construction time. The issue is derived from the cost of maintaining and running a full-scale construction site, hence developers such as Stocklands® push their subcontractors hard to work fast and minimize delays. As a result of this, the cycle times between one pour to the next are often only 3 - 4 days apart. The effect of this rapid construction sequence meant that the site investigations required rapid installation and de-installation of data recording equipment.

Understanding approvals
Many approvals were required in order for the investigations to take place. The highly regulated and unionized construction industry within Australia ensured that gaining approval was not a simple process. Therefore the research and data collected from these investigations is quite valuable.

Approvals required prior to entering a site were not limited to but included the following:

- OH&S and Workcover from The University of Sydney
- Approval from the PO organization, Acrow Formwork and Scaffolding Pty Ltd
- Site Engineer – required to sign off on the location of our testing equipment and grant the use of the site crane to bring our testing equipment on to site.
- Site Electrician – required to test and certify all electrical equipment that was brought onto site
- Formwork subcontractor – required to approve placement of our load cells in their formwork system at each pour. Note this formwork subcontractor could change between one pour to the next. (Hence this relationship had to be carefully managed)
- SACL – the notoriously stringent Sydney Airport Corporation Limited (SACL) required significant paperwork before access could be granted to Site Number Two. This paperwork included:
  - SACL Contractor Safety Induction
  - Multiple Safe Work Method Statements for each activity (see appendix)
  - Electrician certification of all electrical
  - Acrow Professional indemnity insurances
  - Acrow Public liability insurances
  - Proof of residency, copies of driver’s licenses and greencards etc.
  - Note: SACL grant access to areas with direct contact to airline traffic.

Being the first test on a full scale commercial site, both the time and cost elements were obviously paramount to the formwork subcontractor. Since USYD and even the PO had both a loose connection and commercial relationship with the formwork subcontractor. Hence there was at all times a one-sided relationship occurring with no immediate cost advantages seen by the formwork subcontractor. Both the PO and on-site researchers had to continually reinforce that the experiments occurring would benefit them and the industry in the long run, a challenge in any commercial environment.

Testing equipment

The testing equipment that was taken to site had to be rugged enough to endure the harsh conditions subject to on a working, full scale construction site. All u-heads were stainless steel and fully waterproofed, they also were contained neatly between two thick steel plates to prevent damage. All electrical cords were waterproofed and connections watertight. The data logging system and computer were installed in a durable steel site box (portable equipment container) which could be wheeled and craned between tests. The site box was completely sealed from moisture, dirt and dust. It was also electrically earthed to prevent any possible electrocution. By the end of the site tests the site box had sustained many impacts from dropped equipment and was covered in dirt and dust. The site box was also equipped with a cooling fan and filtered intake port to cool the computing and data acquisition systems. The box also contained a raised bottom to protect the valuable testing equipment in the event that the integrity of the box was compromised.

Moving testing equipment

Whilst performing site investigations, it was quite physically taxing. The 20 load cells weighed over 400kg in total and were required to be manually handled and lifted up to 4.5m in height. In preparations for the site experimentation, best efforts were made to ensure that equipment was manually portable, this facilitated rapid installation and removal. At certain times required when a 100 person scaffolding team was about to deconstruct the scaffolding system. Furthermore, the data recording equipment was made to be interchangeable among scaffolding systems which varied in bay size configuration.

There were a number of organization and logistics issues involving the site box (weighing in excess of 300kg), which had to be moved each time via crane between levels. This involved booking in the lift with the crane operator the day before it was required to be moved. Furthermore, delays and emergency lifts meant that at times, a whole day would be wasted waiting for a crane to lift the site box.
Installing testing equipment

The installation of equipment for each test required approximately three full days of labor. Each load cell located high in the scaffolding frame, required an individual numbered cable (over 20m long) to be physically connected from the site box on the slab below. There were many hours of climbing up scaffolding, required to achieve this. Furthermore, to the amount of traffic on the upper working deck of the scaffolding system, all of the data cables were required to be fixed via cable ties out of the way of workmen and equipment. This process was critical to maintaining the integrity of the cables and connections. In the last two data acquisitions, careless construction workers damaged 3 of the connections rendering data acquisition impossible for these particular load cells an affecting the amount of data that was taken.

Concrete Pour Day issues

On the morning of the pour, there were some typical activities which could have affected results if they were not observed and accounted for. In some circumstances, steelworkers would come 1 hour prior to the pour and lay additional reinforcement, which directly affected load data taken.

Furthermore there was the occurrence in other pour areas (did not affect my results), where formwork contractors would come and install additional timber bearers and supports on the day of the pour. These were added to due the site engineer deeming that leg loads in standards were too great.

LOAD EFFECT NOT APPLIED LOAD

The actual load applied on the slab, known as the applied load, is quoted internationally in formwork and construction standards (e.g. AS 3610, 1995). However, it is important to understand that the amount of load, which a vertical shore receives (i.e. the shore load effect), can be vastly different to a traditional tributary area applied load analysis. Hence in order to provide statistical information for construction standards that is useful in the design of support scaffolding, there is a requirement for information on the shore load effect rather than the applied load. This requirement is made even more pertinent in temporary structures used in construction since these structures are commonly designed without the redundancies that occur in the permanent structures they support. This could justify why overloading is the main cause of scaffold failures (Hadipriono, 1987). Analysis of the shore load effect is necessitated by the highly localized and variable nature of construction loads.

Assessing the shore load effect and comparing with the applied load (actual load on slab), provides valuable information on the load-system interaction as well as the variation and magnitude of load effects. It has been suggested by others, (D. Rosowsky, Philbrick, T., & and Huston, D., 1997) that assessing this relationship could be considered an alternative approach to developing more complex structural analysis 'resistance' models.

CALCULATING LIVE AND DEAD LOADS

The purpose of these investigations was to develop more reliable statistical results, a more accurate mean-to-nominal load ratio ($\frac{\bar{D}}{D_n}$ and $\frac{\bar{L}}{L_n}$) and Coefficient of Variation (COV); for both live and dead loads. Each of the ten site investigations utilised the same analysis techniques, installation procedures and load monitoring apparatus to ensure the investigation was statistically consistent.

The Relative Dead Load is the ratio of mean-to-nominal dead load ($\frac{\bar{D}}{D_n}$) and includes the weight of wet concrete, steel reinforcement and timber formwork. It differs slightly from the statistical results calculated by other researchers (H. Ayoub, & Karshenas, S., 1994) as the ratio calculated in this research is the mean load recorded in each vertical shore, and is therefore related to the shore load effect and not the applied load (see Section "Load affect not applied load"). Furthermore, the Relative Live Load is the ratio of mean-to-nominal live load ($\frac{\bar{L}}{L_n}$) and during pouring includes the weight of workmen and equipment as well as the temporary mounding of concrete.

Nominal (Theoretical) shore loads ($D_n$ and $L_n$)

The nominal or calculated loads in shores were determined through a simplified tributary area analysis, based on the loaded area of each shore. Known as the "Simplified Method", this accounted for the weight of
formwork (0.4Kpa), reinforcement (0.4kPa) and wet concrete (22.14kN/m$^3$, based on the concrete suppliers wet density tests per pour). The simplified method was utilised due to the complex and often irregular arrangement of bearers and joists. The simplified method is known to give comparable and similar results to the more accurate beam method from proven studies by (J. Ikaheimonen, 1997). In many instances, bearers under slabs loaded a perpendicular bearer and subsequent load cell, through a timber stub column as see Figure 15, this ultimately warranted the use of the simplified method.

Figure 15 – Complexity of Load paths warranting use of the simplified method

As can be seen in Error! Reference source not found., the theoretical dead load was calculated by determining the concrete thickness, weight of formwork and weight of reinforcement. Concrete thickness was determined from the known pour depth which was recorded on engineering drawings and confirmed on site. The weight of formwork was calculated for each pour area. This was generally found to be approximately 0.4 kPa which was in agreement with the value used by Acrow Formwork and Scaffolding Pty Ltd Engineers and the same as that proposed in AS3610 for the weight of reinforcement. An example of the calculation performed can be seen in Table 4.

Although the weight of formwork and reinforcement was calculated across all ten site investigations, it was clear and evident from a sensitivity analysis into the factors affecting load calculations (see Figure 25, Figure 26, Figure 27, Figure 28) that the weight of formwork and reinforcement had little effect on the final result. This is intuitive as each component makes up less than 10% of the weight of concrete.

The theoretical shore Live Load ($L_{n}$) was calculated based on Australian standards AS 3610 (Standards Australia, 1995), during Stage II of construction. The Australian Standard requires 1 kPa live load to be applied. It must be noted that the theoretical live load results of the investigation would significantly change if a different code were adopted.

**Simplified Method**

The loads on respective standards due to reinforcement, concrete and formwork were calculated for all twenty load cells based on their respective tributary area.

**Formwork**

The self weight of formwork was obtained from the suppliers of each element, estimates where made where no data could be sourced. The following tables present the calculation of component weights of formwork and reinforcement which were used in calculation of the Nominal (theoretical) Loads.

<table>
<thead>
<tr>
<th>Formwork Element</th>
<th>Material</th>
<th>Weight</th>
<th>Load (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Soffit Ply</strong></td>
<td>17-10-7 F14 Ply</td>
<td>9.54 kg/m$^2$</td>
<td>0.094</td>
</tr>
<tr>
<td><strong>Joists, L = 3.6m typ.</strong></td>
<td>95 x 65 LVL</td>
<td><a href="mailto:5@3.6kg">5@3.6kg</a>/m</td>
<td>0.177</td>
</tr>
<tr>
<td><strong>Bearers</strong></td>
<td>50 x 77 LVL</td>
<td><a href="mailto:2@6.7kg">2@6.7kg</a>/m</td>
<td>0.131</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td></td>
<td></td>
<td>0.402 kN/m$^2$</td>
</tr>
</tbody>
</table>
Reinforcement and Post Tensioning

The nominal weight of reinforcement was determined by analysing the reinforcement drawing provided by the steelwork subcontractor and cross referencing this with both photographic evidence as well as through site inspection on the day of the pour. Furthermore, it was generally accepted that the weight due to reinforcement was typically 0.4kPa in most pour areas, which is also the accepted norm used by AS3610 as well as the design engineers from Boral Formwork and Scaffolding. (Density of Steel = 7800 kg/m$^3$). The following calculation was performed for Site 1 Investigation 2 which consisted of a post tensioned slab, a low level of reinforcement and steel decking system as seen in Table 4.

<table>
<thead>
<tr>
<th>Reinforcement component</th>
<th>Material</th>
<th>Weight kg/m$^2$</th>
<th>Load (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top (Transverse and Longitudinal)</td>
<td>SL81 Mesh</td>
<td>7.29</td>
<td>0.071</td>
</tr>
<tr>
<td>Bottom (Transverse and Longitudinal)</td>
<td>SL81 Mesh</td>
<td>7.29</td>
<td>0.071</td>
</tr>
<tr>
<td>Shear Ties + Additional</td>
<td>N12 @ 200 cts</td>
<td>4.3</td>
<td>0.033</td>
</tr>
<tr>
<td>Post Tensioning</td>
<td>5 strands @ 1000 cts both ways (0.775kg/m)</td>
<td>2*(0.775 * 5)</td>
<td>0.077</td>
</tr>
<tr>
<td>Steel Decking</td>
<td>1mm Bondek</td>
<td>13.8 kg/m$^2$</td>
<td>0.135</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><strong>TOTAL</strong> 0.387 kN/m$^2$</td>
</tr>
</tbody>
</table>

Table 4: Reinforcement weight calculation

Concrete

In all site investigations the nominal (theoretical) weight of concrete was determined by measuring the specific wet density of concrete. This was determined by averaging the calculated density of concrete using the product code and docket number of the trucks which delivered to the specific pour area. As can be seen in Table 5, the average wet density of concrete for Site 1, Investigation 1 was determined to be 22.975 kN/m$^3$.

<table>
<thead>
<tr>
<th>Date Cast</th>
<th>Source Name</th>
<th>Docket</th>
<th>Location Description</th>
<th>Product Code</th>
<th>Density_28</th>
<th>Avg Density_28</th>
</tr>
</thead>
<tbody>
<tr>
<td>21/06/11</td>
<td>Granville</td>
<td>26377376</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2280</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Granville</td>
<td>26377381</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2240</td>
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</tr>
<tr>
<td>21/06/11</td>
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<td>26377393</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
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</tr>
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<td>Granville</td>
<td>26377401</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2320</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Granville</td>
<td>26377412</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2320</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Smithfield</td>
<td>26710415</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2320</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Smithfield</td>
<td>26710419</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2310</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Smithfield</td>
<td>26710427</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2310</td>
<td><strong>2297.5</strong></td>
</tr>
</tbody>
</table>

Table 5: Site 1 Investigation 1, documentation of wet density of concrete from each truck

These calculations were impounded into the determination of Nominal (Theoretical) load
Mean (Actual) shore load ($\bar{D}$ and $\bar{L}$)

Mean (actual or recorded) shore loads were calculated on site using the experimental procedure documented above. The mean or measured load data was gathered in such a way, that the installation of load cells did not affect the way the scaffolding had been originally erected. i.e. any erection jack height discrepancies during installation by the formwork subcontractor were not changed and load cells were installed just like any regular U-head. In this way, the most accurate reflection of realistic load data could be recorded and presented. This included anomalies such as those due to overloading, initial pre-loading, gaps in U-heads etc. These will be discussed later in this report.

For each load cell LC1 to LC20 the actual recorded load was taken from the acquired data as a (kg) and then converted to kN. Dead load was measured after the concrete had been poured and all workers and equipment no longer remained on the slab. Generally this occurred at approximately 4.5 hours after the start of the pour, as seen in Error! Reference source not found.. This point in time can be seen as the plateau of all load cells after their initial spike which is due to both live and dead load. Both the spike and plateau of LC17 can be seen in Figure 5 at approximately 3.5 hours and 4.5 hours, respectively.
The actual or mean maximum live loads are known to occur at two distinct periods. Either (a) during the pouring of concrete, from the weight of workmen and equipment. And from site experience, up to 16 workers and equipment, including two mechanical vibrators, were moving around on the concrete deck. Or (b) a few hours after pouring (when ‘power trowelling’ occurs). Power trowelling is simply a process of working the concrete surface using a motorised buggy with trowel blades on its base. This process typically occurs 4-5hrs after pouring of the concrete for typically 3.5 – 4.5hrs duration; depending on wind, air temperature, type of concrete, direct sun light etc. The process of ‘power trowelling’, induces a live load after pouring has completed, into the U-heads, which is evident as short sharp positive spikes in the loading histogram for the 3.5 – 4.5hrs that it occur, as evident in Figure 4. These peaks are consistently 150–240 kg which is approximately the same weight of the power trowel and the operator. The live load peaks or spikes are not always equal to the full weight of the power trowel, this is a result of the weight being distributed throughout the concrete, ply, bearers & joists and finally into the U-heads. i.e. A power trowels live load can be spread to a number of U-heads at any one position on the slab.

Effect of Ambient Temperature Changes and Load Cell Drift

It was know after S111, that there were obviously daily temperature fluctuations which may be affecting results. Analysis of the experiment data demonstrated a distinct daily, cyclical fluctuation in load readings across all 20 load cells. It was hypothesised that this could have come as a result of temperature fluctuations during testing. An investigation was conducted to determine if ambient temperature changes would adversely affect results.

One load cell and one temperature recording device were positioned in an exposed and external location which simulated on-site conditions (Load cell 1 and Temperature logger 1). A second load cell and temperature recording device was positioned in a controlled and indoor location as a comparison to the external conditions (Load cell 2 and Temperature logger 2). Both load cells had an equal force of 2000 kg applied, although Figure 17; shows a baseline of 0 kg at time zero (this was due to zeroing of load cell). This 2000kg simulated average loading conditions which load cells were typically subjected to on site. Results of this investigation are shown in Figure 17;.
As can be seen from Figure 17; there are notable temperature correlated, cyclical fluctuations in recorded load. In a comparison of both LC1, TL1 and LC2, TL2; it is evident that the load cell exposed to a higher degree of temperature fluctuation has a greater recorded load fluctuation. i.e. the external TL1 has notably larger temperature fluctuation which correlates directly with the positive and negative gradient changes in LC1. The same is also true with LC2 and TL2 but to a lesser extent, reflecting the control or lower level of temperature fluctuation recorded in the indoor/controlled environment. However, at the key data points recorded from during site investigations (i.e. at peak dead and live loads), there was not a significant ambient temperature change from the initiation of data sampling. Furthermore, being below the slab surface, meant that the Load cells were essentially insulated from major temperature fluctuations portrayed in LC1. As such, the authors have not impounded any correction factors to the data for temperature fluctuations.

It can be seen that there is a slight downward drift, over the full data sample in both load cells, and is more pronounced in LC1 than LC2. The total drift over the 5 days of recorded data appears quite substantial in the graph, however, over the full 5 days LC2 incurs 4% actual load drift (based on applied load). Furthermore, for the range of data being recorded on site and used for analysis, i.e. 24hrs max, it is evident that there is much less of a recorded load value drift. For this period, the recorded load value drift was calculated as 0.5% for LC2 and 1.5% for LC1 of the total applied load over the timeframe of our sample. It was deemed that this was within acceptable limits and furthermore consistent with the accuracy of the load cells. As such the author has not impounded a correction in the data due to temperature drift in load cells.
SITE INVESTIGATIONS

SITE INVESTIGATION ONE

Job Name: Merrylands Shopping Centre

Job Description: 3 storey shopping centre construction consisting of post-tensioned slabs and beams.

Job Dates: 21/6/2011- 30/7/2011

Job Location: Merrylands, Sydney

Contractor: Stockland and Multiplex Joint Venture

Site Manager: Brookfield Multiplex.

Scaffolding Subcontractor: Rediform Pty. Ltd.

Detailed Site description:

The first building under investigation was a 4 storey shopping complex in Merrylands, Sydney. The general construction was a post-tensioned, one-way slab spanning 8.7m between beams and columns, as seen in Figure 2-3. Floor to Floor heights ranged from 3.8m to 7.672m, slab thicknesses ranged from 170mm to 320mm, grid spacing was 8.2m in north-south direction and 8.7m in east-west direction. Experiments were conducted on three levels of the building; however no experiments were conducted where the base plates of the standards bore on the ground, in turn eliminating any differential settlement occurring in base plates. Investigation into differential settlement effects are expected to begin in the near future. The client and owner of the site was Stocklands™, the project manager and contractor were Brookfield Multiplex Pty Ltd, whilst the formwork subcontractor was Rediform Pty Ltd.

FORMWORK AND FALSEWORK

Most common scaffolding used: 1.5m lifts, 300mm jack extension

Soffit formwork: 17-10-7 F14 Ply (17mm Thick)

Bearers: Truform 95 x 65 LVLs (varying lengths)

Joists: Truform 150 x 77 LVLs (varying lengths)

The general arrangement of formwork from top to bottom was 17mm soffit plywood, Truform 95 x 65 LVL Joists and Truform 150 x 77 LVL Bearers which spanned between U-heads and finally the bays of scaffolding which ranged in size from 1.0 to 1.83m in perpendicular directions. Furthermore, there was typically three 1.5m lifts of scaffolding with an average top jack extension of 300mm.

FOUNDATION DESCRIPTION

Base plates in direct contact with a 200mm thick slab or 600mm thick beam. These were generally 7+ days old (at least 50% strength gain) and were further back propped on at least 1 level below. Hence it was assumed no settlement or vertical translation of the base plates occurred.

CONCRETING METHODOLOGY

Concreting Method (pump): 100mm diameter hose

Number of People on Deck: Ranged from 6-16 people at any one point during pouring

Pumping rate: 60 $m^3/hr$ (approx)

Avg hose suspension height above slab soffit (Avg Drop height): 350mm (approx)

Concrete Placement Pattern: Typical “S-Pattern”

CONCRETE

Concrete Product Code: Boral PT40-20-22 @ 3-90

Concrete Type and Grade: Boral Post-Tensioned 40 MPa mix

Concrete Density: 22.531 $kN/m^3$. (average density of mix in pour area on day from suppliers specifications) Four separate pour areas were investigated on level one and level two of the site. Level one being future retail space and level two being a rooftop car park slab. In each investigation, data was collected for at least 24 hours at each location from twenty shores which each had a 100 kN load cell installed. Concrete was pumped through a 100mm diameter hose by a gang of 6-10 concreters. The pump
rate was typically 60 m$^3$/hr and the average hose suspension height above slab soffit was approximately 350mm. The concrete placement pattern was in a typical “S-shape” across the pour area (Figure 6). The concrete type and grade was a Boral Post-Tensioned 40 MPa mix. The concrete density was determined by an average of each truck load by the supplier and for pouring areas 1 – 4 respectively this was determined to be 22.531, 22.114, 22.899, and 23.144 kN/m$^3$. 

Figure 2-3: Snapshot of the General Arrangement of Level 2 beams, slabs and scaffolding layout.

Figure 2-4: Elevation A-A from Figure 2-3

Figure 2-5: Elevation B-B from figure 2-3
Site One Investigation One _ 21/6/2011

During the first investigation, shore load was measured in a slightly different way to the method documented in above. Due to the time restrictions in set up of load cells with this particular investigation, it was only possible to measure the concrete load during pouring. This occurred since the steel reinforcement and formwork deck were in place and pouring was occurring the next morning. Hence the turnaround period was too short to zero the cells at “absolute zero” and be able to measure the load due to reinforcement and formwork, in this case. The methodology was consistent with how the formwork subcontractors typically adjust the U-head supports and as such no initial shore pre-load could be recorded. The technique of investigation for site 1 pour 1 meant that load cells were zeroed with the weight of formwork and reinforcement already applied. i.e. the recorded values have only the concrete weight and live loads included.

<table>
<thead>
<tr>
<th>Site One Investigation One_ 21/6/2011</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Formwork</strong></td>
</tr>
<tr>
<td><strong>Slab</strong></td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
</tr>
<tr>
<td><strong>Post Tensioning:</strong></td>
</tr>
<tr>
<td><strong>Beams</strong></td>
</tr>
<tr>
<td><strong>Grid Spacing</strong></td>
</tr>
</tbody>
</table>

![Figure 2-6: Underside of Pour,Formwork](image1)

![Figure 2-7: Topside of Pour / Progression of Pour](image2)
Figure 2-10: Load Cell Configuration with DL and LL Ratio's for Investigated area

Figure 2-11: Loading Histogram for all Load cells (26hr Duration)
Site One Investigation Two  _ 30/6/2011

<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
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<td><strong>Formwork</strong></td>
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<tr>
<td><strong>Slab</strong></td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
</tr>
<tr>
<td><strong>Post Tensioning:</strong></td>
</tr>
<tr>
<td><strong>Beams</strong></td>
</tr>
<tr>
<td><strong>Grid Spacing</strong></td>
</tr>
</tbody>
</table>

Figure 2-12: Underside of Pour, Formwork  Figure 2-13: Topside of Pour

Figure 2-16: Load Cell Configuration with DL and LL Ratio's for Investigated area
Figure 2-17: Loading Histogram for all Load cells (18hr Duration)

**Site One Investigation Three _ 24/7/2011**

<table>
<thead>
<tr>
<th><strong>Site One Investigation Three _ 24/7/2011</strong></th>
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<tbody>
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<td><strong>Slab</strong></td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
</tr>
<tr>
<td><strong>Post Tensioning:</strong></td>
</tr>
<tr>
<td><strong>Beams</strong></td>
</tr>
<tr>
<td><strong>Grid Spacing</strong></td>
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<tr>
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</tr>
</tbody>
</table>

Figure 2-18: Underside of Pour, Formwork  
Figure 2-19: Topside of Pour
Figure 2-22: Load Cell Configuration with DL and LL Ratio's for Investigated area

Figure 2-23: Loading Histogram for all Load cells (48hr Duration)
Site One Investigation Four _ 29/7/2011

<table>
<thead>
<tr>
<th>Formwork</th>
<th>Bearers and metal decking (Bondek or similar) between traditionally formed beams.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab</td>
<td>200mm on either side of 350mm deep beam.</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>No mesh in slab, typical reinforced beams</td>
</tr>
<tr>
<td>Post Tensioning:</td>
<td>5 PT strand per duct. Ducts at 1.5m centres in both directions across slab. 5 ducts per beam.</td>
</tr>
<tr>
<td>Beams</td>
<td>600 x 1800</td>
</tr>
<tr>
<td>Grid Spacing</td>
<td>8.7m</td>
</tr>
</tbody>
</table>

|                                                                                       |
|---|----------------------------------------------------------------------------------|
|   | Carpark slab – hence falls to drains                                           |

Figure 2-25: Underside of Pour

Figure 2-26: Topside of Pour

Figure 2-30: Load Cell Configuration with DL and LL Ratio's for Investigated area
Figure 2-23: Loading Histogram for all Load cells (96hr Duration)

Figure 2-23: Loading Histogram for all Load cells (40hr Duration)
SHORE LOAD MONITORING DURING CONSTRUCTION

SITE 1 RESULTS

| Max Load | Peak Values | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | Max | AVG | STD DEV | COV |
| Load 1    |             |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| Load 2    |             |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| Load 3    |             |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |
| Load 4    |             |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |   |

Table 3 - Summary of Peak Loads, Dead Load Ratio and Live Load Ratio (Actual / Theoretical)

Site One Discussion

Instrumentation and data acquisition was undertaken in four separate pour areas between 21st June and the 1st of August 2011. A summary of the four site investigations results are shown in Error! Reference source not found.. On the basis of the survey results, it appears that the actual shore loads, on average, give good agreement with the predicted values using the tributary area method. The results of the investigation have been analyzed by engineers at Acrow Formwork and Scaffolding who have over 25 years in the industry.

Discussion: Site One Investigation One (S111)

The arrangement of load cells was specifically chosen to identify relative shore loads in both slabs and beams. It can be seen from the arrangement of load cells that 4 load cells could be placed under a full 3.6m bearer and this was done in the case of LC1-4, LC11-12, LC13-16, LC19-20 to investigate any continuity effects. There did not appear to be any continuity effect; results were instead dominated by the effects of jack height discrepancies, due to poor erection practice. From the stick plot or relative load ratios, it is evident that two central load cells positioned directly under a beam experienced a lower relative dead and live load than the load cells at the end span of the bearers. Also these load cells had a relative dead load less than 1.0, suggesting that the load experienced was less than that hypothesized (mean-to-nominal ratio less than 1.0). This could have been as a result of a number of factors including: jack height discrepancies, load path discrepancies, incorrectly measured amounts of steel due to last minute changes, concrete thickness changes etc. No continuity effects appear to occur in slab load cells, but there does appear to be a generally higher relative dead load across all slab load cells. E.g. LC5,6,7,8,9,10 all have relative load of approximately 1.2.

Interestingly the recorded load in LC 12 and LC11 drops off significantly, occurring gradually in the first few hours post pour and then more rapidly during the “power trowelling process”. These load cells were located directly beside a concrete column and noticeably reduced in total load after power troweling had occurred and night fell. The drop off almost halves the load in the load cell from dead load of 1350kg to 550kg in around 12 hours. It is postulated that the reason for this trend is due to differential temperature dependent elastic shortening affects between wide concrete columns and narrow steel scaffolding tubes. i.e. the skinny
The results of the site investigation two are quite reliable. A good mean result is indicative of a fairly accurate test, since on average over the entirety of the pour, the theoretical and average dead load was quite close to a mean relative value of 1.04. However again there is considerable variance between the mean (actual or measured) and nominal (theoretical or predicted) dead loads. Although this initial investigation has only utilized the tributary area method to calculate theoretical load, it is apparent that the large standard deviation is a result of the large variance in shore load. There are two outlier load cells yet on average the results are accurate. This suggests that the data collection and actual loads recorded are accurate to the theoretical loads on an overall scale, but there is certainly high variability between individual shores.

There are a number of distinct time dependant groups of load. It is evident that there are distinct peaks in the data as the weight of concrete passed over each load cell. In fact it is clearly visible that there were some cases of partial loading of the load cell as the full depth of concrete had not been reached in one pass. This was noted by a partial increase then a stall in load followed by a continuation of loading to the maximum height as seen in Fig.

Further evidence of the time separated distinct data groups are load cells 17 – 20. These load cells were loaded last in the pour sequence and have a distinctly different loading history from the other groups but are extremely similar as a group. As can be seen in Fig 2.

Another point to note is the distinct period of ‘power troweling’ as it is known, whereby a motorised buggy with trowel blades on its base is driven over the surface to work the surface. The process of ‘power troweling’, as discussed previously, induces a load post-pour into the u-heads, which is evident as short sharp positive spikes in the loading histogram for the 3.5 – 4.5hrs that it occur. These peaks are consistently 150-240 kg which is approximately the same weight of the power trowels that were used on site. One may notice that some of the spikes were not equal to the full weight of the power trowel, however intuitively it must be understood that this weight is distributed throughout a system of concrete, formwork and the u-heads. A classic example of this is load cell 18 which has hardly any visible load increase during power troweling, since this load cell was placed directly beside a concrete column with steel starter bars protruding from the surface of the concrete. This resulted in the adjacent U-head 17 having larger spikes as the trowel passed over it and only a small amount of load was seen by U-head 18.
is obvious that the relative dead loads beneath the slab vary significantly and there is no observable pattern of load behaviour. Load cells LC9-14 which were aligned in the row of bearers running parallel with the concrete beam (as seen in Figure 2-18), all have dead load ratio’s greater than 1 and are 1.4 on average. This is postulated to be related to the continuity effect of the steel decking and this was found to be the case in following investigations. These effects will be discussed in the following section, general observations in all pours.

It is obvious that there are large variations in relative dead load, these are potentially due to falls in the concrete. In this particular site investigation the area being measured was for a future top floor carpark and as such a minor depth change to the slab is made locally to allow for rainfall runoff to drains. These drains were typically positioned midspan between concrete beams and were the low point (i.e. shallowest slab depth). This would also lead one to ponder that perhaps the higher relative dead loads closer to the beam were due to extra concrete being used to achieve the fall gradient. i.e. a slab thicker than the 170mm design depth. Due to the commercial nature of the site, cutting or drilling through the slab to confirm the thickness was not allowed by the building contractor.

**Discussion: Site One Investigation Four (S1I4)**

The results of investigation four are quite typical loading results. As can be seen in the loading histogram, LC2,3,6,7 have consistently higher loads since they were placed directly under a 600 x 1800 concrete beam. However interestingly there is no real correlation or consistency in relative shore loads under the beam. Under the tributary area method, LC1-4 and LC5-8 were placed under identical timber bearer’s supporting the concrete beam. The tributary area method would suggest that these 8 load cells would theoretically have the same load, however the relative dead load values have no correlation or similarity. The relative dead loads of LC1-4 are 0.915, 0.927, 1.139 and 0.817 respectively and for LC 5-8 are 1.246, 1.043, 1.332 and 1.062, as seen in Fig 2-30. This indicates that as well as an apparent jack height discrepancy effect occurring along the bearer, more dead load is taken by LC5-8 than LC1-4, indicating potentially height discrepancy between bearers as more load is being applied to the bearer under LC5-8.

The lower relative loads occurring in LC15-20 are considered to be a result of falls to drains in the concrete slab as mentioned above in Investigation 3, as these load cells were placed in the low lying area nearest the drain. Again this would need to be confirmed by X-ray or drilling the slab and this was not allowed by the contractor.
SITE INVESTIGATION TWO

Job Name: Sydney Airport Carpark

**Job Description:** 8 storey car park consisting of post-tensioned slabs and beams.

**Job Dates:** November 2012 – February 2013

**Job Location:** Mascot, Sydney

**Contractor:** Stockland and Multiplex Joint Venture

**Site Manager:** Brookfield Multiplex.

**Scaffolding Subcontractor:** Rediform Pty. Ltd.

**Detailed Site description:**

The building under investigation was an 8 storey car park in Mascot, Sydney. The general construction was a post-tensioned, one-way slab spanning 8.7m between beams and columns. Floor to Floor on the first level was 2.9m, slab thicknesses ranged from 210mm to 250mm, grid spacing was 8.7m in both directions. Experiments were conducted on the first level of the building. The client and owner of the site was Sydney Airports Corporation Limited (SACL) the project manager and contractor were Brookfield Multiplex Pty Ltd, whilst the formwork subcontractor was Rediform Pty. Ltd.

**FORMWORK AND FALSEWORK**

Most common scaffolding used: 1.5m lifts, 300mm jack extension

Soffit formwork: 17-10-7 F14 Ply (17mm Thick)

Bearers: Truform 95 x 65 LVLs (varying lengths)

Joists: Truform 150 x 77 LVLs (varying lengths)

The general arrangement of formwork from top to bottom was 17mm soffit plywood, Truform 95 x 65 LVL Joists and Truform 150 x 77 LVL Bearers which spanned between U-heads and finally the bays of scaffolding which ranged in size from 1.0 to 1.83m in perpendicular directions. Furthermore, there was typically three 1.5m lifts of scaffolding with an average top jack extension of 300mm. Quite frustratingly, only two concrete pours were investigated as the formwork system was changed from Supercuplok to Acrow Props after level 1, to increase speed of construction as a result of rain delays. As a result only 2 pour areas were investigated.

**FOUNDATION DESCRIPTION**

Base plates in direct contact with a timber packers which sat on an existing asphalt carpark surface.

**CONCRETING METHODOLOGY**

Concreting Method (pump): 100mm diameter hose

Number of People on Deck: Ranged from 6-16 people at any one point during pouring

Pumping rate: 60 m³/hr (approx)

Avg hose suspension height above slab soffit (Avg Drop height): 350mm (approx)

Concrete Placement Pattern: Typical “S-Pattern”

**CONCRETE**

Concrete Product Code: Boral PT40-20-22 @ 3-90

Concrete Type and Grade: Boral Post-Tensioned 40 MPa mix

Concrete Density: 22.5 kN/m³. (average density of mix in pour area on day from suppliers specifications)

Two separate pour areas were investigated on level one of the site. Both pour’s were for a future car park. In each investigation, data were collected for at least 24 hours at each location from twenty shores which each had a 100 kN load cell installed. Concrete was pumped through a 100mm diameter hose by a gang of 6-12 concreters. The pump rate was typically 60 m³/hr and the average hose suspension height above slab soffit was approximately 350mm. The concrete placement pattern was in a typical “s-shape” across the pour area.
The concrete type and grade was a Boral Post-Tensioned 40 MPa mix. The concrete density was determined by an average of each truck load by the supplier.

Site Two Investigation One

<table>
<thead>
<tr>
<th>Site Two Investigation One</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Formwork</strong></td>
<td>Traditional Formwork consisting of Bearers and Joists at approximately 400cts. Traditionally formed beams.</td>
</tr>
<tr>
<td><strong>Slab</strong></td>
<td>210mm and 250mm on either side of beam.</td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
<td>N12’s at 300 centres in both directions.</td>
</tr>
<tr>
<td><strong>Post Tensioning:</strong></td>
<td>5 PT strand per duct. Ducts at 1.5m centres in both directions across slab. 5 ducts per beam.</td>
</tr>
<tr>
<td><strong>Beams</strong></td>
<td>600 x 1200</td>
</tr>
<tr>
<td><strong>Grid Spacing</strong></td>
<td>8.7m</td>
</tr>
</tbody>
</table>

Figure 2-30: Underside of Pour

Figure 2-31: Topside of Pour
Figure 2-37: Load Cell Configuration with DL and LL Ratio's for Investigated area

Figure 2-38: Loading Histogram for all Load cells (142hr Duration)

Figure 2-39: Loading Histogram for all Load cells (40hr Duration)
Site Two Investigation Two _ 4/1/2012

<table>
<thead>
<tr>
<th><strong>Formwork</strong></th>
<th>Bondek metal decking between traditionally formed beams.</th>
</tr>
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<tbody>
<tr>
<td><strong>Slab</strong></td>
<td>210mm and 250mm on either side of beam.</td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
<td>No Mesh in slab, typical reinforced beams</td>
</tr>
<tr>
<td><strong>Post Tensioning:</strong></td>
<td>5 PT strand per duct. Ducts at 1.5m centres in both directions across slab. 5 ducts per beam.</td>
</tr>
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<td><strong>Beams</strong></td>
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</tr>
</tbody>
</table>

![Figure 2-40: Topside of Pour](image)

**Figure 2-40: Topside of Pour**

![Load Discrepancies](image)
Figure 2-43: Load Cell Configuration with DL and LL Ratio’s for Investigated area

Figure 2-41: Loading Histogram for all Load cells (260hr Duration)

Figure 2-42: Loading Histogram for all Load cells (40hr Duration)
Table 4 - Summary of Peak Loads, Dead Load Ratio and Live Load Ratio (Actual / Theoretical)

### Discussion: Site Two Investigation One and Two

The results of Site 2, Investigation 1 are of similar trend to other investigations. On average the relative dead load was accurate across the whole pour with a mean of 1.008. Live load however displayed highly uncharacteristic results with a mean value of 1.217. Considering that equal tributary areas were being sampled across all load cells, it can be seen that there is a reasonable standard deviation in maximum leg loads. Furthermore, it can be seen from the loading histogram, that there is quite a significant difference in average load between LC1 and LC1-4 (in the order of 800kg). Shores were founded at ground level on an existing asphalt carpark. The base plates were erected on three timber spreader boards. This is a standard construction practice at the founding level. On upper levels, base plates are simply placed hard against the concrete slab with no timber spreaders (Figure 2-43).

The results are clear and conclusive since on average the ratio of mean to nominal dead load over the entire pour area is 1.008. Suggesting that on average what was recorded over the approx. 67 sqm area, is a fairly accurate representation of what was theoretically placed on each load cell. However, again, large fluctuations in individual load cell dead load ratios caused a large Std Dev and COV. Sharp drop off in Dead load occurs after 3.75 hours. This occurred due to a delay in the pour (this rarely happened). It can be seen that at the same moment, other load cells increase significantly suggesting that the gang of concrete workers pass over the affected load cells again.

Interestingly there are significant fluctuation in live load ratio in some load cells. LC1-4 have mean to nominal ratios greater than 2.0 with a mean value of 1.217 (LC2 is 2.86). These are significantly higher than ever previously recorded and demonstrate that in some cases live load may be significant. Unfortunately, there was no visual confirmation of this live load and from what caused it. It is the best anticipation by the authors that above average relative live load is due to LC1-4 experiencing higher than average peak live loads during the power trowling process as can be seen in Appendix graphs 101-104. These peaks may be due to 2 fully loaded power trowels being used due to the time pressure being placed on the contractor. This also contributed to the significantly high relative live load standard deviation of 0.764. It can be that after the initial pour LC1-4 slowly gain their full load capacity after approx. 24hours. This is the first and only time this trend
was identified. Again it can only be postulated that this is due to a redistribution of force as the concrete began to harden. Interestingly it can be seen in the loading histogram that when LC1-4 are increasing in load (7 hours after the pour is initiated until approx. 15 hours) other load cells are reducing in load. This suggests that there is some form of equalization of load occurring which seems to stabilise around 20 hours after the pour is initiated (4am in the morning).

It can also be seen that there are distinct upward spikes in the majority of load cells 24 hours after the pour begun. These were due to large stacked materials being placed on the specific area of our load cells and then moved off after approximately 2 hours. Theses materials were scaffolding pallets for an adjacent pour and weighed approximately 0.5kPa. Each load cell spike was approximately 200kg per 3.4sqm.

With almost a 500kg standard deviation in max leg load, a 0.3 STD in dead load ratio and a 0.76 standard deviation in live load ratio, it is clear that load cells of the same tributary area have vastly different maximum leg loads.

![Figure 2-43: Timber spreader boards](image)

**Site 2 Investigation 2**

The results of Site 2, Investigation 2 are of similar trend to other investigations. Investigation 2 occurred over a considerably long time duration. The full sample of data was taken over 13 days (1,100,000 seconds) as can be seen in Figure 2-41. The results of the site two are quite reliable. A mean relative dead load of 0.948 is indicative of a fairly accurate test, since on average over the entirety of the pour, the theoretical and average dead load was quite close to a mean relative value. However again there is considerable variance between the individual load cells mean (actual or measured) and nominal (theoretical or predicted) dead loads. Although this initial investigation has only utilized the tributary area method to calculate theoretical load, it is apparent that the large standard deviation of the results comes as a result of the large variance in individual shore load.

The full sample of data was taken over 13 days (1,100,000 seconds) as can be seen in Figure 2-41. There is a noticeable downward drift in all cells recorded load over this period, this is postulated to be as a result of the concrete strength increasing during this period. For example in LC19, there is a distinct load reduction from post pour dead load of 1800kg to a 13 day dead load recording of approximately 1200kgs. Considering that equal tributary areas were being sampled across all load cells, it can be seen that there is a reasonable standard deviation in maximum leg loads. Furthermore, it can be seen from the loading histogram (Figure 2-42), that there is quite a significant difference in average load in the order of 1100kg between LC19 and LC5.

Again there is a noticeable ongoing load redistribution amongst shores, occurring throughout the recorded duration. Evident in the 24 hour sample of S212 is the load redistribution between LC19 and LC20. LC19 has a significantly higher relative dead load than LC20, 1.348 and 0.943 respectively, suggesting that jack height discrepancy exists along the bearer run.
SITE INVESTIGATION 3

Job Name: Merrylands Shopping Centre

Job Description: 3 storey shopping centre construction consisting of post-tensioned slabs and beams.

Job Dates: 21/6/2011 - 30/7/2011
Job Location: Merrylands, Sydney
Contractor: Stockland and Multiplex Joint Venture
Site Manager: Brookfield Multiplex.
Scaffolding Subcontractor: Rediform Pty. Ltd.

Detailed Site description:
Site 3 was a 5 storey car park and shopping centre structure, located in Western Sydney. In total 4 separate pour areas were investigated, across 3 levels of construction. Site investigation 3 was conducted at stage 4 of construction and included four pours. The general construction was a post-tensioned, one-way slab spanning 8.7m between beams and columns. Floor to Floor heights ranged from 3.8m to 7.7m, slab thicknesses ranged from 170mm to 320mm, grid spacing was 8.2m in north-south direction and 8.7m in east-west direction.

FORMWORK AND FALSEWORK

Most common scaffolding used: 1.5m lifts, 300mm jack extension
Soffit formwork: 17-10-7 F14 Ply (17mm Thick)
Bearers: Truform 95 x 65 LVLs (varying lengths)
Joists: Truform 150 x 77 LVLs (varying lengths)

The general arrangement of formwork from top to bottom was 17mm soffit plywood, Truform 95 x 65 LVL Joists and Truform 150 x 77 LVL Bearers which spanned between U-heads and finally the bays of scaffolding which ranged in size from 1.0 to 1.83m in perpendicular directions. Furthermore, there was typically three 1.5m lifts of scaffolding with an average top jack extension of 300mm.

FOUNDATION DESCRIPTION

Base plates in direct contact with a 200mm thick slab or 600mm thick beam. These were generally 7+ days old (at least 50% strength gain) and were further back propped on at least 1 level below. Hence it was assumed no settlement or vertical translation of the base plates occurred.

CONCRETING METHODOLOGY

Concreting Method (pump): 100mm diameter hose
Number of People on Deck: Ranged from 6-16 people at any one point during pouring
Pumping rate: 60 $m^3/\text{hr}$ (approx)
Avg hose suspension height above slab soffit (Avg Drop height): 350mm (approx)
Concrete Placement Pattern: Typical "S-Pattern"

CONCRETE

Concrete Product Code: Boral PT40-20-22 @ 3-90
Concrete Type and Grade: Boral Post-Tensioned 40 MPa mix
Concrete Density: 22.5 $kN/m^3$ (average density of mix in pour area on day from suppliers specifications)

Four separate pour areas were investigated as part of stage one construction, on level two and level three of the site. In each investigation, data were collected for at least 24 hours at each location from twenty shores which each had a 100 kN load cell installed. Concrete was pumped through a 100mm diameter hose by a gang of 6-10 concreters. The pump rate was typically 60 $m^3/\text{hr}$ and the average hose suspension height above slab soffit was approximately 350mm. The concrete placement pattern was in a typical “s-shape”
across the pour area (Figure 6). The concrete type and grade was a Boral Post-Tensioned 40 MPa mix. The concrete density was determined by an average of each truck load by the supplier.

Site Three Investigation One

<table>
<thead>
<tr>
<th>Site Three Investigation One _ 11/4/2012</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Formwork</strong></td>
</tr>
<tr>
<td><strong>Slab</strong></td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
</tr>
<tr>
<td><strong>Post Tensioning:</strong></td>
</tr>
<tr>
<td><strong>Beams</strong></td>
</tr>
<tr>
<td><strong>Grid Spacing</strong></td>
</tr>
</tbody>
</table>

Figure 2-47: Underside of Pour, Formwork  Figure 2-48: Topside of Pour
Figure 2-49: Load Cell Configuration Site 3 Pour 1
Figure 2-52: Load Cell Configuration with DL and LL Ratio's for Investigated area

Figure 2-53: Loading Histogram for all Load cells (56hr Duration)
Site Three Investigation Two _ 26/4/2012

<table>
<thead>
<tr>
<th>Formwork</th>
<th>Traditional Formwork consisting of Bearers and Joists at approximately 400cts.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab</td>
<td>170mm</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>No Mesh in slab, typical reinforced beams</td>
</tr>
<tr>
<td>Post Tensioning:</td>
<td>5 PT strand per duct. Ducts at 1.5m centres in both directions. 5 ducts per beam.</td>
</tr>
<tr>
<td>Beams</td>
<td>600 x 1800 and 700 x 1800</td>
</tr>
<tr>
<td>Grid Spacing</td>
<td>8.7m</td>
</tr>
</tbody>
</table>

Figure 2-55: Underside of Pour, Formwork  
Figure 2-56: Topside of Pour  
Figure 2-57: Load Cell Configuration
Figure 2-60: Load Cell Configuration with DL and LL Ratio's for Investigated area

Figure 2-61: Loading Histogram for all Load cells (72hr Duration)
The University of Sydney

Site Three Investigation Three _ 3/5/2012

<table>
<thead>
<tr>
<th>Site Three Investigation Three _ 3/5/2012</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Formwork</strong></td>
</tr>
<tr>
<td>Bondek metal decking between traditionally formed beams.</td>
</tr>
<tr>
<td><strong>Slab</strong></td>
</tr>
<tr>
<td>170mm</td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
</tr>
<tr>
<td>One layer of mesh in slab, typical reinforced beams</td>
</tr>
<tr>
<td><strong>Post Tensioning:</strong></td>
</tr>
<tr>
<td>5 PT strand per duct. Ducts at 1.5m centres in both directions across slab. 5 ducts per beam.</td>
</tr>
<tr>
<td><strong>Beams</strong></td>
</tr>
<tr>
<td>600 x 1800</td>
</tr>
<tr>
<td><strong>Grid Spacing</strong></td>
</tr>
<tr>
<td>8.7m</td>
</tr>
</tbody>
</table>

Concrete columns poured prior (i.e. were dry)

Carpark slab – falls to drain points

Figure 2-63: Underside of Pour, Formwork
Figure 2-64: Load Cell Configuration
Figure 2-67: Load Cell Configuration with DL and LL Ratio's for Investigated area

Figure 2-68: Loading Histogram for all Load cells (53hr Duration)
### Site Three Investigation Four _ 9/5/2012

<table>
<thead>
<tr>
<th><strong>Formwork</strong></th>
<th>Bondek metal decking between traditionally formed beams.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Slab</strong></td>
<td>200mm</td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
<td>One layer of mesh in slab, typical reinforced beams</td>
</tr>
<tr>
<td><strong>Post Tensioning:</strong></td>
<td>5 PT strand per duct. Ducts at 1.5m centres in both directions across slab. 5 ducts per beam.</td>
</tr>
<tr>
<td><strong>Beams</strong></td>
<td>600 x 1800</td>
</tr>
<tr>
<td><strong>Grid Spacing</strong></td>
<td>8.7m (CHECK)</td>
</tr>
<tr>
<td></td>
<td>Columns: poured prior – dry conc</td>
</tr>
<tr>
<td></td>
<td>Carpark slab – therefore falls TO DRAIN POINTS</td>
</tr>
<tr>
<td></td>
<td>Non-typical arrangement</td>
</tr>
</tbody>
</table>

Figure 2-70: Underside of Pour, Formwork
Figure 2-71: Load Cell Configuration
Figure 2-74: Load Cell Configuration with DL and LL Ratio's for Investigated area

Figure 2-75: Loading Histogram for all Load cells (165hr Duration)
SHORE LOAD MONITORING DURING CONSTRUCTION

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Figure 2-76: Loading Histogram for all Load cells (40hr Duration)

SITE 3

Table 6 - Summary of Peak Loads, Dead Load Ratio and Live Load Ratio (Actual / Theoretical)
Site Three Discussion

The results of site three are quite reliable. The theoretical and average dead load was quite close to a mean relative value of 1.141. However again there is considerable variance between the actual (measured) and theoretical (predicted) dead loads. Although this initial investigation has only utilized the tributary area method to calculate theoretical load, it is apparent that the large standard deviation of the results comes as a result of the large variance in shore load across all four site investigations. Dead load standard deviation ranged from 0.29 to 0.68 and on average were 0.427. In pour number four LC1, LC9, and LC19 experienced approximately double the theoretical dead load.

In fact for tests one to four the smallest value of standard deviation was in test four, s.d. = 0.203. There are many outlier load cells yet on average the results are accurate. This suggests that the data collection and actual loads recorded are accurate to the theoretical loads on an overall scale, but again there is high variability between individual shores.

Again the method of installing all site investigation equipment did not in any way alter with the pre-existing installation techniques, and as such the results are a true reflection of full scale loads occurring in shores. It was decided that for site three, investigations would focus on the loading in slab area’s rather than the beams. This was due to both the complexity in determining load paths, which affected the theoretical load calculation. And also the difficulty in installing u-heads, as these standards were generally more highly loaded and spaced closer together, resulting in confined spaces. It was deemed that the four investigations at site one, accurately captured the loads occurring in beams.

On the basis of the survey results, it appears that the actual shore loads, on average, give good agreement with the predicted values using the tributary area method.

S3P1

The load cell layout has three distinct rows of bearers between the 1800 x 600 deep concrete beams as seen in Fig 2-49. Row 1 has LC 1 to LC 6, Row 2 has LC7 to LC19 and Row 3 has LC13 to LC20. There is a general trend in the data which suggests that the row of bearers closest to the concrete beams takes a larger amount of load even though it isn't directly affected (i.e. they have the same tributary area) by the concrete beam, See Fig 2-47. There is one last row of bearers (shown on the LHS and RHS of Fig 2-49) which would take the residual load from the beam, however there appears to be a continuity effect across the bondek as load appears to also be overloading the inner set of bearers of Row 1 and Row 3. In almost all cases Row 1 and Row 3 have dead load ratios greater than 1, whilst Row 2 has DL Ratio less than 1, confirming the apparent continuity effect occurring in bondek. This result, although not critical to loading, would be extremely useful for scaffolding design engineers, particularly where deep beams are being poured. Further analysis must be completed into this apparent affect of continuity. The manufacturers of Bondek may have some further insight into this occurrence, however this is outside the scope of this research.

The loading histogram (Fig 2-53) depicts some important information. Since all LC had a similar tributary area and were placed under the same slab of thickness 210mm, they are directly comparable. It is therefore interesting to note that there is again a transfer of load occurring during the curing phase. LC4 and LC7 shed load from hours 8 to hour 12. Whilst LC8 and LC9 appear to gain load during this period. The transfer is presumed to be due to temperature effects on steel reinforcement, and timber bearer continuity.

Pour 1 results form good agreement on average between theoretical and actual results, as can be seen by the average of 1.036 across all 20 load cells. Furthermore a standard deviation of 0.291 and COV of 0.281 indicates again that there is much more deviation in dead load between mean and nominal results.

S3P2

The loading histogram again presents a similar picture. Outliers in the histogram are obviously LC9, LC10 and LC11 who are all positioned directly under a 700 x 1800 beam. LC10 takes the brunt of the load and has over double the load of LC 9 and LC11. This information is critical in design of scaffolding systems and is a useful tool for future formwork design. It could be argued that a multiplying factor be applied to the central LC under a beam, for design purposes, however the occurrence of such higher loads would need to be qualified by further investigation.

It is interesting to note that most load cells appear to drop off in load around 12 hours after the pour commences. These falls are more than the typical temperature fluctuations and it is argued that this is again
a redistribution of load occurring potentially due to the 25% stressing of post tensioning cables in the slab. It is clear that LC1, LC4 and LC6 drop off abruptly whilst LC17, LC8 and LC20 appear to only fluctuate in load due to normal daily temperature fluctuations as seen in Figure 2-61.

Furthermore, it appears that the load cells under the beam do not record any change in load during this period, and this is assumed to be due to the thickness of the beam insulating the concrete and load cells from ambient temperature changes.

**S3P3**

The loading histogram (Figure 2-58) presents typical loading information for the initial dead load due to the weight of concrete and the live load from workmen, equipment and impact forces. Again, load cells are seen to redistribute load and fluctuate slightly during the curing phase, however these fluctuations are quite insignificant in proportion to the magnitude of load. i.e. in the order of 5-10%. The relative live load is quite high \((L/L_n = 1.6)\) and this suggests that the 1kPa suggested in the Australian Standards may not be appropriate.

The higher dead load ratio values noted for LC 17, 18, 19 and LC20 are due to them existing on an end span with an unusual formwork and false work configuration, notably evident in Figure 18.

![Figure 18: LC20 and formwork configuration](image)

**S3P4**

In site investigation four, it is apparent that LC1 has an extremely large peak value of almost 30kN. This result can also be seen in the much higher relative dead load ratio. LC1 was placed centrally under a 600 x 1800 concrete beam, and thus experiences higher than average load. It is clear that no load sharing occurred in this case, with the central load cell experiencing the majority of the load. The higher than expected load is due to the inadequacies of the tributary area method for load cells under concrete beams, which will be discussed below.

The load cells which were placed under the slab experience an above average relative dead load with a mean value for all twenty load cells of 1.447, with a number of load cells in the 1.4 to 1.7 range, indicating a higher than expected dead load. Again this could be due to a combination of jack height discrepancies, thicker than recorded slab areas due to carpark falls and bearer continuity effects.

Again their appears to be a steel decking end span effect on load, with bearers supporting the end span of steel decking (LC5, 7,9,11,15,17,19) experiencing an average relative dead load of 1.44 whilst the adjacent load cells (LC6,8,10,12,16,18,20) experienced an average relative dead load of 1.13, as seen in the stick plot Fig 2-74.

There are some particularly high relative live loads occurring in LC 6, LC8 and LC12, being 1.719, 1.327 and 1.391. These were known to occur during concrete pouring where 14 workmen & equipment were on the deck at the time and most likely concentrated over these load cells. (as seen in appendix figures 186, 188
and 192). Since the relative load is greater than 1, it is clear that these particular shores would be over loaded in comparison to the calculated 1kPa code requirement.

**SUMMARY OF RESULTS**

### SITE INVESTIGATION 1

<table>
<thead>
<tr>
<th>Dead Load</th>
<th>Mean-to-Nominal Ratio ( \left( \frac{D}{D_n} \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MAX</td>
</tr>
<tr>
<td>Pour 1</td>
<td>1.005</td>
</tr>
<tr>
<td>Pour 2</td>
<td>1.831</td>
</tr>
<tr>
<td>Pour 3</td>
<td>1.614</td>
</tr>
<tr>
<td>Pour 4</td>
<td>1.395</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>0.981</td>
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</tbody>
</table>

### Live Load

<table>
<thead>
<tr>
<th>Live Load</th>
<th>Mean-to-Nominal Ratio ( \left( \frac{L}{L_n} \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MAX</td>
</tr>
<tr>
<td>Pour 1</td>
<td>1.283</td>
</tr>
<tr>
<td>Pour 2</td>
<td>1.418</td>
</tr>
<tr>
<td>Pour 3</td>
<td>1.068</td>
</tr>
<tr>
<td>Pour 4</td>
<td>1.272</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>0.740</td>
</tr>
</tbody>
</table>

**MAX LEG LOADS (kg)**

<table>
<thead>
<tr>
<th>Max Leg Loads (kgs)</th>
<th>MAX</th>
<th>AVG</th>
<th>STD DEV</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1</td>
<td>2040</td>
<td>1483</td>
<td>238.023</td>
<td>0.161</td>
</tr>
<tr>
<td>Pour 2</td>
<td>3746</td>
<td>2198</td>
<td>792.205</td>
<td>0.360</td>
</tr>
<tr>
<td>Pour 3</td>
<td>2265</td>
<td>1728</td>
<td>526.335</td>
<td>0.305</td>
</tr>
<tr>
<td>Pour 4</td>
<td>3612</td>
<td>2235</td>
<td>552.513</td>
<td>0.247</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>kg</td>
<td>3746</td>
<td>1911</td>
<td>527.269</td>
</tr>
<tr>
<td></td>
<td>KN</td>
<td>36.736</td>
<td>18.738</td>
<td>5.171</td>
</tr>
</tbody>
</table>

Table 7: Summary of Mean to Nominal (Actual / Theoretical) Dead Load Ratio and Live Load Ratio for Site 1
### SITE INVESTIGATION 2

#### Dead Load

<table>
<thead>
<tr>
<th>Pour</th>
<th>MAX</th>
<th>MIN</th>
<th>AVG</th>
<th>STD DEV</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1</td>
<td>1.400</td>
<td>0.581</td>
<td>1.008</td>
<td>0.307</td>
<td>0.305</td>
</tr>
<tr>
<td>Pour 2</td>
<td>1.348</td>
<td>0.827</td>
<td>1.003</td>
<td>0.254</td>
<td>0.253</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>1.006</td>
<td>0.281</td>
<td>0.279</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Live Load**

<table>
<thead>
<tr>
<th>Pour</th>
<th>MAX</th>
<th>MIN</th>
<th>AVG</th>
<th>STD DEV</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1</td>
<td>2.864</td>
<td>0.720</td>
<td>1.217</td>
<td>0.764</td>
<td>0.627</td>
</tr>
<tr>
<td>Pour 2</td>
<td>1.054</td>
<td>0.633</td>
<td>0.881</td>
<td>0.217</td>
<td>0.246</td>
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<tr>
<td>AVERAGE</td>
<td>1.049</td>
<td>0.490</td>
<td>0.437</td>
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<td></td>
</tr>
</tbody>
</table>

**Max Leg Loads (kg)**

<table>
<thead>
<tr>
<th>Max Leg Loads (kgs)</th>
<th>MAX</th>
<th>AVG</th>
<th>STD DEV</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1</td>
<td>2759</td>
<td>2171</td>
<td>537.134</td>
<td>0.247</td>
</tr>
<tr>
<td>Pour 2</td>
<td>2266</td>
<td>1746</td>
<td>429.938</td>
<td>0.246</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>2759</td>
<td>1959</td>
<td>483.536</td>
<td>0.247</td>
</tr>
</tbody>
</table>

Table 8: Summary of Mean to Nominal (Actual / Theoretical) Dead Load Ratio and Live Load Ratio for Site 2

### SITE INVESTIGATION 3

#### Dead Load

<table>
<thead>
<tr>
<th>Pour</th>
<th>MAX</th>
<th>MIN</th>
<th>AVG</th>
<th>STD DEV</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1</td>
<td>1.350</td>
<td>0.731</td>
<td>1.036</td>
<td>0.291</td>
<td>0.281</td>
</tr>
<tr>
<td>Pour 2</td>
<td>1.424</td>
<td>0.335</td>
<td>1.039</td>
<td>0.404</td>
<td>0.388</td>
</tr>
<tr>
<td>Pour 3</td>
<td>1.263</td>
<td>0.830</td>
<td>1.044</td>
<td>0.336</td>
<td>0.322</td>
</tr>
<tr>
<td>Pour 4</td>
<td>2.142</td>
<td>0.930</td>
<td>1.447</td>
<td>0.678</td>
<td>0.469</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>1.141</td>
<td>0.427</td>
<td>0.374</td>
<td></td>
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</tr>
</tbody>
</table>

#### Live Load

<table>
<thead>
<tr>
<th>Pour</th>
<th>MAX</th>
<th>MIN</th>
<th>AVG</th>
<th>STD DEV</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1</td>
<td>0.959</td>
<td>0.688</td>
<td>0.832</td>
<td>0.203</td>
<td>0.244</td>
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<tr>
<td>Pour 2</td>
<td>1.072</td>
<td>0.206</td>
<td>0.684</td>
<td>0.313</td>
<td>0.458</td>
</tr>
</tbody>
</table>
### MAX LEG LOADS (kg)

<table>
<thead>
<tr>
<th>Max Leg Loads (kgs)</th>
<th>MAX</th>
<th>AVG</th>
<th>STD DEV</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1</td>
<td>2381</td>
<td>1919</td>
<td>509.950</td>
<td>0.266</td>
</tr>
<tr>
<td>Pour 2</td>
<td>3361</td>
<td>2165</td>
<td>818.332</td>
<td>0.378</td>
</tr>
<tr>
<td>Pour 3</td>
<td>2640</td>
<td>2022</td>
<td>708.407</td>
<td>0.350</td>
</tr>
<tr>
<td>Pour 4</td>
<td>2947</td>
<td>1756</td>
<td>859.102</td>
<td>0.489</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>3361</td>
<td>1965</td>
<td>723.948</td>
<td>0.368</td>
</tr>
<tr>
<td>KN</td>
<td>32.960</td>
<td>19.272</td>
<td>7.100</td>
<td>0.368</td>
</tr>
</tbody>
</table>

Table 9: Summary of Mean to Nominal (Actual / Theoretical) Dead Load Ratio and Live Load Ratio for Site 3
Maximum Leg Loads

Of major significance is the surprisingly low maximum leg loads recorded during all site investigations. The average maximum leg load (dead load + live load) recorded across the 4 investigation on Site 1 was 18.74 kN and a individual maximum recorded leg load of 36.74 kN. Site Investigation 2 had generally low maximum leg loads and COV values, the maximum recorded leg load was 27kN. Whereas Site 3 had an average maximum leg load of 19.27 kN and a high individual maximum recorded leg load of 32.96 kN. These results can be seen in Table 6-7.

The results of the max recorded individual leg loads are significant and suggest a much lower COV and standard deviation than the recorded dead and live load values used for calculating relative mean to nominal loads. The maximum total leg load loads recorded are also significantly lower than the ultimate capacity of an individual standard. The data collected suggests that there is a factor of safety in design of around 2.5 (for combined dead and live load). The authors of this paper argue that despite the higher than expected standard deviation and co-efficient of variation for individual shore loads arising during site investigations, a safety factor of 2.5 appears excessive. Furthermore with advanced analysis, calibration to actual full scale models and target reliability indices, as has been achieved in this thesis, it is expected that this safety factor can be significantly reduced, leading to more efficient and economical designs.

<table>
<thead>
<tr>
<th>Site Investigation 1</th>
<th>Average Recorded Leg Load</th>
<th>Standard Deviation</th>
<th>COV</th>
<th>Recorded Max Leg load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1 (kg)</td>
<td>1482.70</td>
<td>238.02</td>
<td>0.160</td>
<td>2040.00</td>
</tr>
<tr>
<td>Pour 2 (kg)</td>
<td>2198.05</td>
<td>792.20</td>
<td>0.360</td>
<td>3746.00</td>
</tr>
<tr>
<td>Pour 3 (kg)</td>
<td>1727.63</td>
<td>526.33</td>
<td>0.304</td>
<td>2265.00</td>
</tr>
<tr>
<td>Pour 4 (kg)</td>
<td>2234.60</td>
<td>552.51</td>
<td>0.247</td>
<td>3612.00</td>
</tr>
<tr>
<td>Total (kN)</td>
<td>18.74</td>
<td>5.17</td>
<td>0.268</td>
<td>36.74</td>
</tr>
</tbody>
</table>

Table 6: Site Investigation 1 Maximum Leg Loads Recorded (Dead Load + Live Load)

<table>
<thead>
<tr>
<th>Site Investigation 2</th>
<th>Average Recorded Leg Load</th>
<th>Standard Deviation</th>
<th>COV</th>
<th>Recorded Max Leg load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1 (kg)</td>
<td>2171.26</td>
<td>537.13</td>
<td>0.247</td>
<td>2759.00</td>
</tr>
<tr>
<td>Pour 2 (kg)</td>
<td>1745.89</td>
<td>429.94</td>
<td>0.246</td>
<td>2266.00</td>
</tr>
<tr>
<td>Total (kN)</td>
<td>19.21</td>
<td>4.74</td>
<td>0.247</td>
<td>27.07</td>
</tr>
</tbody>
</table>

Table 7: Site Investigation 2 Maximum Leg Loads Recorded (Dead Load + Live Load)

<table>
<thead>
<tr>
<th>Site Investigation 3</th>
<th>Average Recorded Leg Load</th>
<th>Standard Deviation</th>
<th>COV</th>
<th>Recorded Max Leg load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour 1 (kg)</td>
<td>1918.74</td>
<td>509.95</td>
<td>0.266</td>
<td>2381.00</td>
</tr>
<tr>
<td>Pour 2 (kg)</td>
<td>2164.50</td>
<td>818.33</td>
<td>0.378</td>
<td>3361.00</td>
</tr>
<tr>
<td>Pour 3 (kg)</td>
<td>2021.56</td>
<td>708.41</td>
<td>0.350</td>
<td>2640.00</td>
</tr>
<tr>
<td>Pour 4 (kg)</td>
<td>1756.06</td>
<td>859.10</td>
<td>0.489</td>
<td>2947.00</td>
</tr>
<tr>
<td>Total (kN)</td>
<td>19.27</td>
<td>7.10</td>
<td>0.371</td>
<td>32.96</td>
</tr>
</tbody>
</table>

Table 7: Site Investigation 3 Maximum Leg Loads Recorded (Dead Load + Live Load)
GENERAL OBSERVATIONS ACROSS ALL TEN SITE INVESTIGATIONS

Through all ten site investigations, there were some obvious trends noted in the data collected. There were some key factors identified on-site that caused cases of distinct variability and non-uniform load distributions in shores. These were identified during the testing and account for the variability between mean (actual) and nominal (theoretical) dead and live loads.

Effect of Temperature fluctuations

Large daily fluctuations in temperature were noted during experimentation. The temperature differential between cool nights and warm days were known to affect results in two ways. The first was effecting testing equipment, specifically the load cells. As such a full analysis was performed to investigate the effect of these temperature fluctuations on all testing equipment (see Figure 17: Effect of temperature fluctuations on Load Cells). The results of this investigation, concluded that the effects of temperature fluctuations were within acceptable limits and furthermore consistent with the accuracy of the load cells. As such the author impounded no correction factor to the data due to temperature drift in load cells.

The second way in which temperature fluctuations affected results was via differential temperature dependant elastic shortening affects between wide concrete columns and narrow steel scaffolding tubes. From the experimental data, there was a notable and distinct drop off in load for cells adjacent to concrete columns (even when columns were poured at the same time as the slab). As noted in the site specific discussion for S1I2, there was a distinct drop off in load for cells adjacent to the columns LC17, LC18, LC19 and LC20. The same occurrence was noted in LC11 and LC12 in S1I1, which were positioned adjacent to a concrete column and can be clearly seen to reduce in total load after power troweling had occurred and night fell. In this case the skinny steel shores would elastically shorten as temperatures drop and more load would be traceable back to the large concrete column, which was not as adversely affected by reducing temperatures. The results of this were impounded into the data, but did not affect the dead and live load statistics, since these were essentially taken 4-5 hours after the pour.

Bearer Continuity Effects

Bearer continuity occurs as a result of overlapping bearers in a U-head. The phenomenon occurs as a result of the end projecting portion of a bearer attracting load as the beam deforms and deflects upwards as seen in Figure 19. The result is a higher load in the centre standard. It is estimated by researchers (J. Ikaheimonen, 1997) that this load attraction generates 55% to the centre standard and 45% to the outside standards.

Figure 19: Bearer Continuity Effects (Adopted from Ikaheimonen (1997))

It was expected that there would be noticeable bearer continuity based loading discrepancies in results, as indicated by other research. There were instances where this was the case, however in general, there were larger forces at play. Namely discrepancies in jack height, which had a much more significant effect on loading results. As seen in Figure 20, there is a central row of Bearers (Orange timbers), and in general it was clear that an unequal distribution of load sharing was occurring. As some load cells displayed relative dead and live loads which were significantly higher than adjacent load cells.
It is apparent where timber bearers were used to support deep concrete beams, that the tributary area method did not accurately calculate nominal loads. In a number of investigations the centre U-head of a three bearer support took more load than anticipated. This is easy to understand, since the left, centre and right jack would take 0.5L, 1.0L and 0.5L by the tributary area method, similar to the reactions in Figure 19. However in actuality by engineering principles the reactions at each point in Figure 19 are 0.375L, 1.25L and 0.375L, respectively from left to right. It is argued by the authors, that a factor equal to 1.25, should be impounded into the design of the system for the central jack in a bearer run (Figure 19) where it exists under a concrete beam.

**Affects of Post Tension (PT) Cables**

There are some distinct spikes in all data occurring after the first 24 hours of data acquisition. These are known to correlate to the times when the Post Tension (PT) cables within slabs and beams are tightened by the PT subcontractor, using a pneumatic jack. Generally this occurs 24 hours after the pour, where each PT strand is tensioned to 25% of its working tension. This arguably causes the permanent structural system to alter its pre-existing load paths and hence a change in load cell data is observed. This change in load path is clearly visible at hour 28 of S3I2 as seen in Figure 2-62. A drop in load magnitude occurs across all load cells in the order of 200kg in slabs to 500kg in beams, with some dropping further than others, however with no apparent correlation. Since no load appears to come back on after stressing, it is apparent that the load has been redistributed. Since the drape of PT is designed to sag through slabs and beams and hog over columns, it is intuitive to assume that this load redistribution is most likely to occur down through the concrete columns.

The Post Tension cables are then stressed to their full 100% stressing load when the concrete has achieved 22MPa for 12.7mm strands and 25MPa for 15.2mm strands. This occurs typically 4 days after the pour as the standard P/T slab mix is a 22MPa at 4 days, mix. The 100% stressing load can be observed in S2I1 at 450,000 seconds (125 hours or 5.2 days). There is a significant load reduction in each load cell dropping 1000kg on average. It is also apparent that after the PT cables have been stressed there is a minor increase in load and this is postulated to be due to elastic relaxation of cables as seen in S1I4.

**Load from Stacked materials**

It can be clearly seen in the loading histograms from each tests that there are daily spikes approximately 24 hours after the day of the pour. These peaks in load occur as workers, equipment and stacked materials, traffic the new slab. Quite surprisingly, in some circumstances, there was as little as 15 hours curing time before the slab was trafficked by workmen the following day. Typically these additional loads caused spikes in the load cell that were higher than the previous days pour loads, however these results were not considered to be during the critical Stage II, rather Stage III (as per AS 3610) and thus not statistically considered. It is important to understand that load spikes and peaks occurring 24 hours or more after the pour were not considered critical. This was due to the fact that the concrete had 24 hours to set and therefore had gained strength in the order of 10-15MPa (in a 40MPa concrete mix).
The non-critical nature of these secondary peaks in shore load can be identified by the following analogy. If a truck were to drive through the scaffolding system below and eliminate a full bay, the concrete slab would most likely be able to hold its own weight after a day of curing, with 10-15MPa strength. As such peak loads occurring after 24 hours were not considered.

Although not directly related to this thesis (since they occur in the “non-critical” Stage III of construction), stacked materials loads were recorded in almost all ten site investigations. In most cases, materials such as PT cable bundles and scaffolding system components including timber formwork, steel reinforcement etc, was stacked on the new slab the day after a pour (i.e. 24 hours after the pour), see Figure 21. The loading effects and peak loads caused by materials being craned or forklifted onto a new slab (which still had load cells and scaffolding supporting it below) can be seen in most of the loading histograms around 24 hours after the pour (seen in Figure 22). It is prudent to understand that the spikes occurring between hour 20 and 30 are due to impact forces on particular load cells. The load is then distributed through the slab and into a number of shores which may be affected by the impact. This phenomenon is seen as a general increase in all load cells during the 20-30hour period. For example, it can be seen in S111 that after 80,000 seconds (22hrs) there is a distinct 200kg increase in load cells. The increase in load of 200kg is due to stacked materials being placed in the early morning.
Jack Height Discrepancies

One of the most critical findings of the investigation was that the portion of load attracted to a particular shore was extremely sensitive to minor jack height changes. That is, if U-heads are not perfectly level, one shore can attract significantly more dead load than an adjacent shore. In many cases, the relative shore load data confirmed that load sharing occurred between two shores on the same beam run. It was clear that where a bearer spanned over a shore with slightly lower height, that this shore would have a relative load less than 1.0 and the adjacent shores a relative loads greater than 1.0.

From a site investigation into the effect of minor U-head or jack height changes, quantitative values were determined prior to the concrete pour. In some cases whilst performing site investigations, it was visibly apparent that some U-heads installed by the subcontractor were actually 5-10mm below the timber bearers (see Formwork Construction Sequence Section), meaning a potential overload of the adjacent standard.

Although it was deemed impossible to do the same test post concrete pour (due to higher load and a rigid slab), a good insight into the effect of relative U-head height differences was determined. In this particular site experiment, a U-head was unscrewed and unloaded allowing the timber bearer to span between the two adjacent U-heads. This effectively zeroed out all load from the cell (tare the load cell), which included the weight of formwork and reinforcement. The U-head was then re-screwed to its original position, using the jack assembly. Following this procedure, it was noted that 5.7 kN of load was present, which concurred with the theoretical calculation of load used by Acrow engineers. This included both a formwork and reinforcement load component as follows:

\[
\text{Tributary Area} = 1.83m \times 1.83m = 3.35sqm
\]

\[
\text{Formwork (1.2 kPa) + Reinforcement (0.5kPa)} = 5.7 \text{kN}
\]

It was also calculated in another load cell (seen in Table 18), after zeroing the cell and then re-screwing the jack to its original position, so that it bore the full weight of formwork and reinforcement, that there was 1250kg present. Furthermore, the adjacent load cell registered -180kg, suggesting that the initial shore preload significantly impacts the final load in the shore. This trend was further qualified by the data, which suggested that where the ratio of mean to nominal shore preload is greater than 1.0, the final load in the shore will typically be greater than the theoretical (i.e. relative load ratio greater than 1.0). And vice-versa for relative dead load ratios less than 1, which was in fact the case in 70% of the cells in Site One, Pour 2. This result was also observed by (D. P. Rosowsky, 1997) who suggested that a magnification factor of 2.0 may be appropriate to account for the spatial variability among a group of shores in a common pour area. (D. Rosowsky, Philbrick, T., & and Huston, D., 1997) has also commented that the degree of shore variability appears to be a function of the amount of pre-compression imparted during installation, which the authors of this report also confirmed.

The results of the tests conducted on two separate load cells in site three investigation two (S3I2) are evident in Table 18. It was determined that rotations of each jack in the shore preload stage (formwork plus the weight of reinforcement) had the following effects:

<table>
<thead>
<tr>
<th>Number of Rotations</th>
<th>Change in U-head Height</th>
<th>Load Cell A (kg)</th>
<th>Difference (kg)</th>
<th>Load Cell B (kg)</th>
<th>Difference (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 full 360 degree rotation</td>
<td>12.6 mm</td>
<td>1250</td>
<td>1600</td>
<td>1470</td>
<td>1600</td>
</tr>
<tr>
<td>1 full rotation</td>
<td>6.3mm</td>
<td>500</td>
<td>850</td>
<td>720</td>
<td>850</td>
</tr>
<tr>
<td>Original Installed Position</td>
<td>0mm</td>
<td>-220</td>
<td>570</td>
<td>440</td>
<td>570</td>
</tr>
<tr>
<td>Loose</td>
<td>-350</td>
<td>0</td>
<td>-130</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 18: Effect of height change in U-heads on load in standards

Essentially the results presented in Table 18 demonstrate that a U-head which is not at its correct level will either have a reduced or increased load depending on whether it has been installed lower or higher than the other U-heads which surround it, respectively. It can be seen that one full 360 degree revolution produces 6.3mm of height change and generates 850kg (8kN) of force, or 280kg (2.5kN) more than its original installation load. Furthermore, two full rotations of 720 degrees produce 12.6mm of height change and 1600 kg (15kN) of force, or 1030kg (10kN) more than their original installation load.
Bearer Height Discrepancies

In Site 1 Investigation 4, it was clear that two adjacent parallel timber bearers which were of equal length, equal tributary area and had the same configuration of formwork and standards supporting them (essentially identical) had vastly different relative dead loads. In fact the cumulative dead load average for Load cells on one bearer were less than 1.0 and on the adjacent parallel bearer greater than 1.0. For example in Figure 20, load cells supporting the orange bearer on the left hand side had relative dead loads greater than 1.0, whilst the load cells supporting the orange bearer on the right hand side of the figure, had relative dead loads less than 1.0.

This suggests that as well as jack heights playing a role there is also the effect of adjacent bearers taking an uneven distribution of load. This was particularly apparent under concrete beams, where many sets of bearers supported by three or four standards were used and joists parallel with concrete beams could span between three bearers.

Pouring Slab with Kibble (Bucket)

A kibble was used on Merrylands site one (S1I4), it was used to finish of the slab as not enough concrete was pumped from the last truck. Hence a new truck filled up the one cubic meter kibble and deposited the required concrete to the slab via crane. As is noted in Figure 23, the kibble is depositing at a height of approximately 300mm from the top of the slab. This was the maximum drop height the kibble achieved and in terms of impact loading and mounding effects it is quite obvious that they would be minimal. Unfortunately no load cell existed beneath the bucket drop site. In any case an analytical check of the load resulting from this impact event was performed.

![Figure 23 Kibble (Bucket) in use at S1I4](image)

Force calculation (conservative)

\[ F = ma \quad mass = 0.05m \times 0.45m \times 0.5m \times \frac{2500 \text{ kg}}{m^3} = 28.125 \text{ kg} \]

\[ acc^v = 4.905 \frac{m}{s^2} \]

\[ F = 140 N = 0.14kN \]

Thus the additional load in this case is only 0.14kN or 0.5% of average leg loads, and will therefore be ignored. Note the acceleration coefficient is a conservative assumption based on the fact that the kibble shute is angled at approx. 60 degrees from horizontal and conservative estimates for friction and viscosity of concrete.
Concrete Mounding effect
From a visual inspection of concrete mounding occurring during pouring by both pump or kibble and from photographic evidence collected during site investigations, the effect of mounding concrete can be calculated as:

\[ V = \frac{1}{3} w x d x h \]

\[ F = \frac{1}{3} x 0.75m x 0.5m x 0.3m x \frac{25kN}{m^3} = 0.9375kN \]

Overall Load = Impact forces (above) + mounding effects

\[ F = 0.14kN + 0.9375kN = 1.1kN \text{ over a } 0.75 \times 0.5m \text{ area} \]

\[ F = 2.9kPa \]

It must be noted that the mounding effects alone considered by AS 3610 is \( Q_c = 3.0 \text{kPa over } 1.6 \text{ m } x \text{ } 1.6m. \)

Hence, the standard seems to estimate the overall load from impact and mounding forces quite well.

Peak Live Loads
Peak live loads were known to occur at two periods of time in construction Stage II, during the concrete pour when impact loads and workmen were involved, or during power trowelling, the process of working the concrete surface.

It was found that approximately half of all load cells had peak live loads occurring during power troweling (3-5hrs after pouring). During this time the concrete would not have gained enough strength to be self supporting and is therefore at risk of collapse. These peak live loads were therefore considered in the statistical investigation. The other half of peak live loads were experienced when a gang of concrete workers (up to 16 in some cases) and equipment were pouring the concrete.

Continuity Effect in steel decking
There was an apparent continuity effect occurring due to the span of steel decking. In a number of tests where steel decking was used as lost formwork, load cells supporting the end span of steel decking typically had higher dead loads than load cells supporting bearers midspan of a bondek sheet. This is known to be due to the fact that the last span of the steel sheet rests on the top edge of the concrete beam’s formwork and therefore does not have a rigid load path down a steel shore. Instead load would track through the often complex system of timber bearers which form the concrete beam shell. The following figure identifies the direction of the steel decking span, running perpendicular to the concrete beam and timber bearers. The load cells in the foreground are the centre row of bearers whilst in the background the higher relative dead load LC’s are seen beside the concrete beam’s formwork. The continuity effect as a result of end spans and steel decking was identified but not considered statistically relevant to this investigation, as it was the variance in shore loads that were being investigated, and the continuity of steel decking was merely a contributing factor to this.
Possible causes of discrepancy affecting results

There were some issues noted by the authors which could have possibly affected results and have therefore been disclosed for future analysis purposes. Firstly, the actual thickness of the slab being poured was not perfectly known, rather it depended on the accuracy of the concrete pour by the subcontractor. It was not possible to measure the exact completed pour thickness as this would have required drilling through the new slab and this was disallowed by the contractor.

Secondly, in open and exposed concrete slabs of rooftop carparks, there was a requirement for concrete to be mounded and fall towards a drain point. This would have had the effect of creating a variable thickness of concrete and would have affected dead load calculations slightly. Without knowing the concrete thickness exactly a small variation in concrete thickness, in the order of 5mm, may have occurred. The affect of which may be in the order of 0.5 kN (over 1.83 x 1.83m grid) and being so small was not considered in this research.

Sensitivity Analysis

It was noted from a sensitivity analysis that the wet density of concrete can affect results. Hence, it must be accurately measured in order to determine the theoretical loads. Typically a value of 2500 kg/m$^3$ is used in design as specified in AS 3610 Supplement 2 (Standards Australia, 1996), which includes an allowance for reinforcement. Most literature points to using a value of 2450 kg/m$^3$, however for the purposes of this investigation the actual measured density was used in calculation. This was calculated from an average across all trucks that contributed to the total pour. For example, in site 1, investigation 1, the average density was 2297.5 kg/m$^3$ determined from the eight trucks which contributed to the concrete pour as seen in Table 9.

<table>
<thead>
<tr>
<th>Pour # 1</th>
<th>DateCast</th>
<th>SourceName</th>
<th>Docket</th>
<th>LocationDescription</th>
<th>ProductCode</th>
<th>Density_28</th>
<th>Avg Density_28</th>
</tr>
</thead>
<tbody>
<tr>
<td>21/06/11</td>
<td>Granville</td>
<td>26377376</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2280</td>
<td>2280</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Granville</td>
<td>26377381</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2240</td>
<td>2240</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Granville</td>
<td>26377393</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2280</td>
<td>2280</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Granville</td>
<td>26377401</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2320</td>
<td>2320</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Granville</td>
<td>26377412</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2320</td>
<td>2320</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Smithfield</td>
<td>26710415</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2320</td>
<td>2320</td>
<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Smithfield</td>
<td>26710419</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2310</td>
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<td></td>
</tr>
<tr>
<td>21/06/11</td>
<td>Smithfield</td>
<td>26710427</td>
<td>LEVEL 1 POUR 4</td>
<td>PT40-20-22@3-90</td>
<td>2310</td>
<td>2310</td>
<td></td>
</tr>
</tbody>
</table>

Table 9: Site 1 Investigation 1, documentation of wet density of concrete from each truck

A sensitivity analysis was conducted to understand what the affect of varying the concrete thickness, concrete density, formwork weight and reo weight, had on the mean to nominal dead load ratio ($\overline{D}/D_n$).

Although the weight of formwork and reinforcement was calculated individually across all ten site investigations, it was clear and evident from a sensitivity analysis into the factors affecting load calculations
(see Figure 25, Figure 26, Figure 27, Figure 28) that the weight of formwork and reinforcement had little effect on the final result. This is intuitive as each component makes up less than 10% of the weight of concrete.

Figure 25: Sensitivity Analysis: Concrete Thickness
Figure 26: Sensitivity Analysis: Reo Weight
Figure 27: Sensitivity Analysis: Concrete Density
Figure 28: Sensitivity Analysis: Formwork Weight
DISCUSSION

The shore load investigation and data collection process was highly successful. The results are extremely valuable for a number of reasons. Firstly, the ten site investigations took over two years to complete with and in total 188 shores were instrumented on full scale commercial construction sites. Secondly, a uniform process of data collection was maintained throughout all ten site investigations, leading to the validity and accuracy of results.

The Relative Dead Load was calculated as the ratio of mean to nominal dead load. Actual dead load was recorded using Load Cell data at the point in time after pouring had ceased and all workmen and equipment were removed from the slab. The theoretical dead load was calculated by a tributary area analysis and included the weight of wet concrete, steel reinforcement and timber formwork. The Relative Dead Load is also the same as the mean to nominal load ratio \( \frac{\bar{D}}{D_n} \).

Furthermore, the Relative Live Load is the ratio of mean to nominal live load and during pouring includes the weight of workmen and equipment as well as the temporary mounding of concrete. The Relative Dead Load is also the same as the mean to nominal load ratio \( \frac{\bar{L}}{L_n} \).

Although shores were highly loaded and in some cases had double the recorded load of the shore directly beside them, the maximum leg load appeared to be within half of the ultimate capacity of the shore load. This result indicates that although highly variable, the design engineer from Acrow Formwork and Scaffolding Pty Ltd appears to be applying a safety factor of around 2 to most scaffolding designs.

The live load results were determined as the difference between the dead load and the peak maximum load observed after concrete pouring had occurred. Peak live loads were observed either during the pour process (Figure 2-28); when the gang of concrete workers was pouring the area, or during the process of “power trowelling”, when one or two workers were working the surface of the wet concrete with a power trowel machine. These peaks were observed either during the pour process or 4-6hrs after the pour had occurred, respectively. Interestingly in all tests, except S2I1, the theoretical live loads were greater than the actual or measured live loads. i.e a mean to nominal ratio less than 1.0. Even though no factor of safety was used in either actual or theoretical calculations, it is clear that the actual live load results are quite conservative compared to the 1kPa theoretical live load required in AS3610-1995.

On the basis of the survey results, it appears that the actual shore loads, on average, give good agreement with the predicted values using the tributary area method. The results of the investigation have been analyzed by engineers at Acrow Formwork and Scaffolding who have over 25 years in the industry. Their analysis notes the similarity between actual and theoretical values is “within expected limits”. The full set of dead and live load statistics can be seen in Table 10.

<table>
<thead>
<tr>
<th>Site Investigation</th>
<th>DEAD LOAD ( \frac{\bar{D}}{D_n} )</th>
<th>LIVE LOAD ( \frac{\bar{L}}{L_n} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AVG</td>
<td>STD DEV</td>
</tr>
<tr>
<td>S1P1</td>
<td>0.720</td>
<td>0.149</td>
</tr>
<tr>
<td>S1P2</td>
<td>1.038</td>
<td>0.341</td>
</tr>
<tr>
<td>S1P3</td>
<td>1.181</td>
<td>0.351</td>
</tr>
<tr>
<td>S1P4</td>
<td>0.986</td>
<td>0.231</td>
</tr>
<tr>
<td>S2P1</td>
<td>1.008</td>
<td>0.307</td>
</tr>
<tr>
<td>S2P2</td>
<td>1.003</td>
<td>0.254</td>
</tr>
<tr>
<td>S3P1</td>
<td>1.036</td>
<td>0.291</td>
</tr>
<tr>
<td>S3P2</td>
<td>1.039</td>
<td>0.404</td>
</tr>
<tr>
<td>S3P3</td>
<td>1.044</td>
<td>0.336</td>
</tr>
<tr>
<td>S3P4</td>
<td>1.447</td>
<td>0.679</td>
</tr>
<tr>
<td>AVG</td>
<td>1.050</td>
<td>0.334</td>
</tr>
</tbody>
</table>

Table 10: All Results for the Ten Site Investigations- Relative Dead and Live Load’s
Figure 29: Mean to Nominal Dead and Live Load Ratio

Figure 30: Mean to Nominal Dead and Live Load COV

Figure 29 clearly describes the mean to nominal dead and live load ratios for all ten site investigations. The dead load results demonstrate that relative dead load ratios are independent of test sites, i.e., there is no one trend occurring at a particular site, which confirms the accuracy of the data. Furthermore, Figure 30 describes the mean to nominal dead and live load COV’s for all ten site investigations.

The results of the investigation contribute to the noted lack of loading statistical data, and assist in developing a probabilistic framework for support scaffolding systems. An average mean to nominal dead load ratio of 1.05 (see Table 11), confirms and correlates with previous research in the dead load fields. However, importantly for this research, being shore load specific, the dead load COV for individuals shores is known now to be much higher. This is due to a number of factors, but the predominant cause was jack height discrepancies from poor erection practices. The authors of this thesis would argue that the dead load COV would be more accurate as 0.3 rather than the old 0.1 value, as seen in Table 13.

Furthermore, the live load statistics for this investigation are extremely valuable, there is a significant lack of data in the literature regarding real construction live load statistics. Previous studies suggested a Type I extreme distribution with a mean to nominal ratio of \( \frac{L}{L_n} = 0.9 \) and a COV of 0.7 based on a design live load of 1.0 kPa for the phase of concrete placement. It is evident that the statistical results of this research...
report actually suggest lower values. In fact, the relative live load ratio \( \bar{L}/L_n \) was determined to be 0.821 with a lognormal distribution and a COV = 0.404, as seen in Table 11.

It is worth emphasizing that \( \bar{L}/L_n \) depends on the design formwork live load, and that the value may vary considerably from standard to standard. For example, the American Concrete Institute specifies a design live load of 2.4 kPa (50 psf) for all construction stages [30], while in the Australian Standard for formwork for concrete AS3610 [31], the design live load varies for different construction stages; it is 1.0 kPa for the phase of concrete placement. These results could be compared and contrasted to the requirements of other international codes for future investigations, however this is outside the scope of this research.

<table>
<thead>
<tr>
<th>Load</th>
<th>Mean-to-nominal value ( \bar{x}/x_n )</th>
<th>Coefficient of variation COV</th>
<th>Probability distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead</td>
<td>OLD = 1.05</td>
<td>NEW = 1.05</td>
<td>OLD = 0.1</td>
</tr>
<tr>
<td>Live</td>
<td>OLD = 0.821</td>
<td>NEW = 0.6</td>
<td>0.404</td>
</tr>
</tbody>
</table>

Table 11: Previous vs Suggested Dead and Live load Statistics

**Nominal Live to Dead Ratio**

The most critical stage in loading scaffolding is during concrete placement for scaffolding systems. It was proven that 74% of scaffolding collapses occur during concrete placement (Hadipriono, 1987). Since the weight of concrete is known to be the largest load on the scaffolding system, the live-to-dead ratio at this time will be dominated by the dead load of concrete. During this phase, the live load component is made up of the weight of workmen and equipment, however these loads are proportionally insignificant in comparison to the dead load.

It has been assumed by particular authors that the nominal live-to-dead \( (L_n/D_n) \) ratio for reinforced concrete structures varies from 0.5 to 1.5 (B. Ellingwood, 1980a). Zhang, (2011) considers a nominal live-to-dead \( (L_n/D_n) \) ratio between 0.3 and 0.7 and uses a representative value for \( L_n/D_n = 0.5 \) for reliability index calculations.

During all ten site investigations, the live-to-dead load ratio was accurately calculated. It is clearly evident from Table 12, that the average nominal live-to-dead \( (L_n/D_n) \) ratio from ten site investigations is 0.178 or approximately 0.2. This is the most appropriate ratio to consider during the most critical stage of loading, i.e. concrete pouring. This result is extremely beneficial for the academic community and will be utilised in a probabilistic study of the reliability of scaffolding systems.

<table>
<thead>
<tr>
<th>( (L_n/D_n) )</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>S1P1</td>
<td>0.167</td>
</tr>
<tr>
<td>S1P2</td>
<td>0.144</td>
</tr>
<tr>
<td>S1P3</td>
<td>0.199</td>
</tr>
<tr>
<td>S1P4</td>
<td>0.139</td>
</tr>
<tr>
<td>S2P1</td>
<td>0.196</td>
</tr>
<tr>
<td>S2P2</td>
<td>0.237</td>
</tr>
<tr>
<td>S3P1</td>
<td>0.180</td>
</tr>
<tr>
<td>S3P2</td>
<td>0.166</td>
</tr>
<tr>
<td>S3P3</td>
<td>0.182</td>
</tr>
<tr>
<td>S3P4</td>
<td>0.170</td>
</tr>
<tr>
<td><strong>AVG</strong></td>
<td><strong>0.178</strong></td>
</tr>
</tbody>
</table>

Table 12 Nominal Live-to-dead Load ratio for all site investigations
CONCLUSION

This full scale shore load survey of construction live and dead loads occurring on an actual construction site in Sydney, gives an adequate baseline for comparison to nominal or theoretical loads. This paper explicitly details the types of scaffolding structures being analysed, the causes of past scaffolding failures and past research into shore loading. The majority of this report is dedicated to describing the equipment and experimental procedure used to acquire load data on construction sites. In total ten pour areas were surveyed during both the concrete casting and curing phases to obtain the mean or actual loads transmitted to the supporting scaffolds. The contribution to the academic community and validity of the research is quite significant due to the complexity, cost and time required to measure and gather such load data.

The results of the investigation have been conclusive and show good correlation between mean (actual) and nominal (theoretical) dead and live load statistics across the entire pour i.e. the mean of all load cells. However, the results bring new light to the significantly high variances in standard deviation and co-efficient of variation, seen in individual shore loads. This result identifies the potential need for a higher safety factor or a revised construction sequence which minimises variances between shore loads. The results are deemed to be founded on a large enough data set including site investigations on 3 separate construction sites and over 10 measured pour areas. The results highlight the variance in individual shore load and require a revision of existing statistical data as seen in Table 13.

<table>
<thead>
<tr>
<th>Load</th>
<th>Mean-to-nominal value $\bar{x}/x_n$</th>
<th>Coefficient of variation COV</th>
<th>Probability distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead</td>
<td>1.05</td>
<td>0.3</td>
<td>Normal</td>
</tr>
<tr>
<td>Live</td>
<td>0.85</td>
<td>0.4</td>
<td>Type I extreme value</td>
</tr>
</tbody>
</table>

Table 13 Adopted Dead and Live load Statistics

Additionally, a newfound understanding into the causes of variability in shore loads will hopefully initiate safer and more reliable construction techniques in the erection of scaffolding. A summary of construction techniques leading to varying shore loads is presented below. Critically this investigation contributes to the distinct lack of data available with respect to evolution and magnitude of construction loads, particularly construction live loads. This new statistical information will be used to build a more reliable and accurate finite element model for advanced analysis of the scaffolding structures, leading to a probabilistic based design code rather than the current code, which is based on engineering judgement and empirical evidence.

The results have contributed to a more clarified understanding of actual loads occurring in formwork supporting systems. The results will be used to provide guidelines and a framework for more reliable and safer formwork design. In particular to investigate the adequacy of the current scaffolding design practice (e.g., the factor of safety, the load combination rules).
AREA FOR FURTHER INVESTIGATION
The research conducted in this report, has lead to the need for further research into areas which are deemed to be incomplete, including:

- Understanding why and how much effect, jack height discrepancy (i.e. looseness of some U-heads) causes and if this phenomenon occurs across all formwork erection subcontractors.
- Measurement of the actual thickness of the slab to determine the accuracy of the pours by a subcontractors. This would obviously effect the nominal (theoretical) calculations.
- The rationalisation of concrete mounding effects where falls in top level car parks were required.
- Effects of settlement in shores and determining what effect re-shoring has on relative shore loads.
- From the experimental data, a distinct drop off in load for cells adjacent to the columns could be seen (even when columns were poured at the same time as the slab). It was assumed that this was possibly due to differential temperature dependant elastic shortening affects between wide concrete columns and narrow steel scaffolding tubes.

ACKNOWLEDGEMENTS
The authors would like to thank:

- Acrow Formwork and Scaffolding Pty Ltd, for their financial support and for allowing us to investigate their ‘Supercuplok’ scaffolding system, a product used widely in Australia for support scaffolding.
- Rediform Pty Ltd, for allowing us access to the scaffolding system and their on-site support.
- Brookfield Multiplex Pty Ltd and Stocklands for allowing access to their construction sites.
- Sydney Airports Corporation Limited (SACL) for allowing access to the airport for site experimentation.
- The staff at the Centre for Advanced Structural Engineering (CASE) at the University of Sydney for their support during the project.
APPENDIX A: FORMWORK CONSTRUCTION SEQUENCE

During the on site investigations it was noted that the formwork construction sequence was a definite contributor to the proportion of load, which was attracted to a particular vertical shore. It is a result of the on-site investigations that formwork construction sequence was deemed to directly and significantly affect loads in standards and hence the loading results attained from site investigations. The load results were affected by the arrangement of timber formwork, erection techniques and erection sequence, all of which will be discussed in the following section.

Process of formwork erection

The following depicts the formwork erection process, as observed on site during the 2 years of site investigations.

a. The main vertical and horizontal members of the steel support scaffolding system is erected by scaffolding team.

b. U-heads are dropped into the tubular steel shores and are roughly adjusted to predicted heights. Typically however, vertical RL of U-heads varies significantly (in the order of 200mm). Interestingly, these are not adjusted by scaffolding teams, (this is suggested for future erection sequences as an opportune time to adjust U-head heights as a preliminary safety measure).

Figure 31 - Leveling of Timber Bearer
c. A new team of Formworkers is then tasked with erection of the entire timber formwork system. A single bearer is placed over up to 4 U-heads and its associated vertical standard. The formworker then uses a basic angle device with laser level to adjust the height of the two outermost U-heads as seen in Figure 32. This is the point where the internal U-heads may not be correctly adjusted upward and may maintain a gap between the timber bearer above (Figure 35). Conversely, the opposite can also occur, where these U-heads may be over tightened and lead to a gap forming at the exterior U-head. These bearers form the basic set out for the concrete beams and run perpendicular to the direction of a traditionally formed beam. As seen in Figure 33.

d. A further row of bearers is placed between each line of u-heads perpendicular to the concrete formed beams. As seen centrally in Figure 33.

e. Depending on whether a traditionally formed deck (Figure 34) or a Bondek steel decking (Figure 33) is used, it will slightly alter the sequence; however essentially: timber joists or metal decking is placed perpendicular to the bearers in between the traditionally formed concrete beams of a one-way slab.

Figure 32 Bearer levelling technique

Figure 33 Traditionally formed beams supported by bearers and Slabs formed from metal decking (BONDEK)
Separation between bearers and U-head

Loose u-heads cause load to be redirected to alternate U-heads and hence larger COV in Load statistics. The support points for a timber bearer are obviously the U-heads. However if a U-head is loose the timber bearer will span over it, take load and deflect until the point in time where it meets the U-head support. i.e a 3.6m length of timber bearer will be supported by three or four u-heads even at maximum bay size of 1.83m. (seen in Figure 33).

From on-site observation, the formation of a gap between bearer and U-head readily occurred. More concerning was that once formwork is laid, there is no check by formworkers either in the steel erection team or timber formwork team to ensure that all U-heads are erected to the same level and that there are no gaps between bearer and U-heads.

The formation of a gap between u-head and bearer occurs as a result of imperfect levelling of u-heads, allowing one u-head to become redundant until enough load is applied to deform the bearer and form a new load path into the u-head. The gap has an effect on the distribution of load between shores, and importantly can cause overloading of some shores and redundancy of other shores. As can be seen in Figure 35, bearers are typically aligned and levelled on the outermost U-heads. i.e. U-head 1 and 4.
Figure 35 Alignment of Bearers under two future concrete beams

Figure 36: Example of a Formwork team adjusting timber Bearers (note gap between interior U-head and bearer)

Note formwork layout around column location – 2 bearers either side of column.
Effect of Gap between U-head and Bearer
The formation of this gap has been postulated to cause the variance in dead and live load in U-heads positioned adjacent to each other and under the same bearer.

In the investigations conducted as part of this research, there could have been multiple combinations of height variances and overlapping of bearers. Therefore, it is difficult to actually track load paths and determine the full extent of the problem.

More prevalent was load cells situated under beams where the exact configuration of 4 U-heads and the associated load, was known. It was determined that where this occurred the outside U-heads typically were subjected to more load and hence verified the assumption that construction sequence does play a significant part in the COV of load.

Site 3 – Pour 1 Merrylands Stage 4

Figure 37: Gap between U-head and Bearer

Site 2 – Pour 1 Sydney International Airport
Effect of Point Loads on a Continuous Beam

It has been suggested that the load transferred from each joist to the bearer below, can be simplified as a uniformly distributed load (UDL), Hurd (1989). Hurd (1989), asserted that stresses and strains should be checked by a more accurate calculation when the gap between joists / point loads was more than one third of the span. As was the case in the majority of the site's investigated as a part of this thesis, the distance between point loads was generally less than one-third of the span of the bearer.

In a study undertaken to determine if the point loads from joists could be simplified into a uniform distributed load (UDL), (J. Ikaheimonen, 1997) compared six beams ranging from 1 span to 6 spans acted upon by point loads versus beams acted upon by a UDL. Moments and deformations of these beams were confirmed through calculations on a rigidly fixed cantilever. The results showed that, in calculating shore loads, point loads can be replaced by a uniformly distributed load, (J. Ikaheimonen, 1997)

Furthermore the study by (J. Ikaheimonen, 1997) utilised both the Beam method and tributary area method to estimate the loads applied to individual shores. The investigation compared the two methods across nine different sites. Following which he made the conclusion that the two methods were largely comparable. The Beam method did give a slightly better mean value of relative shore load. As such, the Tributary area method has been utilised in the above investigation.
REFERENCES

Aus.: John Wiley & Sons.


APPENDIX B: PARAMETRIC STUDY ON LIVE LOAD STATISTICS

A simple dead and live load combination was considered with load and resistance statistics been drawn from (B. R. Ellingwood, Galambos, Macgregor, & Cornell, 1980) and (D. V. Rosowsky & Stewart, 2001). The results of this parametric investigation can be graphically illustrated in and the full analysis and results can be viewed in XX. Results indicated that increasing COV in the live load has the effect of lowering the overall β-γ curve dramatically. Similarly, the β-γ curve was also sensitive to small changes in mean to nominal design load. Accurate estimates of the mean to nominal live load value are therefore required if specific target reliabilities are to be achieved.

In this appendix, different live load parameters will be investigated to determine their effects on the β-γ curve (target system reliability and the live load factor). In doing so, the effect of certain live load statistics on the reliability of the system, as well as the choice of load factor, shall be examined. During this process, a simple load combination of dead load + live load on a scaffolding system will be considered with a resistance factor of 0.8 (φ=0.8). Although previous researchers in this field (D. V. Rosowsky, 1996) have used simple normal distributions of load and resistance to accomplish this task, in this study live loads shall assumed to be Type 1-extreme distributions and resistance shall be a lognormal distribution, which is in-line with steel scaffolding systems. Once the parametric distributions have been normalised, FOSM reliability analysis will be used to determine the relative effects of these load parameters on reliability, providing insight into the most important statistics for live load modelling. Furthermore, in accordance with Australian steel scaffolding standards AS3610 (Standards Australia, 1995), the following checking equation will be considered,

\[ \phi R_n \geq 1.2D_n + 1.6L_n \]

Along with the corresponding limit state reliability function:

\[ g(x) = 0.8R_n - (1.2D_n + 1.6L_n) \]

Consequently, as the limit state function exists in simple load and resistance form, the reliability index (β) of the system for the FOSM method is therefore;

\[ \beta = \frac{R_n \phi - \frac{D_n}{D_n} - \frac{L_n}{L_n} \frac{L_n}{L_n} - D_n \phi - L_n \phi}{\sqrt{(\sigma_R^2 R_n \phi)^2 + \sigma_D^2 D_n + \sigma_L^2 L_n}} \]

For this investigation, the same statistics derived in the previous chapters will be used. This means that the mean to nominal dead load and COV are taken as 1.05 and .11 respectively (B. R. Ellingwood et al., 1980). The live load will be assumed to be a type 1 extreme distribution with a mean to nominal ratio of 0.9 and a COV of 0.6. Finally, resistance statistics derived from probabilistic modelling (H. Zhang, Chandrangsu, & Rasmussen, 2006) of steel scaffolding systems with a 300mm jack extension will be used in this example. The mean to nominal resistance value of 1.08 and COV of .11 is therefore imputed into the model. Results can be seen in Figure 1-1.
Figure 1-1 Effect of varying (a) COV in LL (b) Mean to Nominal Ratio for Live Load (c) Ln/Dn Ratio
The results of this study clearly illustrate the trends that occur when different live load parameters are altered. From Figure 1-1(a) it can be seen that increasing the COV has the effect of lowering the overall $\beta$-$\gamma$ curve, meaning a decrease in beta and an increase in the probability of failure for the system. Similarly, Figure 1-1 (b) also suggests that an increase in mean to nominal live load leads to a downward parallel shift of the $\beta$-$\gamma$ curve. However, Figure 1-1(c) shows that an increasing live to dead ratio means that there is an increase in beta for the system with a slight rotation to the $\beta$-$\gamma$ curve as well. This means that as the live load becomes the more dominant load force in the equation considered, the reliability of the system increases and furthermore, becomes more sensitive to changes in the load factor.

For the purposes of live load modelling then, the following conclusions and observations can be drawn:

+ Accurate estimations of the COV of and the mean to nominal ratio of live loads are required as the reliability of the system is greatly affected by small changes in these parameters.
+ The more dominant the live load in the load combination of the system, the more important the COV in this load. Subsequently, large changes in reliability can be achieved with only a minor alteration to the live load factor.
+ A final note should also be made that with an increase of dead to live ratio, there is an increase in the possibility of failure and a subsequent decrease in failure.

Understanding these illustrative examples provides the necessary information required to make more accurate engineering judgements regarding the live load parameters. Although statistics specific to steel scaffolding systems have been used in this example, the general trends can be extended without any loss of simplification.
APPENDIX 2a: (J. Ikaheimonen, 1997) Relative shore load results

<table>
<thead>
<tr>
<th>Author/s</th>
<th>Year</th>
<th>Literary Review of Formwork and Loading Publications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nielsen (1952)</td>
<td></td>
<td>Investigated load distribution throughout slabs as a result of formwork systems. Nielsen measured shore loads on various sites, however after the concrete had been poured. Developed theoretical models for load distribution between slabs and the standards. Nielsen also investigated and measured shore loads in scale models of formwork, varying the moisture content in the formwork.</td>
</tr>
<tr>
<td>Backsell and Hammarlund (1959)</td>
<td></td>
<td>Developed initial formwork design calculations for traditional slab formwork.</td>
</tr>
<tr>
<td>Grundy and Kabaila (1963)</td>
<td></td>
<td>Published new design calculations for loads on standards and slabs.</td>
</tr>
<tr>
<td>Backsell et al (1966)</td>
<td></td>
<td>Published design charts for formwork, based on earlier research.</td>
</tr>
<tr>
<td>Marosszeky (1972)</td>
<td></td>
<td>Explained a Swedish construction technique whereby the lower level slab carries its own weight prior to the pouring of the slab above it. This technique ensures that loads on shores (below the level being shored) will never be higher than the loads on the shores above.</td>
</tr>
<tr>
<td>Agarwal and Gardner (1974)</td>
<td></td>
<td>Measured on site loads in shores after pouring had occurred. Obtained good agreement with the simplified method developed by Grundy and Kabaila (1963). Their research focused on the distribution of load between shores and slabs in multistorey buildings.</td>
</tr>
</tbody>
</table>
Richardson (1997)  Determined that there can be up to 40mm eccentricity and up to 1° inclination from the vertical, in loaded shores. Further, determining that these shores have 60% of the loadbearing capacity of vertical, non-eccentrically loaded shores.

Mohammed and Simon (1979)  Utilised and altered the Grundy & Kabaila Simplified method to include a live load of 2.4 kPa during the pour. To confirm the size of the live load, they also measured live loads in four shores during a site pour.

Gardner (1979)  Concluded that the load on slabs increases with the number of storeys which are shored. Recommended that one storey of shores should be used.


Fattal (1983)  Was the first researcher to measure loads in standards whilst concrete was being poured. Importantly he failed to measure the initial load due to timber formwork in the shores. Assuming that the large variations in relative shore loads occurred due to differential levels of jacking in standards.

Sbarounis (1984)  Furthered the Grundy & Kabaila method to include slab stiffness reductions as a result of cracking.

Gardner (1985)  Investigated and developed a new methodology for calculating the load in shored multistorey buildings.

McAdam (1985)  Investigated how creep in concrete and the stiffness in shores, effects slabs in multistory structures.

Lew (1985)  Postulated that; due to the distinct lack of statistical data relating to loads on formwork during construction (i.e. what this thesis is investigating), the partial factors method could not be validated.

Liu and Chen (1985)  Developed a three dimensional computer model to calculate maximum shore loads and loads in slabs for multistory buildings.

Liu, Chen and Bowman (1985a)  Through an analysis of the Grundy Kabaila method, they proposed a correction factor of 1.05-1.10 should be utilized.

Liu, Chen and Bowman (1985b)  Developed a revised method for calculating loads in shores.

Hurd and Courtois (1986)  Developed a revised computer model to calculate shore loads in multistory construction.

Gardner and Chan (1986)  Results of an analysis into shoring techniques showed that shore loads are higher, but permanent deformation in slabs is reduced, by using progressive reshoring.

Gross and Lew (1986)  Developed a revised computer model to calculate shore loads in multistory construction.

Liu, Chen and Bowman (1986)  Proposed a correction factor to the ‘simplified method’ in the order of 1.05 – 1.10. Further investigated the failure of shores and its effect on the system.

Aguinaga-Zapanta and Bazant (1986)  Similar to McAdam (1985), they devised a method to calculate shore loads where the effects of creep were included. Determined that creep has only minor effects on shore load.
<table>
<thead>
<tr>
<th>Author(s)</th>
<th>Study Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liu and Chen (1987a)</td>
<td>Used the ‘refined 3D’ model to determine that creep in slabs had little effect on loads in shores. The loads on slabs and shores are reduced by approximately 5-10%.</td>
</tr>
<tr>
<td>Liu and Chen (1987b)</td>
<td>Furthered research from (1987a) utilizing their 3D model to investigate the change in maximum shore load by varying the elastic modulus, curvature, inclination, load eccentricity etc of the shore.</td>
</tr>
<tr>
<td>Liu, Lee and Chen (1988)</td>
<td>Used the Grundy &amp; Kabaila ‘simplified method’, to create a computer program calculating loads in slabs and shores.</td>
</tr>
<tr>
<td>Liu, Lee and Chen (1989)</td>
<td>Developed simplified rules for optimizing the number of shores and reshores depending on slab strength.</td>
</tr>
<tr>
<td>Karshenas and Ayoub (1989)</td>
<td>Calculated the load effect due to stacked materials on newly poured slabs. Investigated 20 full-scale structures to make measurements.</td>
</tr>
<tr>
<td>Gardner and Muscuti (1989)</td>
<td>Used the Grundy &amp; Kabaila ‘simplified method’, to create a computer program calculating loads in slabs and shores in multistorey buildings. The program was also able to check the strength development of slabs.</td>
</tr>
<tr>
<td>Stivaros and Halvorsen (1990, 1991)</td>
<td>Formulated a more accurate method for calculating load distribution in shores which generated better agreement than previous models with real structures.</td>
</tr>
<tr>
<td>Gardner (1990)</td>
<td>Wrote that long term deformations in the finished slab are dependent on the load they were subjected to during construction, when these cracked due to high loads and low strength.</td>
</tr>
<tr>
<td>Mosallam and Chen (1991)</td>
<td>Compared the two-dimensional computer model with the Grundy-Kabaila method. In the two-dimensional model they also take into consideration that change in load distribution when a new slab is poured. They showed that the Grundy-Kabaila method gives excessive loads for slabs and shores, due mainly to the assumption that shores are infinitely rigid, and because no account is taken of the change in load distribution when a new slab is poured.</td>
</tr>
<tr>
<td>Lee, Liu and Chen (1991)</td>
<td>Presented a method for the calculation of loads on slabs and shores in multistorey buildings in which the calculations allow for creep and other time dependent effects in the materials.</td>
</tr>
<tr>
<td>Lee, Chen and Liu (1992)</td>
<td>Studied the loads due to motorised concrete dumpers on formwork using calculation models. They found that a load of 4.06 kN/m² is more realistic than the present 3.58 kN/m².</td>
</tr>
<tr>
<td>Reference</td>
<td>Description</td>
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<tr>
<td>Stivaros and Halvorsen (1992)</td>
<td>Compared the equivalent frame method EFM with the simplified Grundy-Kabaila method. They show with an example calculation where the reshores are of timber that the simplified method misjudges the load effect on floor slabs which are seven days old and thus leads to unsafe assumptions.</td>
</tr>
<tr>
<td>Ashraf, El-Shahhat and Chen (1992)</td>
<td>Developed an ‘improved analysis’ for shored multistorey buildings which, in contrast to the ‘refined’ analysis, includes the accumulated deformations due to the loads from previous poured slabs.</td>
</tr>
<tr>
<td>Peng, Rosowsky, Pan and Chen (1993)</td>
<td>Developed a structural model and a loading model for a tower during the construction stage, and a probabilistic design procedure for this. Peng (26) studied double layer timber scaffolding systems, particularly with respect to how the load carrying capacity was affected by the length of horizontal timber stringers, vertical shores, the stiffness of stringers and the position of strong shores. It was determined that the load carrying capacity of a system could be increased by adding strong shores (vertical shores with horizontal bracing).</td>
</tr>
<tr>
<td>Ayoub and Karshenas (1994)</td>
<td>Investigated loads on formwork before concreting. They made a thorough survey of 22 structures where they weighed and surveyed all loads on the formwork and then calculated an equivalent uniformly distributed load.</td>
</tr>
<tr>
<td>Karshenas and Ayoub (1994a)</td>
<td>Developed a probabilistic model for live loads on a newly poured concrete slab.</td>
</tr>
<tr>
<td>Karshenas and Ayoub (1994b)</td>
<td>Studied loads on formwork before concreting. They made a thorough survey of 22 structures where they weighed and surveyed all loads on the formwork and then calculated an equivalent uniformly distributed load.</td>
</tr>
<tr>
<td>Karshenas and Heinrich (1994)</td>
<td>Developed a theoretical 'single-degree of freedom, variable mass, dynamic model' to calculate the dynamic load due to skip discharges on a traditional timber formwork for a slab. They also carried out laboratory tests in which concrete was discharged from a skip at different heights above a model of a traditional timber formwork.</td>
</tr>
<tr>
<td>Rosowsky, Huang, Chen and Yen (1994)</td>
<td>Studied what effect the advance of the pouring front had on shore loads, i.e. the way concrete was progressively placed on the formwork. They carried out field studies in 19 buildings in Taiwan. One storey per building was analysed by recording the work on video. They could distinguish between 5 pouring routes when the slabs were poured. They also built a full scale model of a formwork. In order to simulate concreting, they placed containers with water on the different pouring routes to study the effect on shore loads.</td>
</tr>
<tr>
<td>Duan and Chen (1995a)</td>
<td>Studied the effect of creep on shore loads.</td>
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<tr>
<td>Author(s)</td>
<td>Description</td>
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<tr>
<td>Peng, Rosowsky, Pan, Chen, Chan and Yen (1996)</td>
<td>Presented a ‘simplified analysis procedure’. The method is based on influence areas for the calculation of shore loads, and yields values of shore loads which differ by no more than 5% from the labour intensive and complex 3-D model. The method does not take account of discontinuities in the formwork or differences in ‘prestressing’ between shores prior to concreting.</td>
</tr>
<tr>
<td>Peng, Yen, Lin, Wu and Chen (1996)</td>
<td>Carried out broadly the same tests as Rosowsky, Huang et al (1994), but they used sandbags instead of water, and the different pouring routes had little influence on the magnitude of shore loads.</td>
</tr>
<tr>
<td>Kamala, Dickens and Pallet (1996)</td>
<td>Measured loads on shores in two structures. In the first structure the loads were measured on nine shores and the loads were in good agreement with those calculated, except for two of the shores where the relative shores loads were 0.88 and 1.28. The authors said that uneven prestressing was a possible reason for this. At the other structure the shore loads were not reported, but the authors stated that the variations could be attributed to uneven prestressing.</td>
</tr>
<tr>
<td>Duan and Chen (1996)</td>
<td>Gave guidelines for the design of safe formwork.</td>
</tr>
<tr>
<td>Catala Moragues and Pellicer (1996)</td>
<td>Made a FEM analysis of shore loads in two multistorey buildings. These loads were compared with loads measured on shores and reshores. Their conclusions were that the ‘simplified’ method can usually be employed in designing shore loads in multistorey buildings.</td>
</tr>
<tr>
<td>Philbrick Rosowsky and Huston (1997)</td>
<td>Analysed shore loads measured in three structures. Continuous measurements were made (sampling frequency 100 Hz). There was relatively large scatter in the relative shore loads they obtained.</td>
</tr>
<tr>
<td>Zhang</td>
<td>In a paper produced at the University of Sydney (R905), there were a range of other factors identified which were deemed to affect the load carrying capacity of the scaffolding system. These include the bracing arrangement, ground irregularities and load eccentricity.</td>
</tr>
<tr>
<td>Yu et al (10)</td>
<td>An investigation into the resistance or load carrying capacity of scaffolding was performed by Yu et al (10) , to investigate how boundary conditions and the number of storeys affected the aforementioned. Scaffolds of up to three storey’s were analyzed and it was determined that the load carrying capacity of two and three-storey scaffolds was only 85% and 80% of a single storey scaffold, respectively, due to the large variation in buckling behaviour. Furthermore, the load carrying capacity varies between 50%-120% in experimental tests where the top and bottom boundary conditions were altered.</td>
</tr>
</tbody>
</table>
Figure 2-1: DL and LL ratio's for Investigated area
Figure 2-2: DL and LL ratio's for Investigated area
Figure 2-3: DL and LL ratio's for Investigated area
Figure 2-4: DL and LL ratio's for Investigated area
Figure 2-5: DL and LL ratio's for Investigated area
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Figure 2.15: DL and LL ratio's for Investigated area
Figure 2-16: DL and LL ratio's for Investigated area
Figure 2-17: DL and LL ratio's for Investigated area
Figure 2-18: DL and LL ratio's for Investigated area
APPENDIX 3 GRAPHES – SUB 2

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Figure 3-2:
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