Sustainability for Existing Steel Structures: Renovations and Life Extensions of Steel Frame Buildings using SPS Panels

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Abstract

Carnegie Hall in New York and the Alexander Memorial Coliseum (AMC) in Atlanta have recently gained a new lease of life by incorporating SPS panels into their renovation programs. This light prefabricated steel composite structural panel minimized the strengthening required of the existing frame, shortened schedules, reduced programme complexity and helped to deliver a new lease of life to these two historic steel frame structures.

With tightening budgets and increasing momentum to more sustainable structural inventories, owners need to innovate during the renovation and life extension of their existing structures as well as during the creation of more sustainable new structures. SPS® is a composite (sandwich) plate comprised of steel faceplates and an elastomer core that was developed for use in civil, maritime and offshore structures. The plate and core thicknesses are tailored to provide a structural plate with the required strength and stiffness for the given structure. The core properties were engineered to preclude local buckling of the faceplates and to provide sufficient shear stiffness so the faceplates can reach yield in compression, flexure or any combination.

This paper will describe these two recent renovation projects as well as the product development and project engineering that allowed their successful delivery. It will provide details of the key testing and approvals involved followed by the limit states performance assessments involved for each project. Design guidelines developed for similar future projects will also be presented.

Introduction

Many structural engineers searching for innovative methods to renovate existing steel structures are typically faced with a mix of:

- Constrained construction sites and build schedules
- Inability to increase loads on existing frames and foundations
- Increased safety risks from working within old structures
- Significant changes to main structures typically incur substantial costs.

SPS Floors and Terraces are light, thin, stiff structural SPS panels and units that are manufactured off-site to high accuracy and quality. They are installed using standard steel working practices and enable easier, faster and less complex fit-out. SPS Floors and Terraces have been used on a wide range of projects over the last 5 years. A list of typical projects is provided in Table 1.

### Table 1 - SPS Projects

<table>
<thead>
<tr>
<th>Projects</th>
<th>Location</th>
<th>Area (ft²)</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ascot Racecourse</td>
<td>Ascot, UK</td>
<td>32,500</td>
<td>Spring 2007</td>
</tr>
<tr>
<td>The O2 Arena</td>
<td>Dublin, Ireland</td>
<td>10,200</td>
<td>Spring 2008</td>
</tr>
<tr>
<td>LG Arena</td>
<td>Birmingham, UK</td>
<td>42,000</td>
<td>Fall 2009</td>
</tr>
<tr>
<td>Grand Pier</td>
<td>Weston-super-Mare, UK</td>
<td>43,055</td>
<td>Winter 2009</td>
</tr>
<tr>
<td>Aquatics Centre</td>
<td>London, UK</td>
<td>66,500</td>
<td>Spring 2011</td>
</tr>
<tr>
<td>Custody Centre</td>
<td>Croydon, UK</td>
<td>44,917</td>
<td>Spring 2011</td>
</tr>
<tr>
<td>Carnegie Hall</td>
<td>Manhattan, NY</td>
<td>9,060</td>
<td>Spring 2011</td>
</tr>
<tr>
<td>Gatwick Airport</td>
<td>London, UK</td>
<td>2,980</td>
<td>Spring 2011</td>
</tr>
<tr>
<td>USTA Stadium “E”</td>
<td>Queens, NY</td>
<td>13,925</td>
<td>Fall 2011</td>
</tr>
<tr>
<td>GaTech Basketball</td>
<td>Atlanta, Ga</td>
<td>12,800</td>
<td>Fall 2011</td>
</tr>
</tbody>
</table>

SPS Floor Development

In 2008, working with the Building Research Establishment [BRE, 2010: Client Report Number 248-766 and BRE, 2010 Test Report Number 250917] (UK) and the University of Sheffield (UK), IE completed a series of large-scale tests on an SPS Floor bay to determine its static and dynamic performance. The tested floor comprised three SPS Floor plates with plan dimensions of 10 x 30 ft that were bolted to the supporting steel beams and girders to form a 30 x 30 ft floor bay, as shown in Figure 1. The steel beams framed into 11’6” long columns to maintain the floor at an 8’ elevation above the laboratory strong-floor and to facilitate instrumentation. The SPS Floor plates were composed of
3/16 inch thick faceplates and a 2 inch thick bubble core. The perimeter frame consisted of 4 inch deep channel sections that were placed web-face-down such that the flanges maintained the faceplates at the desired 2 inch separation for the core. Diagonal cross-braces were used to prevent a global rotational vibration mode of the assembly.

A finite element model was developed using solid brick elements, to predict the behaviour of the floor and as a verification of the modeling technique. Note that the FE model was loaded in the same manner as the test – through 24 discrete points. Indicative load-deflection curves and load-strain curves are given in Figure 3 and Figure 4. From the measured data, there is a distinct change in slope (see Figure 3) indicating a reduction in stiffness beyond the serviceability limit state load. This is attributed to slip in the bolted connection. Slip critical connections are designed to prevent slip until the service loads are exceeded. The predicted deflection and strain values correlate well with the measured values and the softer response beyond the SLS loading is a well understood and expected behavioural response.

The finite element test predictions and measured strains show that the structure at these loads will remain elastic. In fact, the design of SPS Floors is driven by the vibration serviceability requirements and therefore the structural resistance of SPS Floors exceeds the ultimate limit state, meaning the SPS plates will be lightly stressed.
under walking excitations at various pacing frequencies. For these tests, floor and ceiling finishes that are commonly used in UK offices, were added to the specimen to give dynamic response results that are expected from a finished space. These finishes added a dead load equivalent to 20 psf to the bare structure and consisted of: raised access floors, fiberboard fire insulation, suspended metal ceiling, and mechanical and electrical ducting and cabling. The measured results were subsequently used to calibrate finite element models for the calculation of response factors.

The testing was conducted by the University of Sheffield Vibration Engineering Section [Pavic et al, 2008] and Arup’s at the BRE Laboratory and consisted of a two-stage process. Firstly, three electro-dynamic shakers were placed as shown in Figure 5 & 6, and used to vibrate the floor to establish the modal properties of the system (damping, frequencies, and modal mass). For this test, accelerations were measured at 91 grid points, to determine the global mode shapes of the floor. Shaker shut down tests were conducted, with a shaker positioned at the centre of the floor bay to excite the fundamental mode of vibration, to determine the amplitude- and frequency-dependency of the damping. Secondly, walking response tests were conducted, where acceleration responses were recorded due to walking excitations for pace frequencies in the range of 1.5 Hz to 2.4 Hz in increments of 0.2 Hz. For this test, three walking paths were selected as shown in Figure 6. Response factors for each walking path and pace frequency were calculated for subsequent analytical verification.

Finite element models were developed using ANSYS, in accordance with the recommendations of the SCI guide [Smith et al, 2007], with SHELL elements for the SPS plates and BEAM elements for the supporting steel framing. The bracing was modeled with LINK elements. Note that the bracing is not required in typical SPS Floor construction, but was applied to this laboratory-based test specimen to control global horizontal vibration modes.

Analytical and measured modal properties are given in a side-by-side comparison chart in Figure 7 and indicate good correlation for mode shapes, modal masses and natural frequency values. With the same FE-model used to match the modal properties of the system, calculations were conducted in accordance with the SCI guide and the probabilistic approach [Zivanovic et al, 2007], to determine the acceleration response.

In North America, the AISC Design Guide, gives vibration limits for flooring systems in terms of peak accelerations, expressed as a percentage of the acceleration due to gravity, g. For people standing in a shopping mall or on indoor footbridges, the AISC Design Guide suggests an acceptable acceleration due to walking vibrations of about 1.5%g. For office floors the acceptable acceleration is 0.5%g. Vibrations that exceed this limit may make people uneasy and give them the perception that the structure is inadequate even though the structural system may be very lightly stressed. The AISC Guide does not specify a pace frequency for walking excitation.
In accordance with design guidelines and research on determining the dynamic response of floors subject to footfall, to provide acceptable dynamic performance the design should consider the modal properties (frequency, damping and modal amplitudes) of the floor structure. These are established using measured modal properties or those calculated using the finite element method. Two design strategies that take this into account are:

- those presented in The Steel Construction Institute (SCI) and The Concrete Centre (TCC) [Wilford et al, 2006], hereon referred to as Route 1
- the probability-based method [Zivanovic et al, 2007], hereon referred to as Route 2

The vibration limits, used in these calculation methods, are expressed in terms of a response factor (R-factor) that is related to the root mean square value of the acceleration response. The response factor is a multiplier on the level of vibration at the average threshold of human perception (a measure of acceleration in terms of human perception). A response factor of 1 represents the magnitude of vibration that is just perceivable by a typical human. A response factor of 2 is twice that.

For the Route 1 approach, floors are grouped into two categories: high frequency and low frequency floors. A modal analysis of the floor is used to establish its natural frequency. The cut-off between low and high frequency floors, as stated in the Route 1 method, is 10 Hz. In both cases (high or low) the R factor is evaluated from the modal analysis using the mode shape amplitudes for each node in the model. Response factor calculations for low frequency floors (f_1 < 10 Hz) consider the resonant response of the floor. In this case, the acceleration responses to each of the first four harmonics of the walking frequency are calculated and converted to R-factors using a baseline peak acceleration. The total R factor is the ‘square-root-sum-of-squares’ combination of the response factors for each harmonic. Response factor calculations for high frequency floors (f_1 > 10 Hz) consider the impulsive response of the floor. This method calculates the RMS velocity over the period of one footstep and converts this to an R factor using the baseline RMS velocity for the natural frequency of the floor. The maximum footfall rate prescribed in TCC for office bays is 2.0 Hz (footfalls/sec).

Route 2 considers the variability of the pacing excitation and the probability of achieving particular pacing frequencies. Using this method gives a single R-factor value, which is based on 1000 random pedestrians walking at an average pacing frequency of 1.87 Hz along a predetermined walking path. The pacing frequency is defined with a normal distribution such that the incidence of pedestrians walking at elevated pacing frequencies exists, but has a low probability of occurrence. Note that for the probability based calculation method (Route 2), the R-factor will be calculated considering a 25% chance of exceedence, which is implicitly “built-into” the Route 1 methodology.

Measured and predicted response factors are summarized in Figure 8 for the walking path through the middle of the central SPS Floor panel, where peak R-factor values occur. The measured values are indicated on the graph with the green circles. Response factor values were calculated using Route 1 and 2 methods. The measured and calculated fundamental frequency of the floor was 6.6 Hz and the corresponding 3rd and the 4th harmonic of the floor are indicated in the Figure.

![Figure 7 - FEA Predictions and Measured Modal Properties of the SPS Floor](image)

This floor test specimen was designed to serve as a floor in an open plan office. The design criteria for this application would require the R-factor to be less than 8. Because this is an isolated floor bay, the continuity provided by adjacent bays in a real structure does not exist. Therefore, it is anticipated that the R-factor value that is measured or calculated for this laboratory test would be higher than would be experienced on a similar floor within a completed building.

Calculations were performed using the three methods listed above on the test specimen and on an actual floor for a multi-
story office building with the same floor bay dimensions as the test specimen.

From the Route 1 calculations, it can be observed that the peak response value of 19.1 occurs at a pacing of 2.2 Hz. Because the peak occurs at the 3rd harmonic of the fundamental frequency, it suggests that resonant excitation occurs and peak response values are expected. However, this pacing frequency is not common in office environments, and therefore the probability of achieving this R-factor value is unlikely. Therefore, in this case, the predicted R-factor value would be overestimated for this type of application. The pacing frequency in office floors is typically limited to 2.0 Hz for design. As shown in Figure 8, the peak R-factor that occurs within a pace frequency range of 1.5-2.0 Hz is at the 4th harmonic of the floors fundamental frequency (1.65 Hz). According to the Route 1 design calculations, the peak R-factor value for the laboratory test specimen is 14.2.

Route 2 calculations consider the variability of the pacing excitation and the probability of achieving elevated pacing frequencies. R-factor calculations shown in Figure 8 using this method give a single R-factor value, which is based on 2000 random pedestrians walking at an average pacing frequency of 1.87 Hz along the predetermined walking path. The pacing frequency is defined with a normal distribution, such that, walking at the elevated pacing frequency of 2.2 Hz exists, but it has a low probability. Figure 9 shows a comparison of the R-factor, determined in accordance with Route 2 methods using the measured and calculated modal properties. From the curves, considering 25% chance of exceedence, the measured and predicted R-factor values are 12.4 and 11.6, respectively. These R-factor values are very close and suggest excellent correlation.

Figure 10 gives the results of similar Route 1 and 2 computations conducted on a continuous floor plate (actual office building condition) to determine the influence the adjacent floor bays would have on the isolated sample. In accordance with the Route 1, the maximum footfall rate for office bays is 2.0 Hz. At this pace frequency, the calculated R-factor value is R = 4.1. R-factor calculations made using the Route 2 method, give a maximum R-factor value of R = 7.3, which is based on 1000 random pedestrians walking at an average pacing frequency of 1.87 Hz along the predetermined walking path. This indicates that including the adjacent bays provides sufficient continuity that helps to reduce the larger R values to one that would be considered acceptable.
Other SPS Floor Certification Tests

SPS Floors fire resistance performance has been verified by test with designs meeting 1 and 2 hour endurance periods. Tests were conducted in accordance with both the European Standard BS EN 1365-2 and the American Standard ASTM E119 (UL 263) by BRE Global Ltd.

SPS Floors have been tested together with several types of underside protection: stone wool (mineral wool) fire board protection and sprayed protection (wet cementitious). However, all standard fire protection schemes are fully compatible with SPS Floors: gypsum board, sprayed coatings (including dry fibre), wraps and blankets and fire rated suspended ceilings.

SPS Floors sound insulation properties have been verified by tests conducted by BRE Acoustics and SRL Laboratories. Airborne and impact sound insulation properties were determined for SPS Floors with various floor finishes typical of residential and office schemes, demonstrating that SPS Floors meet the sound insulation requirements in the UK Building Regulations. [United Kingdom Building Regulations, 2004 and British Council for Offices Guide, 2005]

Carnegie Hall – 8th Floor Mezzanine Level

Introduction

SPS Floors were used as a lightweight alternative to concrete to create a new mezzanine floor (8M) in Carnegie Hall South Tower and floor additions in the North Tower. The SPS Floors allowed:

- Minimal disruption to the music hall
- Provision of immediate load capacity on installation
- An easy installation in a site restricted area

The project Team comprised Tishman Construction, the Architects IU + Bibliowicz, the Structural Engineers Robert Silman Associates, and Steel Contractor Metropolitan Walters LLC. Andrew Carnegie was a steel tycoon and philanthropist. It is fitting that Carnegie is the first US building to make use of such an innovative steel floor.

The SPS Floors including supporting framing, for the 8M level, were installed during 8 hour shifts using a crew size of 5 (4 + site supervisor). The total installation period was 4 weeks.

The SPS Floor was also designed to act as a temporary roof for 3 months while the roof above was demolished and replaced. A DOW product was spray applied to the SPS Floor to provide weather proofing. The roof demolition contractor drove a bobcat with the load from the debris of the demolished roof above. SPS Floors have adequate capacity to support these loads.

The SPS Floor static design is based on the performance criteria for Load and Resistance Factor Design (LRFD), requirements prescribed in ANSI/AISC 360-05. Dynamic performance assessment has been evaluated in accordance with industry accepted practices using a rational based analysis approach as described in the design report. Analyses and design performed by Intelligent Engineering result in units that are fully compliant with static performance limits, vibration performance, and fire ratings. Standard structural steel materials used for SPS Floor fabrication conform to a European grade S235J0, which has a minimum yield strength of 34 ksi.

The following section describes the results of the static analysis and dynamic performance assessment for a typical bay of the 8th floor mezzanine level with approximate plan dimensions of 18 x 90 ft.

Design

The 8M SPS Floor is supported by structural steel framing which is suspended from hangers that are attached to the underside of the beams of the floor above (9th floor). The floor consists of 6 bays, each measuring approximately 18 ft by 90 ft and interrupted by the webs of trusses, as shown in Figure 11. Each floor bay consists of thirty-six 5 x 9 ft plates each weighing 789 lbs, which are arranged in two rows of 18 plates. The plate plan dimensions are restricted by the construction erection scheme, as the plates are lifted to location with a freight elevator with limited size and capacity. The SPS plates are composed of 1/8 inch faceplates and 3/4 inch polyurethane elastomer core, referred to as an SPS 1/8-3/4-1/8. Each plate is connected along its length to the top flanges of the secondary beams with ¼” countersunk tension control bolts spaced 24” on centre. A similar connection was made to tertiary beams, which are located at midspan of the secondary beams. The countersunk bolts provide a flush surface for floor finishes. The general arrangement is given in Figure 12.
The construction sequence was: remove existing 8M floor; install new 8M framing; install the 8M SPS Floor; apply a spray waterproofing system; demolish the roof while landing debris on the SPS; rebuild the roof; suspend the SPS on hangers connected to the roof structure; and apply finishes. The photograph shown in Figure 13 was taken during the SPS Floor installation, prior to the application of the waterproofing, which, in combination with the SPS Floor plate, would protect the ornate ceiling of Carnegie Hall during the summer of 2011.

**Dynamic Design Method and Assessment Criteria**

For this design, the floor R-factor was calculated and compared against the criteria given in the following standards:

- **AISC Steel Design Guide Series 11**: Peak acceleration for office floors and shopping malls shall be less than 0.5%g (R ≈ 7) and 1.5%g (R ≈ 20), respectively. Peak acceleration limits in accordance with AISC can be approximately converted to R-factor values with the following expression:

  \[
  R = \frac{200a_{\text{peak}}}{\sqrt{2}},
  \]

  where \(a_{\text{peak}}\) is expressed in m/s\(^2\), where a peak is expressed in m/s\(^2\). [Murray et al, 1997]

- **SCI Guide**: where the acceleration due to the static force exerted by an average person weighing 170 lb walking with a pace frequency of 1.8-2.2 Hz, must produce accelerations less than R = 8 for an office floor. [Smith et al, 2007]

- **TCC Guide**: where minimum footfall rates of 2 Hz are specified with a design criteria of R less than 8 for office floors. [Wilford et al, 2006]
The acceleration limit for office floors given in the AISC is 0.5%g, which when converted to an R-factor approximately gives R = 7. It is important to note that the limits are based on the human acceptance of vibration, which is subjective, and therefore it is not practical to use calculated R-factors in a binary pass-fail decision for the acceptability of the floor. It is necessary to exercise good engineering judgment in determining the acceptability of a floor in respect to the R-factor limits. For this design, given that R-factor values from the various standards for typical open plan office concepts are close to 8, this value was selected as the acceptability limit.

For the Route 1 method, R-factor values were found for the entire extent of the floor, indicating that the excitation and the response can occur anywhere.

For the Route 2 method, to calculate the critical R-factor values, the definition of the walking path is required. For this design, an open plan office space was assumed, where although walking paths are interrupted by furniture, moveable partitions, and equipment, a path was selected in order to determine the maximum response. Typically, a path that crosses through the centre of the floor bay gives peak R-factor values.

In accordance with the SCI guide, 10 percent of the live load was applied to the floor for the vibration calculations (5 psf). A constant damping value of 2.5% was used in the calculation.

**Structural Design**

Finite element models were used to determine the modal properties that are required for computing the acceleration response (R-factors) of floors. These same models were used to calculate the static behaviour of the floor.

Finite element models were developed using ANSYS 12.1, in accordance with the recommendations given in the SCI guide, with SHELL elements for the SPS plates and BEAM elements for the supporting steel framing, as shown in Figure 14. The hangers were modeled with LINK elements and are connected to the primary beams with contact elements. Walkways, with glass floors, join the 6 floor bays together. These glass floors are supported by built-up steel plates. These are included in the model as LINK elements that provide lateral support.

The boundary conditions, as shown in Figure 14, indicates that the top of the hangers are fixed against all translations. The walkway supports and the east end of the floor platform are restrained against translation.

The SPS plates were evaluated for conformance to service and ultimate design requirements described in Table 2. Finite element contour plots showing the peak deflections and stresses for SLS and ULS loading are given in Figure 15. The stresses in the SPS plates and supporting steel structure were found to be less than the limits:

- For SPS: 31 ksi (0.90F\text{\text{Y}}), where F\text{\text{Y}} = 34 ksi for grade S235 steel.
- For steel frame: 32 ksi (0.90F\text{\text{Y}}), where F\text{\text{Y}} = 36 ksi for grade A36 steel

**Table 2 - Summary of design criteria for SLS and ULS for Vertical Loads**

<table>
<thead>
<tr>
<th>Performance Requirement</th>
<th>Reference Standard</th>
<th>Load Combinations</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Static Load</td>
<td>ANSI/AISC 360-05</td>
<td>For SLS, 1.0L; the designer should consider the inclusion of specified dead load such as non-permanent partitions.</td>
<td>SLS Criteria: Typically deflections are restricted to L/360, where L is the span.</td>
</tr>
<tr>
<td></td>
<td>IBC 2009</td>
<td>For ULS, 1.2D + 1.6L</td>
<td>ULS Criteria: Steel stresses less than 0.9F\text{\text{Y}}.</td>
</tr>
</tbody>
</table>

Notes: Dead loads, D, were 18.3 psf for the SPS and 21 psf for the supporting steel framing. Superimposed dead loads of 48 psf, which accounts for the floor finishes and mechanical and electrical fit-out is included in the design. The live load, L, was defined as 50 psf.

The maximum relative deflection of the SPS deck under serviceability limit state (SLS) loading, as shown in Figure 15, is 2.7 mm (~1/16") or L/2030, which is well within the limits of L/360.
In summary, the analytical results indicate that the requirements for deflections and strength have been satisfied.

**Composite Action**

SPS Floors act compositely with the supporting steel beams to increase the beam’s effective stiffness and load carrying capacity. Pre-tensioned bolts are used to connect SPS floor panels to beams and distribute the shear from the SPS floor plate to beam, like shear connectors in concrete composite floors. The design of the bolted connection on SPS Floors:

- considers the development of the full plastic section of the supporting beam and SPS faceplates for full composite action (bolt shear resistance at steel plastic limit),
- achieves slip resistant (slip-critical) joints at service load levels, and
- ensures that if slip occurs beyond service loads that the bearing strength is adequate.

To calculate the size and spacing of bolts for composite action, the effective width of the SPS Floor plate contributing to the composite action shall be taken given below.

For an interior beam:

\[ b_{se} = \text{less} \text{er of } L_x = 59'' \]

or,

\[ b_{se} = 1.9t \sqrt{\frac{E}{F_y}} = 1.9 \times 1'' \times \frac{29000}{34} = 55.4'' \text{ governs} \]

These calculations indicate that a maximum bolt spacing of 29 inches (737 mm) is required to achieve full composite action. The bolts spacing was limited to 24 inches (610 mm) to ensure that joints remain at a constant elevation.

**Dynamic Analysis**

R-factor results (and thereby the accelerations), were determined in accordance with Route 1 and Route 2 methods using calculated modal properties of the floor from the same finite element model used for the static analysis. The fundamental frequency of the floor, including the superimposed dead and 10% live loads, was found to be 8.8 Hz.

Figure 16 shows a comparison of the R-factor, determined in accordance with Route 1 and Route 2 methods using the calculated modal properties. In accordance with the Route 1, the maximum footfall rate for office bays is 2.0 Hz. At this pace frequency, the calculated R-factor value is \( R = 3.6 \). R-factor calculations conducted using the Route 2 method, give a maximum R-factor value of \( R = 7.3 \), which is based on 1000 random pedestrians walking at an average pacing frequency of 1.87 Hz along the predetermined walking path.

This floor was designed for service in an open plan office. The design criterion for this application requires the R-factor to be less than 8. This floor meets this requirement and is deemed to be satisfactory.

**Carnegie Hall Project Summary**

The SPS 1/8-3/4-1/8 Floor has been used for the new 8M floor at Carnegie Hall. A global finite element model was developed to assess the response of the floor structure for strength (stresses) and serviceability (deflection and vibration). The analytical results indicate that all relevant requirements specified for the floor have been satisfied.

**SPS Terrace Development**

In 2007, also with the Building Research Establishment (UK) [BRE, 2007; BRE, 2007b; BRE Assessment Report CC 242565] and the University of Sheffield (UK) [Pavic et al, 2008], IE completed a series of large-scale tests on SPS Terraces to determine their dynamic behaviour and
performance. The terraces that were tested comprise of SPS treads and a steel rise, as shown in Figure 17. For instances where longer spans are desired, a dynamic reinforcing plate (soffit plate) was fastened to the underside of the risers to increase the stiffness and improve the dynamic response, as shown in Figure 18. The typical dimensions for the rise, tread, and span are given in Figure 19. The SPS is composed of 4 mm thick faceplates and a 20 mm thick core.

Figure 17 – SPS Terraces Configuration

Figure 18 - SPS Terrace Structural Configuration

At BRE, four configurations of SPS Terraces were tested, as summarized in Figure 19. The dynamic tests were conducted to determine the vibration properties (natural frequency, modal response, and the critical damping coefficient) of the empty terrace and, to determine the dynamic response (critical damping coefficient and accelerations) due to human-structure interaction with the terrace loaded with an increasing number of people. Dynamic load effects tested included controlled rhythmic crowd excitation tests where people bounced, jounced and jumped; and excitation resulting from dancing to rock and roll music. For the design of stadiums and arenas, accelerations caused by rhythmic crowd excitations are limited to the range of 10-18%g in NBCC [Can. Bldg. code Users Guide NBC, 2005]. The IStructE [Dynamic Performance Requirements for Permanent Grandstands Subject to Crowd Action, 2007] requirements for grandstands subject to crowd action limits the maximum RMS acceleration to 7.5%g for sporting events (scenario 3) and 20%g for high energy events such as pop/rock concerts (scenario 4).

The test results summarized in Table 3 indicate that with increasing the number of people on the terrace, the natural frequency remained almost unchanged, but the critical damping ratio increases considerably. This vibration response is in keeping with the concept of human-structure interaction, whereby the humans are not simply a rigid mass placed on the structure, but a mass-spring-damper system attached to the structure, as described in the IStructE guide and shown schematically in Figure 20. For the heel drop test one person was active while all the remaining crowd members were passive. The measured accelerations due to crowd excitations, where all members were active, are within the recommended limits of acceptability for vibrations due to rhythmic activities.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Rise mm</th>
<th>Run mm</th>
<th>Span m</th>
<th>Mass kg/m²</th>
<th>Soffit plate</th>
</tr>
</thead>
<tbody>
<tr>
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<td>180</td>
<td>840</td>
<td>10.0</td>
<td>137</td>
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</tr>
<tr>
<td>2</td>
<td>360</td>
<td>840</td>
<td>10.0</td>
<td>114</td>
<td>-</td>
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<tr>
<td>3</td>
<td>360</td>
<td>840</td>
<td>10.0</td>
<td>147</td>
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</tr>
<tr>
<td>4</td>
<td>360</td>
<td>840</td>
<td>7.5</td>
<td>114</td>
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</tbody>
</table>

Figure 19 – SPS Terrace Dynamic Testing at BRE
Table 3 - Summary of Terrace Dynamic Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Crowd size</th>
<th>Natural frequency, f&lt;sub&gt;n&lt;/sub&gt; (Hz)</th>
<th>Damping Ratio</th>
<th>Accl. %</th>
<th>MTVV&lt;sub&gt;p&lt;/sub&gt;</th>
<th>MTVV&lt;sub&gt;m&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>people</td>
<td>empty</td>
<td>%</td>
<td>crowd</td>
<td>%</td>
<td>m/s&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>1&lt;sup&gt;(a)&lt;/sup&gt;</td>
<td>80</td>
<td>7.5</td>
<td>5.3</td>
<td>4</td>
<td>19</td>
<td>6.0% at 1.5 Hz</td>
</tr>
<tr>
<td>2&lt;sup&gt;(b)&lt;/sup&gt;</td>
<td>80</td>
<td>8.0</td>
<td>8.5</td>
<td>3.4</td>
<td>13</td>
<td>15.1% at 2.8 Hz</td>
</tr>
<tr>
<td>3&lt;sup&gt;(c)&lt;/sup&gt;</td>
<td>24</td>
<td>7.1</td>
<td>7.5</td>
<td>1.7</td>
<td>12.0</td>
<td>-</td>
</tr>
<tr>
<td>4&lt;sup&gt;(c)&lt;/sup&gt;</td>
<td>48</td>
<td>10.1</td>
<td>10.1</td>
<td>0.5</td>
<td>11.3</td>
<td>2.5% at 2.5 Hz</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10.7</td>
<td>11.3</td>
<td>0.5</td>
<td>10.1</td>
<td>4.0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12.2</td>
<td>10.9</td>
<td>0.7</td>
<td>37.5</td>
<td>5.5% at 4.0 Hz</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11.0</td>
<td>13.8</td>
<td>2.4</td>
<td>34.0</td>
<td>6.2</td>
</tr>
</tbody>
</table>

Notes:
(a) Tests 1 and 2 were conducted by BRE(5) (6).
(b) Tests 3 and 4 were conducted at BRE by UoS(4).
(c) MTVV is the maximum root-mean-squared acceleration value.

Figure 21 - Test Predictions and Measured MTVV Values for SPS Terrace with a 7.5 m Span

The measured acceleration response agree well with analytical predictions and hence indicate that the crowd-structure interaction methodology described in IStructE is a suitable technique for the prediction of crowd-induced dynamic responses of grandstands with SPS Terraces.

Other SPS Terrace Certification Tests

SPS Terraces (stepped floors) fire resistance performance has been verified by test with designs meeting 1 and 2 hour endurance periods. Tests were conducted in accordance with both the European Standard BS EN 1365-2 and the American Standard ASTM E119 (UL 263) by BRE Global Ltd. Similar to SPS Floors, SPS Terraces have been tested together with various types of underside protection [BRE Assessment Report CC 242565].

Alexander Memorial Coliseum (GaTech)

Introduction

The Alexander Memorial Coliseum (AMC) Replacement Facility Project in Atlanta, Georgia is a renovation project for an indoor basketball arena. SPS Terraces, Stair Access Platforms and Ramps have been specified for use on the new upper bowl. The project team comprises:

- Architect: Populous
- Engineer: KSi Structural Engineers
- Contractor: Whiting- Turner
- Steel Fabricator & Erector: Schuff Steel

Georgia Tech approved a $45 million renovation project for the Alexander Memorial Coliseum, which will be renamed to the Hank McCamish Pavilion. The facility is located on the northeast end of the campus and is the home of the Georgia
Tech Yellow Jackets basketball team. It has also served as the boxing venue during the 1996 Summer Olympics, as the temporary home of the Atlanta Hawks for five seasons, and occasionally hosts the women’s volleyball team during the NCAA tournament. The arena has a listed capacity of 9,191. However that number will be reduced to 8,900 after the renovations have been completed.

The renovation project will include small changes to the outside of the arena with the addition of greenery and park-style space. Inside, the court will remain the same, however the current concourse walls will be knocked down to give spectators more opportunities to see the court while walking to washrooms or concession areas. The demolition and reconstruction of the Alexander Memorial Coliseum began the summer of 2011, and the renovation work is expected to be completed in time for the 2012-2013 basketball season. This renovated arena will comprise a steel frame structure supporting the SPS Terraces. Images of the current and renovated arena are shown in Figure 22.

### Static Design

SPS Terrace sections are composed of two folded-Z-shape plates, which when assembled provide SPS \( \frac{5}{32}'' - \frac{3}{4}'' - \frac{5}{32}'' \) tread and two \( \frac{5}{32}'' \) thick overlapping plates that form the rise. Once welded, the SPS cavity is injected with polyurethane, which flows into the SPS cavity with the consistency of motor oil and cures within minutes to form a solid elastomer. The complete terraces are delivered to site painted and shop-bolted together, with \( \frac{5}{8}'' \) Tension Control (TC) bolts in a friction grip connection along the overlapping rise plates, into 2 tiered units, where possible.

The SPS Terraces are designed with a standard, double rise bolt spacing of 18” on centers. The pre-assembled 2-tiered SPS Terraces are placed on the raker stools, bolted down and bolted together to form an entire bank. All SPS units are bolted to the supporting steel framing using one sided fasteners, Lindapter Hollo-Bolts. The Hollo-Bolt is a multi-part fastener, which when tightened causes the cone to compress a sleeve which splays open to grip the connecting steel plates. The following was evaluated as part of the structural design of the SPS Terraces:

- Normal stresses in the SPS steel faceplates was determined in order to establish the thickness and steel grade. The shear stresses at the steel-elastomer interface are calculated to ensure that the bond strength resistance is not exceeded.
- Rise-to-rise and tread-to-stool connections
- Lindapter 1/2” Hollo-Bolts as shown in Figure 23 are used for the SPS Terrace rise to rise connection. These bolts are evaluated for their shear and tensile resistances.
- Lindapter 5/8” Hollo-Bolts as shown in Figure 24, are used for the tread-to-stool connections. These bolts are evaluated for their shear and tensile resistances.
- The SPS Terraces are sealed with rise-to-rise welds and tread-to-perimeter bar welds. The welds are evaluated for their shear and tensile resistances.

### SPS Terraces Performance (SLS, ULS)

- The upper bowl configuration will use 16 tapered bays of risers. The SPS \( \frac{5}{32}'' - \frac{3}{4}'' - \frac{5}{32}'' \) Terraces will be lifted in groups of 2 units and bolted to the steel raker beams through steel stools made of channel sections.

The following section describes the structural and dynamic assessment of the SPS Terraces and supporting steel framing for this project.
Design Loads

Design considerations and parameters include:

- **ASCE/SEI 7 – 1.3.2 Serviceability**: Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of buildings and other structures.
- **NFPA 5000 – 35.1.2.8 Deflection**: Deflection limits shall not exceed L/360 for floor members under an applied unfactored live load. L/240 was applied for the design in accordance with the Project Specification.
- **ASCE/SEI 7 – Combining Factored Loads using strength design as applied to the SPS components are as follows**: 1.2D + 1.6L

All the other load combinations are excluded as they do not govern the design, and for indoor structures, snow loads, wind loads, and rain loads are not considered.

- **ASCE/SEI 7 – Table 4-1**: For reviewing stands, grandstands and bleachers, the minimum uniformly distributed live load is 100 psf.
- **Project Specification - Superimposed dead load accounting for seat and footboard, risers and steel framing is 6 psf**.

**Structural Analysis**

Static analyses have been conducted using Finite Element models of the SPS components using ANSYS FEA software. Both the ULS and SLS states have been considered for erection and occupancy conditions.

The static analysis gives the deflections, normal steel stresses, interlaminar shear and normal stresses in the elastomer core, bolt forces in the rise-to-rise lap plate connection, the weld forces for the fillet welds joining the SPS faceplates, and the bolt forces in the terrace-to-stool plate connection.

A local model of the stands in between K1 and K5 with a terrace rise and run of 24.5” and 33” was built and is presented in Figure 25. The geometry, boundary conditions, loads and other modeling details are described as follows:

**Figure 25 - Finite Element Model Description for the Stands between K1 and K5**

**Geometry**

- The rise-to-rise bolted connection starts at 6” (152 mm) from each end and bolts within the span are at 18” (457 mm) centres.
- The terrace-to-stool bolt location is kept at a constant value of 2-1/8” (54 mm) from the end of tread and 6” (152 mm) from the terrace nose.

**Modelling**

- The material properties and element types used in the finite element analyses are below.

<table>
<thead>
<tr>
<th>Material Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Steel</strong>:</td>
</tr>
<tr>
<td>E = 29 000 ksi, ( \nu = 0.287 ), ( \rho = 490 \text{ lb/ft}^3 )</td>
</tr>
<tr>
<td>( \sigma_{\text{yield}} = 34 \text{ ksi} )</td>
</tr>
<tr>
<td><strong>Elastomer</strong>:</td>
</tr>
<tr>
<td>E = 7110 ksi, ( \nu = 0.36 ), ( \rho = 7016 \text{ lb/ft}^3 )</td>
</tr>
<tr>
<td><strong>FEA Element type</strong>: SHELL181/ SOLID185</td>
</tr>
</tbody>
</table>

- For the SPS core SOLID elements, 4 elements through the core thickness were used. Plan element dimensions were, for the most part, 4” square. For the SPS tread faceplates, SHELL elements are attached to the SOLID core elements. SHELL elements are also used to model the rise plates and the stools.

**Boundary Conditions**

- All bolted connections are modeled by coupling coincident nodes on joining parts.
Table 4. The method of analysis utilizes the principles of crowd-structure interaction, as corroborated by the tests conducted on SPS Terraces at the BRE laboratory [Pavic et al, 2008]. The designer must calculate the maximum root mean square (RMS) acceleration of the seating deck and compare the results with the RMS limits for the different design event scenarios described in Table 4. The design calculations assume that sections of crowds are idealized as being either predominantly active (standing and generating a rhythmic motion) or passive (act as a mass dampers). For the calculations, the fundamental frequency is reduced by 0.5 Hz to account for the variation between calculated and measured values of natural frequency in accordance with Clause 4.3 of IStructE.

### Table 4 - Event Scenarios and Criteria for Design

<table>
<thead>
<tr>
<th>Event Scenario</th>
<th>Example Event</th>
<th>Crowd Behaviour</th>
<th>Acceptance Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Stand used for viewing sporting and similar events with less than maximum attendance</td>
<td>Normally relaxed viewing public with spontaneous response to single events</td>
<td>Route 1 with 3.5 Hz minimum or accepted at the discretion of a Listed Engineer</td>
</tr>
<tr>
<td>2</td>
<td>Classical concert and typical well attended sporting events</td>
<td>Audience seated with only few exceptions – minor excitation</td>
<td>Route 1 with 3.5 Hz minimum. Not normally adequate but otherwise Route 2 and 3%g maximum RMS acceleration</td>
</tr>
<tr>
<td>3</td>
<td>Commonly occurring events including, inter alia, high profile sporting events and concerts with cross generational appeal</td>
<td>Potentially excitable crowd with crowd participation</td>
<td>Route 1 with 6 Hz minimum. Route 2 an 7.5%g maximum RMS acceleration</td>
</tr>
<tr>
<td>4</td>
<td>More extreme events including high energy concerts with periods of high intensity music</td>
<td>Excited crowd, mostly standing and bobbing with some jumping</td>
<td>Route 1 with 6 Hz minimum. Route 2, and 20%g maximum RMS acceleration</td>
</tr>
</tbody>
</table>

The Route 2 approach, described in the IStructE guide is used to estimate the performance under dynamic loadings specified for the different classes of activity and size of crowd as described in Table 4. The designer must calculate the maximum root mean square (RMS) acceleration of the seating deck and compare the results with the RMS limits for the different design event scenarios described in Table 4. The design calculations assume that sections of crowds are idealized as being either predominantly active (standing and generating a rhythmic motion) or passive (act as a mass dampers). For the calculations, the fundamental frequency is reduced by 0.5 Hz to account for the variation between calculated and measured values of natural frequency in accordance with Clause 4.3 of IStructE.

### Dynamic Assessment

There are two dynamic assessment approaches prescribed in the IStructE Dynamic performance requirements for permanent grandstands subject to crowd action.

In the Route1 approach, the fundamental frequency of an unloaded grandstand is limited to specific values depending on four performance based scenarios (rated 1-4) that correspond to increasing crowd involvement and activity together with increased loading (Table 4). For this design criteria, stadia with a minimum fundamental frequency of 3.5 Hz, are suitable for scenarios 1 and 2 where the audience is predominantly seated and stadia with a minimum fundamental frequency of 6 Hz are suitable for scenario 3 (concerts and high profile sporting events) or scenario 4 (high energy events such as a pop/rock concerts).

The same model used for the static analysis is used to calculate the frequency and acceleration response.

As shown in Figure 26, the fundamental frequency is 7.7 Hz, which exceeds the limit of the frequency based assessment method (> 6 Hz) and thereby is suitable for high energy events.
Figure 26 – Frequency and Mode Shape

Acceleration Assessment (SLS)

Figure 27 gives the acceleration response for the SPS stands for IStructE event scenarios 2, 3, and 4 where crowd-structure interaction is considered. In each case, the acceleration response is less than the limit for typical excitation frequencies. A damping ratio of 1% was used for the calculations.

Conclusions & Design Guidelines for Future Projects

This technical paper has summarized the development work for SPS Floors and Terraces and gave design examples for two applications, where the use of SPS allowed engineers and architects to revitalize their existing steel frame structures with minimal impact on the existing framing. For new construction, SPS Floors and Terraces can provide significant benefits including:

SPS Floors
- weigh one quarter of comparable concrete flooring, they are supported by light frames and foundations;
- are delivered to site finished to the highest tolerances ready to be assembled using standard steel working practices;
- reduce construction schedules;
- provide 100% working load capacity as soon as they are fixed in position and with no wet-work above ground level work can start earlier and be completed faster;
- reduce health and safety risks by providing increased protection against falling objects and minimizing a range of onsite hazards;
- reduce contractors’ costs associated with time on site and project risk, and
- provide increased revenue for developers from earlier project completion and extra floors on tall buildings.

SPS Terraces
- weigh less than 25% of concrete terraces which enables a 25% reduction in structural frame weights and 15% lighter foundations;
- can be erected quickly with 6-10 times more units being transported per truck and 3-4 times more units lifter per hoist than concrete alternatives;
- project cost savings of over 20% are achieved, and
- they can be readily demounted and reused on future venues.

To aid designers in adopting SPS for renovation of existing building or for design of new structures, design guidelines have been developed and are readily available [Engineering Design Guidelines for SPS Floors, 2010; Engineering Design Guidelines for SPS Terraces, 2010]. Each guideline document includes:

- A description of the application (products)
- A description of the materials and the structural configuration
- Standard details

SPS Terraces provide a lightweight alternative to precast terraces with excellent dynamic performance and an improved finish. The lightweight section pre-assembled into doubles or triples allow fast erection times.
- Design criteria and FEM recommendations
- Computer software routines for dynamic design evaluation.

Based on these engineering guidelines, design tables have been developed for SPS Floor and Terraces, as shown in Table 5 and Table 6. These tables provide designers with SPS component sizes, faceplate and core thicknesses, for typical spans of Floors (unsupported widths) and Terraces (spans). Designers can use the component dimensions for preliminary designs, but for final design the more rigorous engineering calculations must be conducted to establish a suitable structural configuration.

### Table 5 – Design Guideline for SPS Floors

<table>
<thead>
<tr>
<th>Materials</th>
<th>Faceplates: Steel (34 ksi-50 ksi), stainless steel or aluminium are available. Core: Polyurethane (fillers are used to lighten core when appropriate).</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometry</td>
<td>Typically rectangular (cut-outs and curved edges possible).</td>
</tr>
<tr>
<td>SPS Specification</td>
<td>3-20-3</td>
</tr>
<tr>
<td>Length (ft)</td>
<td>Typical Length : 15’ – 40’ (smaller or larger sizes can be provided)</td>
</tr>
<tr>
<td>Width (ft, typical)</td>
<td>Up to 5'</td>
</tr>
<tr>
<td>Thickness (inches)</td>
<td>1&quot;</td>
</tr>
<tr>
<td>Weight [1] (lb/ft²)</td>
<td>17.2</td>
</tr>
<tr>
<td>U Value (Btu/ft²oF)</td>
<td>0.678</td>
</tr>
</tbody>
</table>

Note 1: Unit weight varies with panel size and edge detail. Weights listed are typical for panels with associated SPS specification.

### Table 6 – Design Guideline for SPS Terraces

<table>
<thead>
<tr>
<th>Materials</th>
<th>Faceplates: Steel (34 ksi-50 ksi) Core: Polyurethane (fillers are used to lighten core when appropriate).</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometric Possibilities</td>
<td>SPS Terraces are custom made to suit each venue with geometries designed to suit each area of the bowl including facetted corners and parabolic bows.</td>
</tr>
<tr>
<td>Span (ft)</td>
<td>20</td>
</tr>
<tr>
<td>Rise (in.)</td>
<td>8</td>
</tr>
<tr>
<td>Tread (in.)</td>
<td>33&quot; for all scenarios shown (different tread widths can be provided)</td>
</tr>
<tr>
<td>SPS Specification</td>
<td>4-20-4</td>
</tr>
<tr>
<td>Steel Grade</td>
<td>355</td>
</tr>
<tr>
<td>Weight [1] (lb/ft²)</td>
<td>22</td>
</tr>
<tr>
<td>Natural Frequency [2] (Hz)</td>
<td>10</td>
</tr>
<tr>
<td>Deflection (L/-)</td>
<td>400</td>
</tr>
</tbody>
</table>

Note 1: Unit weight is quoted as 1b/ft² on plan and varies with terrace geometry and specification.

Note 2: Natural frequency is for the bank of SPS Terraces only and must be combined with the natural frequency of the supporting frame to assess the natural frequency of the whole system.
References

1. Institute of Structural Engineers (IStructE). (2008). “Dynamic Performance Requirements for Permanent Grandstands Subject to Crowd Action – Recommendations for management, design and assessment”.


6. BRE (Report 240274): “Structural performance for grandstand unit (stadium riser)”

7. BRE Assessment Report CC 242565: “Fire Performance for grandstand unit (stadium riser)”


